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Fabrication and Erection Techniques for Extra-Heavy Steel High-Rise Structure, Shenzhen Development Center, China

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Fabrication and Erection Techniques for Extra-Heavy Steel High-Rise Structure, Shenzhen Development Center, China*



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1 Introduction

With the increased height of buildings, main structural members are required to sustain increasingly high tensile forces resulting in the adoption of extra-heavy sections. Heavy steel members of up to about 100-mm thickness have been used in Japan, and structural steel buildings using heavier steel members exceeding 100 mm have already been constructed in other countries, in particular in Southeast Asia. The use of these extra-heavy steel frames became possible only when sophisticated welding and welding control techniques, as well as steel products with excellent weldability, became available.

The Engineering and Construction Division of Kawasaki Steel Corporation participated in the construction of the high rise Shenzen Development Center in the People's Republic of China. Extra-heavy members 64 to 130 mm in thickness were used as box-section columns, and extra-heavy members 64 to 127 mm in thickness were used for box-section girders. Since these main members cannot be field connected with high-tensile bolts, all were welded. In the steel-frame fabrication of extra-heavy members, which was an unusual case, two great technical problems were anticipated: shortening the fabrication period for field welding, because of the very large amount of welding required, and the establishment of a welding procedure capable of producing sound field welds while ensuring highly accurate erection.

A newly developed two-consumable-nozzle electroslag (CES) welding method featuring very high efficiency

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was used for girder connections at columns, where the amount of welding work is concentrated in the shop fabrication of steel frames.

Field welding procedure tests were conducted using full-size column-to-column, girder-to-girder, and column-to-girder connections, and the welding deformation behavior and the incidence of internal weld defects were confirmed. On the basis of these tests, an appropriate field welding procedure and a method of repairing defects were established. The construction was carried out according to these execution procedures and in May 1987 the first high-rise structure with extra-heavy steel members in China was finished.

This paper describes the fabrication of the extraheavy steel frames and the execution of the field work which was carried out in constructing the high-rise Shenzhen Development Center.

2 Outline of Work

The Shenzhen Development Center was constructed in the Shenzhen Economic Development Zone of the People's Republic of China adjacent to Hong Kong. An outline of the building statistics is given in **Table 1**. The building is large cylindrical high-rise building about 160 m tall and is the highest steel-frame structure in China. A typical floor plan is shown in **Fig. 1**. The structure comprises reinforced concrete load-bearing walls and a steel frame. In order to provide rigidity for these members, five main steel-frame columns were arranged in a fan configuration opposite to the load-bearing wall; the unique structural feature of the building. The five columns are made up of extra-heavy steel plates with a maximum thickness of 130 mm in a box section with girder members that form rigid frames also fabricated from extra-heavy steel plates with a maximum thickness of 127 mm (flange) in a box section. These steel plates are fabricated from steel grades ASTM A572 Gr. 42 and Gr. 50. The AISC (Specification for the Design, Fabrication and Erection of Structural Steel for Buildings) was adopted as the design standard and AWS D1.1 was applied for the welding.

Table 1 Outline of Shenzhen Development Center Building

Client	Shen Han Enterprise LTD.
Basic design	CRS Corporation (U.S.A.)
Structural design	Watson Architectual & Engineering Designing Consultants
Constructor	China Construction 3rd Division 1st Company
Use	Hotel, office, exhibition hall, etc.
Floors	1B + 41F, heliport (roof top)
Height	≒160 m
Structure	Steel frame + reinforced concrete load bearing wall
Steel frame weight	≒10 500 t
Structural steel specification	ASTM A572 Gr. 42, 50
Maximum plate thickness	Column 130 mm, girder 127 mm
Steel fabrication	Kawasaki Steel Corp. (Yokogawa Bridge Works LTD., Komai Bridge & Steel Engineering Inc., and Shikoku Iron Works Co., LTD.)

