SEISMIC MOMENT RESISTING GIRDER CONNECTIONS TO DIAGONALLY ALIGNED COLUMNS

By

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SYNOPSIS

To build an aseismic frame which can easily be fabricated, we propose the "diagonal wide flange column construction" as a new structural system. The following characteristics have been ascertained theoretically and experimentally: (a) easiness in shop fabrication and field construction; (b) excellent properties in beam-to-column connection; and (c) better moment distribution in frames.

NOTATION

The following symbols have been adopted in this paper:

- A — Parameter on moment of inertia, I ; and sectional area.
- C — Parameter on distribution of external shear force.
- D — I_{y2}/I_{x2}.
- α — Parameter on stiffened length of column compared to half of story height, H.
- κ — Factor on the concentration of shear stress; (maximum shear stress divided by average shear stress based on the total sectional area.)
- τ — Shear stress.

INTRODUCTION

In Japan, on account of large lateral forces from earthquake and typhoon, most structures require rigidity and resistance to bending moment in beam-to-column connections. Therefore, in steel structure, it is most important to design strong, rigid and ductile joints. The maximum moment at joints causes difficulty to make the concise and sound connections.

As shown in Figure 1, joint details used generally in Japan can be classified into the following several types; gusset plate, top and seat angle, tee stub, end plate and welding types.

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Joints with gusset plates or top and seat angles are most popular in frames consisting of columns and beams fabricated with angle shapes. The former has high rigidity and is considered as a rigid joint, but the latter lacks rigidity and is considered as a semi-rigid joint.

In frames with wide flange shapes, beams are connected to columns using high strength bolts with tee stubs or end plates, or are jointed to columns by welding. In this case, high continuity is obtained and rigid joints are formed. Recently joints with tee-stubs and high strength bolts have been increasing because of its ease in fabrication and construction.

Shop welding is widely used in fabricating members from plates and in connecting them. But welding in the field is rare because of the difficulty in work, of deformation and shrinkage by heat input due to welding. As high rise buildings are increasing recently, the size and thickness of shapes in columns and beams are increasing. It is the most usable method that welds column to column and beam to column in the field. Up to now, CO_2 arc and electro-slag welding have been in use, but more efficient and better methods should be developed.

Ordinary wide flange frames make bad bending moment distribution, because of the considerable difference of moment of inertia on strong and weak axes in columns. In order to improve this poor moment distribution and to simplify details of joints, we propose a new structural system named "diagonal wide flange column construction". In this structural system, as shown in Figure 2, columns are aligned diagonally in plan. Beams are simply connected to columns with high strength bolts by using connecting Y-pieces or by welding. As an example, typical details are shown in Figure 3.

This system has the following merits:

(a) By use of high strength bolts and Y-pieces, a simple detail is obtained.
(b) As for shear, a connecting panel zone has high strength and rigidity.
(c) Better moment distribution in frames is obtained by uniform rigidity in all directions, which saves material.
(d) Local deformation and stress concentration are minimized by the detail used.

In this paper, it relates to the results of (a) theoretical research on strength, rigidity, distribution of force and stiffening method in wide flange column and (b) experimental investigation on beame-to-column connection relating to the properties of joint as a whole, panel zone, connecting Y-piece and stiffeners, and on the increase of tension in bolts.

THEORETICAL RESEARCH ON BENDING OF WIDE FLANGE COLUMN

As rigidity of building in ridge and span directions differs and horizontal force by earthquake acts from any arbitrary direction, the building in general does not sway in the plane of external horizontal force. For convenience, we consider separately the forces divided in the ridge and span directions.

When the centers of rigidity and of mass coincide, building sways in the direction of the component of the external force in that direction. Then the effective moment of inertia and modulus of section are expressed by formula (1).

$$\left.\begin{aligned} I_n &= 0.5(I_x + I_y) \\ Z_n &= \sqrt{2}\,(I_x + I_y)/(b+h) \end{aligned}\right\} \quad \text{——— (1)}$$

External force Q is divided into x and y directions, Q_y , Q_x respectively and formula (2) follows.

$$\left.\begin{aligned} Q_x &= \frac{I_x}{I_x+I_y}\cdot\sqrt{2}\,Q = \frac{C}{1+C}\cdot\sqrt{2}\,Q \\ Q_y &= \frac{I_y}{I_x+I_y}\cdot\sqrt{2}\,Q = \frac{1}{1+C}\cdot\sqrt{2}\,Q \\ Q_n &= \frac{Q_x-Q_y}{\sqrt{2}} = \frac{I_x-I_y}{I_x+I_y} = -\frac{1-C}{1+C}\cdot Q \\ C &= I_x/I_y \end{aligned}\right\} \quad \text{——— (2)}$$

Referring to Figure 4, the force on the column is divided into x and y directions in proportion to the moment of inertia. Therefore, the direction of resultant of Q_x and Q_y, is different from that of column deflection. And as a reaction of the force Q_n, the axial force is produced in the two beams perpe ular to the external force. In other words, as the neutral axis of column in this structural system is perpendicular to external force, column acts as composite one.

When the joint is welded, a simple connection is obtained by using vertical stiffeners between two flanges. It creates an effective panel zone for shear force and the resisting moment of the column is considerably increased by extending these stiffeners. The degree of stiffening, that is thickness and length of plates, is considered in view of resolution of forces and economy in weight.

Referring to Figure 5, formula (3) gives the deflection curve of the column, on the assumption that the inflection point of bending moment is at the middle of the column and the column deflects in the direction of the external force.

$$EI\,\frac{d^2x}{dz^2} = Q_x(H-z) \quad \text{——— (3)}$$

Deflection in x, direction is obtained by integrating and by satisfying the end conditions. Deflection in the y direction is obtained in the same way. Then equalizing these deflections, formula (4) to (6) are obtained.

$$\left.\begin{aligned} Q_x &= \frac{C}{1+C}\cdot\sqrt{2}\,Q \\ Q_y &= \frac{1}{1+C}\cdot\sqrt{2}\,Q \\ Q_n &= -\frac{1-C}{1+C}\cdot Q \end{aligned}\right\} \quad \text{——— (4)}$$

$$C = \frac{I_{x1}\cdot I_{x2}}{I_{y1}\cdot I_{y2}}\cdot\frac{2I_{y1}-\alpha^2(3-\alpha)(I_{y1}-I_{y2})}{2I_{x1}-\alpha^2(3-\alpha)(I_{x1}-I_{x2})} = \frac{I_{x1}}{I_{y1}}\cdot\left\{\frac{2A_y-\alpha^2(3-\alpha)(A_y-1)}{2A_x-\alpha^2(3-\alpha)(A_x-1)}\right\} \quad \text{— (5)}$$

$$A_x = I_{x1}/I_{x2}\ ,\quad A_y = I_{y1}/I_{y2}$$

$$\text{when}\quad \alpha = 0\ ,\quad C = I_{x2}/I_{y2}$$

$$\alpha = 1\ ,\quad C = I_{x1}/I_{y1}$$

$$I_n = \frac{1+C}{C}\cdot I_{x2}\cdot\left\{\frac{A_x}{2A_x-\alpha^2(3-\alpha)(A_x-1)}\right\} \quad \text{——— (6)}$$

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That is, parameter C is expressed as the function of and t in formula (5).

Formula (7) is obtained on the assumption of yielding at the ends of column and stiffeners simultaneously.

$$C = -\frac{I_{x1}}{I_{y1}} \cdot \left\{ \frac{(1-\alpha) A_y \cdot Y_{y2} - Y_{y1}}{(1-\alpha) A_x \cdot Y_{x2} - Y_{x1}} \right\} \qquad (7)$$

Equalizing formula (5) and (7)

$$\frac{(1-\alpha) A_y \cdot Y_{y2} - Y_{y1}}{(1-\alpha) A_x \cdot Y_{x2} - Y_{x1}} = \frac{-2A_y + \alpha^2(3-\alpha)(A_y-1)}{2A_x - \alpha^2(3-\alpha)(A_x-1)} \qquad (8)$$