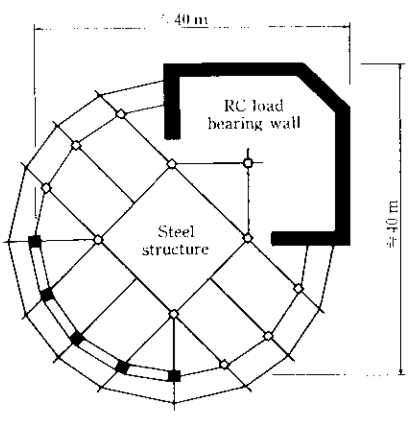


Fig. 1 Typical floor plan

3 Shop Fabrication of Extra-Heavy Steel Frames

All shop fabrication of steel-frame members was carried out in Japan. Among the shop fabricated steel structures, the fabrication of box-section columns required the highest technology and the most fabrication processes. In order to increase the efficiency of fabrication, the deep-penetration submerged arc welding method (the KX welding method) was applied to corner seams and the consumable electroslag (CES) welding with a large heat input was used in welding beam penetrating joints. The safety of these welded joints was verified based on various data related to joint property with these welding methods.

The joints produced by CES welding are shown in **Fig. 2**. They are beam penetrating T-joints. The throat

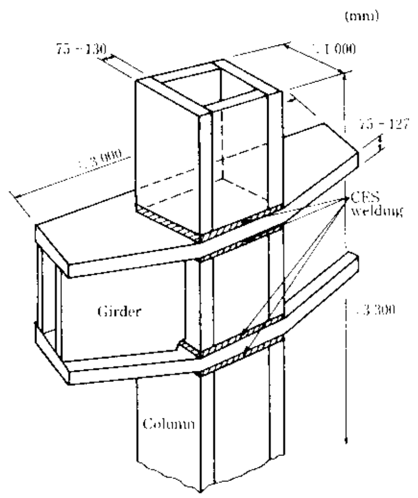
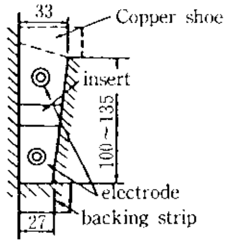


Fig. 2 Joint using two consumable nozzles electroslag (CES) welding

Table 2 Standard procedure specification for CES welding of column-girder joint

Joint detail (mm)	
Welding condition of both electrodes	
Welding current	430A~550A
Welding voltage	37V~40V
Wire	KW-50C, 3.2 mmϕ
Consumable nozzle	KU-1 000, 12 mmϕ
Flux	KF-100

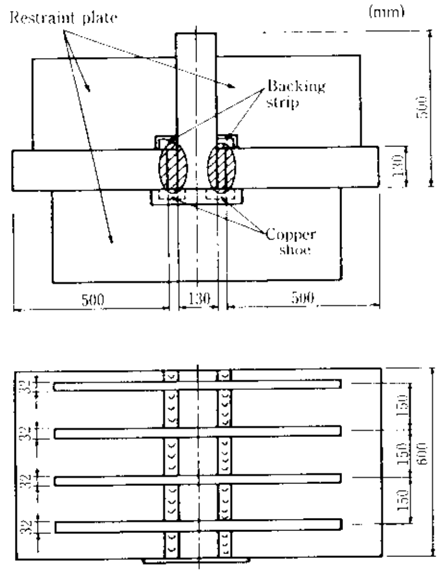


Fig. 3 Crack test specimen for CES welding

Table 3 Mechanical properties of CES welded joint

Tension test for deposited metal*[1]			Tension test for welded butt joint*[2]		V-notch charpy impact test*[3], $_vE_0$ (J)			
YS (MPa)	TS (MPa)	El (%)	TS (MPa)	Location of fracture	Weld metal	Fusion line	HAZ	Base metal
451	590	26	495	base metal	79	45	56	115

*[1] Test piece: JIS Z3111-A2 *[2] Test piece: JIS Z3121-1 *[3] Location: $t/4$ (Test piece: JIS Z2202-4)

thickness, i.e., the thickness of the column skin plate is a maximum 130 mm. The two-electrode one-pass process was adopted for CES welding. In order to establish optimum welding conditions, welding experiments were conducted using groove profile and welding conditions as parameters. As a result, the welding conditions for the two-electrode CES method shown in **Table 2** were obtained. Steel backing strips were used on the back side and water-cooled copper strips were used on the front side. Inserts were placed in the grooves in order to reduce the amount of metal deposited, increase the welding speed, and prevent welding deformation. The mechanical properties of CES welded joints are shown in **Table 3**. Laminations (laminated cracks) in the girder flange serving as the sandwiched plate were considered a potential problem because of the high level of restraint in the connections during actual welding. A CES weld crack test was therefore conducted using the specimen shown in **Fig. 3** in which the connection is modelled. The soundness of the weld was checked ultrasonically and a macroetching test of the section was made with no abnormalities detected. An example of the macroetching specimen is shown in **Photo 1**.