When the change of depth is neglected

$$\frac{(1-\alpha)A_y - 1}{(1-\alpha)A_x - 1} \cdot D = \frac{-2A_y + \alpha^2(3-\alpha)(A_y-1)}{2A_x - \alpha^2(3-\alpha)(A_x-1)} \qquad (9)$$

Wide flange column shapes, equal in depth and width, are on the market and they have the following general characteristics.

$$t_f \fallingdotseq 1.6\, t_w \quad \text{and} \quad h \fallingdotseq 20\, t_w \qquad (10)$$

With reference to Figure 5, the following sectional properties are shown.

$$\left.\begin{array}{ll} A_2 = 4.2\, t_w \cdot h, & A_1 = 4.2\, t_w \cdot h + 2\, t_s \cdot h \\ I_{x2} = 10.6\, K, & I_{x1} = (10.6 + 2t_k) \cdot K \\ I_{y2} = 3.2\, K, & I_{y1} = (3.2 + 6t_k) \cdot K \\ I_{x2}/I_{y2} = 3.3, & A_x = 1 + 0.189\, t_k \\ K = t_w \cdot h^3/12, & A_y = 1 + 1.87\, t_k \end{array}\right\} \qquad (11)$$

Results of calculations based on formula (4) to (11) are shown in Figure 7 to 10. They show that:

(a) Distribution of force Q to x and y direction is almost constant within the limit of $\alpha < 0.6$, regardless of stiffener plate thickness; t_k. Ratio of Q_y to Q_x increases rapidly over $\alpha = 0.7$ (Fig. 6)

(b) Effective moment of inertia increases with α and t_k. (Fig. 7)

(c) Ratio of resisting bending moment to effective moment of inertia continues increasing with α to yielding at the end of column and of stiffeners simultaneously. (Fig. 8)

(d) Thickness of stiffener plate, which causes yield at the end of column and of stiffener at the same time, increases with the length of stiffener. (Fig. 9)

(e) Strength of column for bending moment increases with α, but it is restricted by the thickness of stiffener. (Fig. 9)

(f) When $t_k \fallingdotseq 1.6$ and $\alpha \fallingdotseq 0.5$, maximum saving in material is obtained and it is about 40 percent. (Fig. 10)

EXPERIMENTAL INVESTIGATION ON BEAM-TO-COLUMN CONNECTIONS

The main purpose of this part of the study was to see whether, and under what conditions, it is possible to develop the strength, stiffness, and ductility in the beam-to-column connections.

DESCRIPTION OF TESTS.- Mechanical properties of material used are shown in Table 1. The average strength and proof load of high strength bolts are 15.1 tons and 13.6 tons, respectively.

The project consists of five series, as shown in Figure 11. We have investigated:

(a) The deformation and the unit strain in angle shapes in Y-piece and the increase in bolt tension. (Y-series)
(b) The effective width of horizontal stiffener for bending moment for vertical loads. (W-series)
(c) The stress distribution and the deflection in elastic and plastic ranges, and the ductility factor for bending of column. (M-series)
(d) The properties of connection for vertical loads (B-series) and for horizontal forces due to earthquakes. (X-series)

In the X-series, we neglected the effect of axial force in columns, and the characteristics of each specimen are as follows. X-11 specimen has a common detail with T-stubs and web-stiffener. There are X-13, X-16 and X-17 as diagonal column specimens. X-13 has connecting Y-pieces and vertical stiffener in the panel zone. The beams of X-16 are welded to the boxed column. X-17 has a welded joint similar to X-16, and it consists of wide flange column with extended vertical stiffeners.

5/8" bolts used were torqued to 26 kg·m to get bolt tension of 9.5 ton.

MEASUREMENTS.- The following measurements were made: (a) maximum and proof load, (b) total deflection or elongation, (c) partial deflection of column, beam, panel, etc., (d) unit strain of members, (e) bolt tension, (f) state of fracture, residual bolt tension and permanent deformation. Dial gages, strain meters and reflection type photoelastic method were used.

Specimens were loaded with the Amsler type testing machine of 200 ton capacity. Horizontal force on columns in X-series was added from the right, and then from the left repeatedly.