Photo 2 shows how the beam penetrating joint is constructed using CES welding. The efficiency of pre-

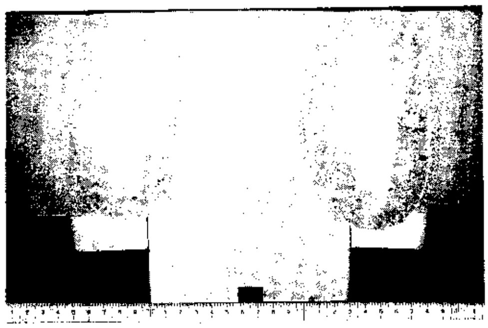


Photo 1 An example of macro-etch specimen for CES welding crack test

Photo 2 Two wire CES welding for column-to-girder joints

fabrication of extra-heavy steel frames is substantially increased by adopting this welding method.

4 Field Welding of Extra-Heavy Steel Frames

4.1 Field Welding Experiment

Connecting steel-frame members by field welding is broadly classified into three types, as shown in **Fig. 4**: column-to-column, girder-to-girder, and column-to-girder connections. As shown in **Table 4**, the total number of connections was high at 1 700 and the amount of welding materials used totalled about 40 t. For this reason, it was decided to adopt GMAW (shielding gas: CO_2) completely for the field welding of the steel building frames, although this welding method had rarely been used in China.

There was, however, no example in which steel frames using extra-heavy steel plates with a thickness as great as 130 mm had been field welded, making it necessary to examine field welding procedures in order to ensure high erection accuracy and sound connections. Before the field welding was executed, field welding procedure tests were conducted using full-size models of the box-section column-to-column connection and the box-section girder-to-girder connection. The major test items were: (1) the appropriateness and soundness of the welding procedure, (2) the type and position of weld defects that may occur in the field welding of extra-heavy steel-frame members and preventive measures, (3) methods of welding repairs of defects, and (4) amounts of deformation from welding in each connection and measures to ensure erection accuracy. The test specimens of the connections are shown in **Figs. 5** and **6**. The test specimens of extra-heavy steel plates conformed to the specifications shown in Table 5; the same specifications were used in the actual work. Welding procedure tests using full-size models were also carried out for the column-to-girder connection, which are not included in this paper.

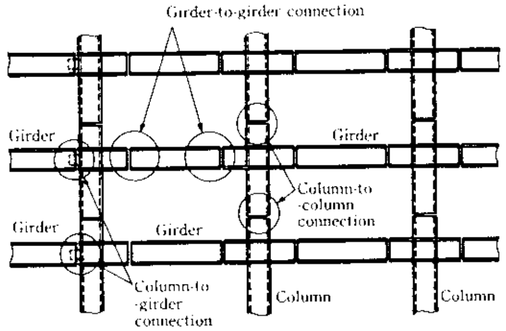


Fig. 4 Types of field connection

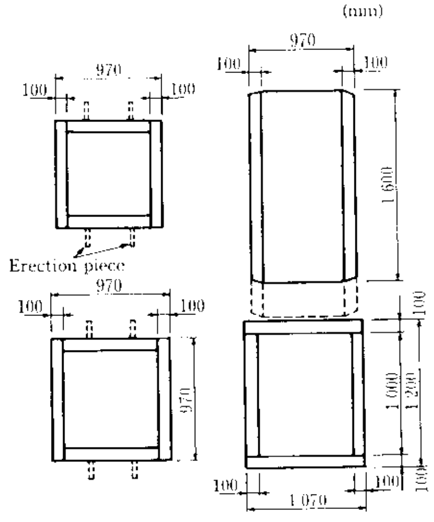


Fig. 5 Test specimen of column-to-column connection

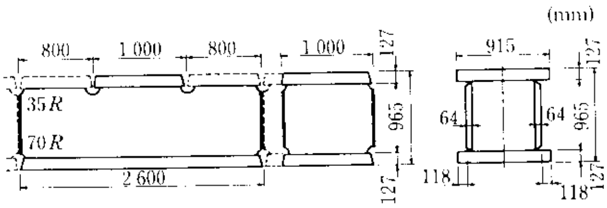


Fig. 6 Test specimen for girder-to-girder connection

The key points of the field welding procedure established by these tests are stated below.

Column-to-column connections:

(1) Welding operation is conducted according to the

Table 4 Type and number of field connection

Connection	Shape of member	Steel grade	Plate thickness (mm)	Penetration*1	No. of field connection
Column to column	□, H	A572 Gr. 42	64-130	FP or PP	268
	H	A572 Gr. 50	18.1-124.7	PP	111
Girder to girder	H, □	A572 Gr. 42	75, 127(F)-75, 64(W)	FP	128
	H	A572 Gr. 50	40-75	FP	32
Column to girder	H	A572 Gr. 50	13-43	FP	≒1 200

*1 FP: Full penetration PP: Partial penetration

Table 5 Test results of chemical composition and mechanical properties of base metal

Specification	Plate thickness (mm)	Chemical composition (%) C	Si	Mn	P	S	Nb	Tension test YS (MPa)	TS (MPa)	El (%)	Notes
ASTM A572 Gr. 42	100	0.13	0.34	1.26	0.017	0.005	0.012	333	493	36	□-column skin plate
	127	0.13	0.31	1.29	0.020	0.005	0.011	292	485	33	H-girder flange
	64	0.13	0.34	1.27	0.018	0.014	0.015	310	499	36	H-girder web
Requirement	>38.1	<0.21	0.15~0.40	<1.35	<0.04	<0.05	0.005~0.05	>289	>414	>20	Type-1

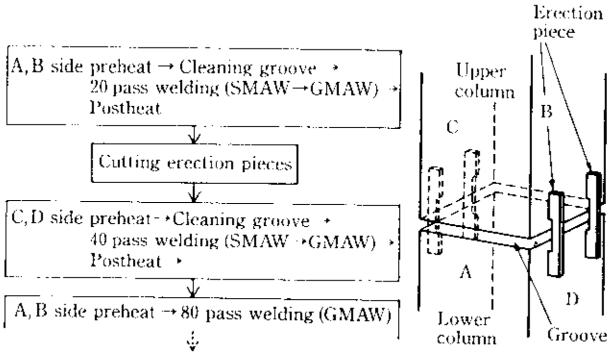


Fig. 7 Welding process for column-to-column connection

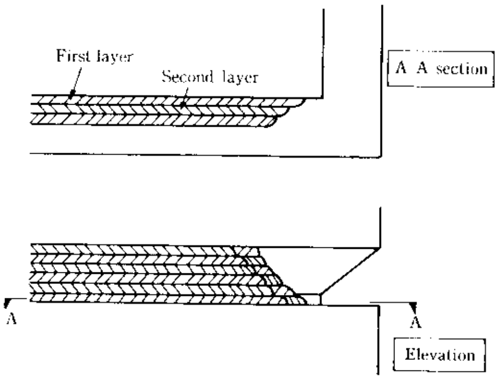


Fig. 8 Laying process of weld beads for column-to-column connection

procedure shown in **Fig. 7**.

(2) In the root, where weld defects are apt to occur, SMAW is used for the first to third layers until the GMAW welding torch can be properly positioned within the groove.

(3) In this connections, which are made in the horizontal position, inadequate fusion, as shown in **Photo 3**, along the upper wall of the groove is possible. To prevent this, spatter and slag are removed before the final pass for each layer.

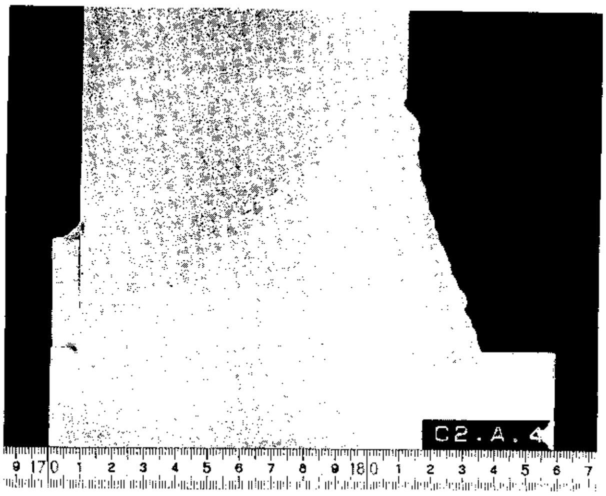


Photo 3 An example of lack of fusion at groove face of upper column

(4) Slag inclusions and lack of fusion frequently occur in the connection zone of welding beads. As shown in **Fig. 8**, therefore, the connection of welding beads are not concentrated in the same section.