RESULTS.- Y-series, As shown in Figure 16, bolt tension began to increase from the applied load about 60 % of pretension. The thicker angle is, the larger increasing ratio of bolt tension and the separation load. It is possible to use comparatively thinner angle by folded plate effect. Fracture occured in plate which had about 30 % stress concentration.

W-series, Yielding and maximum load were 12 tons and 19 tons, respectively. On account of large deformation of column, the load reached a maximum, and the ductility factor of about 6.5 was indicated. The measured unit strain of stiffener coincided with the computed value as tee section assuming the effective width of column plate.

M-series, The deflection curve and the unit strain at maximum moment are shown in Figure 12. The computed and the measured values coincided in elastic and plastic range. Yielding and maximum load were 21.5 tons and 35 tons, respectively. The large ductility factor of about 33 was obtained.

B-series, The load-deflection curves are shown in Figure 13. High strength and rigidity in B-17 in comparison with those in B-13 are due to the use of horizontal stiffeners, extension of vertical stiffeners and stronger beams. It came to the maximum load by excessive deformation of column section in B-13 and B-16 and by local buckling of beam in B-17.

X-series, The computed and experimental results are shown in Table 2 and Figure 14 to 17.

(a) As a whole, X-11 and X-13 have about the same strength and rigidity. Owing to the extension of vertical stiffeners, X-17 was superior in strength.
(b) In X-11 and X-13, it broke by the tension of connecting piece and by the tension of bolts in the test of '67 and '68 year respectively. It came to the maximum load by lateral buckling in the beam of X-16 and local buckling in the beam and the column of X-17. This fact resulted in rather low ductility factor about 4 in X-11 and X-13 in comparison with the large factor about 11 in X-16 and about 7 in X-17. Therefore it is necessary to design bolts with more safety margin in practice.
(c) Strength and rigidity for shear in panel of X-13 and X-17 are due to the use of vertical stiffeners. In spite of stiffening web in panel zone of X-11, it produced rather low rigidity. Rigidity and yield strength in panel were decreased a little by pre-loading to plastic range.
(d) Computed results on unit shear, its concentration factor κ and deformation in joining panel are shown in Table 2 and Figure 17. Shear force is resisted only by column web in X-11, and by all plate elements of panel in X-13, X-16 and X-17. This results in large value of $\kappa \doteqdot 2.6$ in X-11, compared with $\kappa \doteqdot 1.5$ in other specimens.
(e) Bolt tension increased gradually and reached the maximum value on the tension side, and it decreased on the compression side. Practically, applied load should be restsicted to 60 % of pre-tension.
(f) As local deformation and stress concentration did not almost occur in any specimen, forces were transmitted smoothly.
(g) The unit shear stresses of horizontal stiffeners in X-16 and X-17 were comparatively low.

CONCLUSIONS

As an aseismic structure, we propose the diagonal column construction. Theoretical and experimental studies have been carried out on this system. The conclusions of this study, some of which have been included in previous chapters, are as follows:

(a) As the neutral axis of column in this structural system is perpendicular to external force, strength and rigidity of wide flange column have the average value of weak and strong axes. The force from any arbitrary directions on the column is divided into two main axes in proportion to each moment of inertia. Therefore better moment distribution in rigid frame is obtained. (Formula 1, 2 and Fig. 4)
(b) It is effective to extend vertical stiffeners in panel in view of strengthening of column. When $t_k \doteqdot 1.6 = t_f$ and $\alpha \doteqdot 0.5$, the maximum saving in material is obtained and it is about 40 %. (Formula 4~11, Fig. 5~10)
(c) The use of vertical stiffeners is very effective for large shear force in panel. Shear force is resisted only by column web in ordinary structures like X-11 and by all plate elements of panel in this system. This results in large value of $\kappa \doteqdot 2.6$ in ordinary structures compared with $\kappa \doteqdot 1.5$

in this system. (Table 2, Fig. 15, 17)

(d) Applied tension in bolt should practically be restricted to 60 % of pre-tension thinking about joint deformation. Applied force on tension bolt in Y-piece is developed into tensile strength of bolt. It is possible to use comparatively thinner angle in Y-piece by folded plate effect. (Fig. 16)

(e) On bending a column from any arbitrary directions, the computed result coincided with the observed in elastic and plastic ranges.