(5) At corners of box-section columns, where defects may occur, boxing is conducted without breaking the arc.

(6) The two symmetrical surfaces of the columns are welded simultaneously by two welding operators to prevent column deformation by inclination.

(7) A shrinkage of 2.5 mm in the axial direction of the column due to welding is expected (**Fig. 9**), so this length is added when the columns are prefabricated.

Girder-to-girder connections:

(1) Girders are connected using the welding sequence shown in **Fig. 10**.

(2) Flat position welding, which is both reliable and efficient, is conducted from inside the girders. (This welding must be conducted in a narrow space for butt joints of the lower flanges.)

(3) SMAW, which ensures positive welding, is used for vertical butt joints of the webs and overhead butt joints of fillets, where the volume of welding was relatively small.

(4) High efficiency GMAW was adopted for flat butt joints of flanges, where the volume of welding was large. In the root, where defects occur, however, SMAW was used for the first to third layers.

(5) In the welding of the lower flange, defects are concentrated in the bead connection zone if welding is

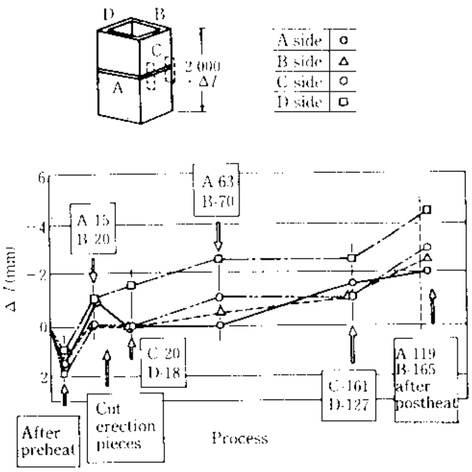


Fig. 9 Shrinkage Δl due to welding of column-to-column connection

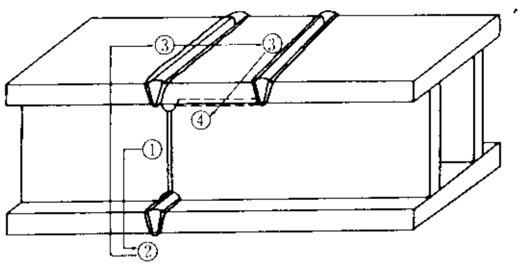


Fig. 10 Weld sequence for girder-to-girder connection

conducted separately for the interior and exterior of the box-section girders. Therefore, two welding operators carry out the operation simultaneously in and outside the girders in order to prevent differences in layer levels (**Photo 4**).

(6) The large bending deformation in the vertical direction of the girders which occurs due mainly to differences in the volume of welding between the

Photo 4 Welding lower flange of girder

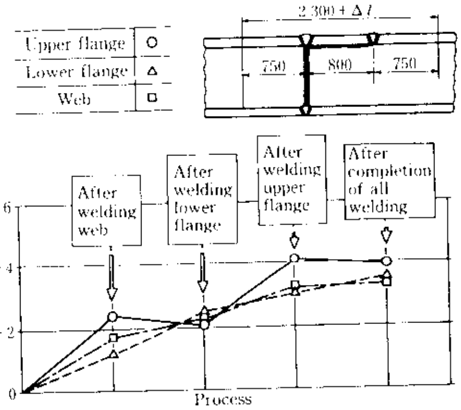


Fig. 11 Shrinkage due to welding of girder-to-girder connection

upper and lower flanges is about 0.5 mm. Large bending deformation in the horizontal direction can be virtually prevented if two welding operators butt weld the right and left webs simultaneously.

(7) The shrinkage in the axial direction of the girders due to welding is about 3.5 mm. Half the shrinkage is generated from the initial butt welding of the web (**Fig. 11**). This shrinkage is therefore taken into consideration in advance of the welding.

4.2 Execution System for Field Welding

About 100 welding operators were required for the field welding. In China, however, welders with experience in the field welding of steel building frames and GMAW were not available. For this reason, welding engineers and ten selected welding operators were trained in Japan. These ten welding operators then trained other welders in China.

The welding operation time for field connections is very long—about 55 h for a column-to-column connection by two welders (plate thickness, 130 mm; the amount of welding materials used, about 250 kg/connection) and about 35 h for the girder-to-girder connection in the box section by two welders (flange thickness, 127 mm; the amount of welding materials used, about 150 kg/connection). In the field welding of extra-heavy steel frames, it is necessary to minimize interruptions of operation in considerations of the temperature control of the welded joints. In order to maintain continuous, round-the-clock operation until the connections were completely welded, a three-shift system was adopted. Each group was composed of nine persons: one welding controller, two welders, two assistant workers and four preheating workers (for maintaining the interpass temperature and protecting weld joints during the interruption of welding due to rain).

Photo 5 shows a column-to-column connection being field welded.

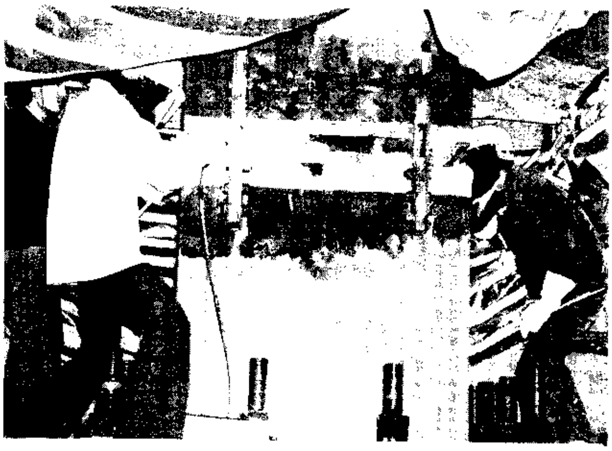
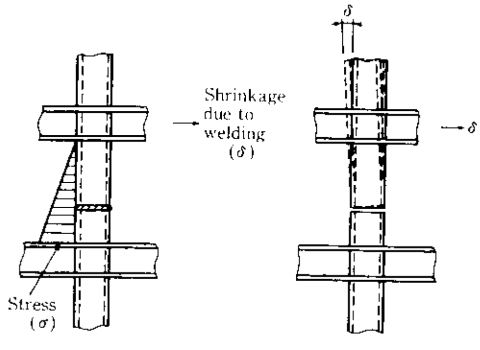


Photo 5 Field welding of column-to-column connection

4.3 Sequence of Field Welding and Measures to Prevent Welding Deformation

In the field welding, the stress conditions imposed on the steel frames subject to welding deformation changes depended on the sequence of column-to-column connection and girder-to-girder connection. Generally the sequence of execution of field connections was similar from the lower to higher floors. Stresses exceeding the design value were generated in the bottom portion of columns whose connections were already welded, as shown in **Fig. 12** (a), owing to the shrinkage of the girders in the axial direction from girder-to-girder welding. In contrast, when girder-to-girder welding was conducted on the upper floor before column-to-column welding, the connection between the columns was considered a pin-connection, as shown in Fig. 12 (b), and the above-mentioned stresses were minimal. Based on this, the latter sequence of field joining, i.e., the sequence shown in **Fig. 13**, was adopted so as to avoid added stress resulting from the welding of the extra-heavy members during the field welding and thereby minimize residual stress.



(a) In case of girder-to-girder welding after column-to-column

(b) In case of column-to-column welding after girder-to-girder

Fig. 12 Influence of difference in welding sequence on residual stress and incline of column

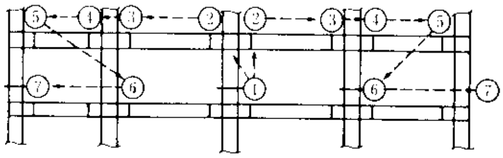


Fig. 13 Sequence of field connection

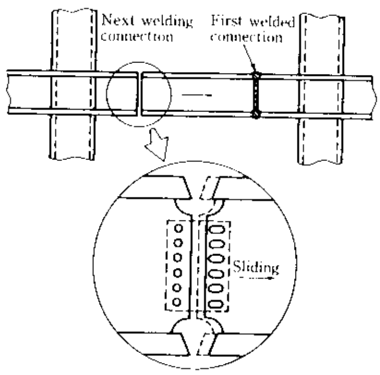


Fig. 14 Consideration of shrinkage due to welding in girder-to-girder

The deformation of steel frames due to field welding is mainly due to the shrinkage of the floor height from column-to-column connections, the shrinkage between the girders due to girder-to-girder connection, and the inclination of the columns associated with the latter shrinkage. To cope with the former shrinkage, the columns were shop fabricated with an excess length of 2 mm and fine adjustment was made by inserting liners as required in the connection between the columns during the erection of the upper column, observing measured levels in the field. The latter shrinkage involves two connections between the girders. To cope with the welding shrinkage in the first connection, the erection bolt holes in the girder web on one side of the connection were kept loose and the girder proper was moved into place as shown in **Fig. 14**. The welding shrinkage in the second connection was compensated for by the inverse inclination of the column provided during erection.

Results of the measurement of the inclination of the columns of the extra-heavy steel frames after field welding are shown in **Fig. 15**. The degree of inclination of the columns was high at the fourth-floor level of the side columns (columns C1A and C1B) where the welding shrinkage in the axial direction of the girders was concentrated, although deformation was within the allowable range on higher floors. This is because in the girder-to-girder connections on the fifth and higher floors, the inverse column inclination, which was insufficient during erection, can easily be obtained from the loose erection bolt holes in the web. This also is due to the welding shrinkage occurring in the first girder-to-girder connection which is compensated for by

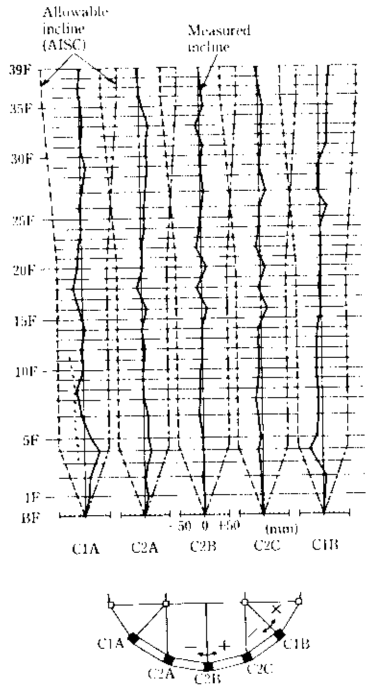


Fig. 15 Actual incline of column after welding


Photo 6 View of steel structure under construction

the loose bolt holes in the other connection, thus minimizing the column inclination. An view of the construction of the structure is shown in **Photo 6**.

5 Conclusions

Welding experiments were conducted on the steel-frame work of the Shenzhen Development Center, which is a high-rise building using extra-heavy members, and the construction work was satisfactorily carried out. The following results were obtained:

(1) In shop fabrication, deep-penetration SAW (KX welding method) was applied to corner seams of box-section columns in order to increase the efficiency of shop fabrication.
(2) A two-electrode on-pass CES welding method was developed and used for beam penetrating joints of the box-section columns.
(3) The crack resistance of connections made using these welding methods is good and it was possible to substantially shorten the construction period.
(4) For field welding, welding procedure tests were conducted in advance using full-size models of column-to-column and girder-to-girder connections, and methods of preventing weld defects, repairing defects, and ensuring accurate erection were established.
(5) Chinese welding engineers and operators were trained in Japan and a system of execution of field welding suited to the procedure adopted was established.
(6) The sequence of connections adopted was such that the welding of girder-to-girder connections was conducted in advance of that of column-to-column connections to reduce residual stresses on the members.
(7) The accurate erection of steel frames was sufficient to comply with the AISC (American Institute of Steel Construction) standards as a result of providing excess length to the columns during shop fabrication based on the results of the welding experiments, making fine adjustments using liners inserted in the column-to-column connections on the basis of the results of field measurements, and maintaining loose erection bolt holes in the web of the girder-to-girder connection.

The field erection of the steel frames of this building posed many problems, such as field welding of extra-heavy steel members (column-to-column and girder-to-girder connections), the inexperience of welding operators, and joint work by Chinese and Japanese. The results of weld inspection by ultrasonic examination were good, however, and the accuracy of erection of steel frames was high. The steel structure was completed in May 1987. The authors believe that this was accomplished because the persons related to the work had a comprehensive knowledge of welding technology.

The authors wish to express their appreciation to staff members of the Osaka Branch Office, Yokogawa Bridge Works, Ltd. and of Kyusyu Komai Iron Works Co., Ltd. for their cooperation in the fabrication of specimens for the field welding procedure tests using full-size models.