(f) As for vertical load, horizontal stiffener should be designed as a part of tee section considered on the effective width. (Fig. 12)

(g) *Effectiveness of vertical stiffener for shear force in panel was proved* by the experiment of X-series. Rigidity and yield strength in panel were decreased a little by pre-loading to plastic range. (**Fig.** 15)
All specimens had ample ductility factors, comparing with the factors of dynamic response of building frame in earthquake. (Fig. 14)

By the studies mentioned above, the characteristics of this system have been clarified.

ACKNOWLEDGEMENT

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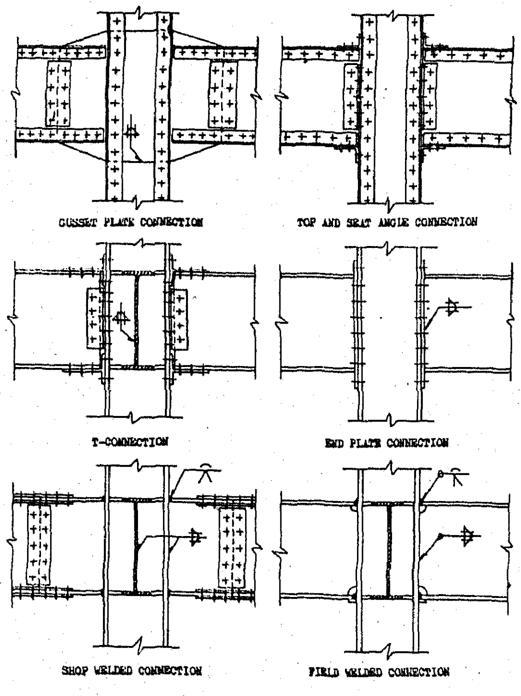


FIG. 1. TYPICAL BEAM-TO-COLUMN CONNECTION

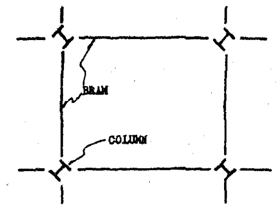


FIG. 2. TYPICAL ALIGNMENT OF COLUMNS

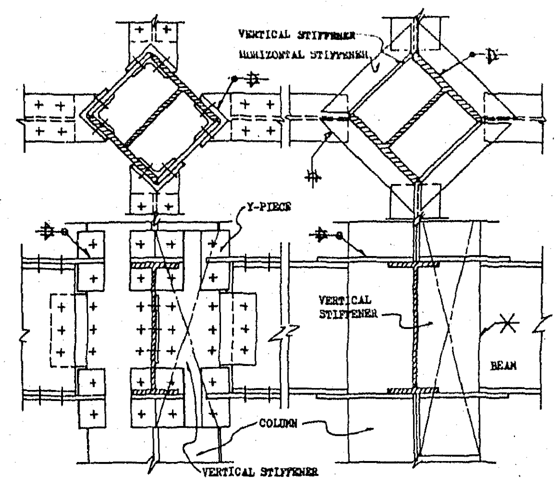


FIG. 3. TYPICAL DETAILS OF BEAM-TO-COLUMN CONNECTION IN DIAGONAL COLUMN CONSTRUCTION

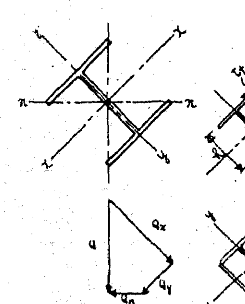


FIG. 4.
DISTRIBUTION OF SHEAR FORCE ON COLUMN

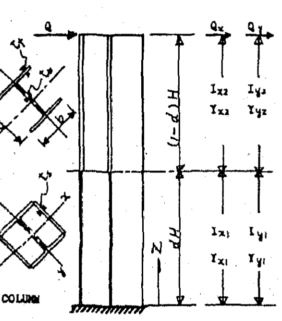


FIG. 5.
NOTATIONS ON PARTIALLY STIFFENED COLUMN

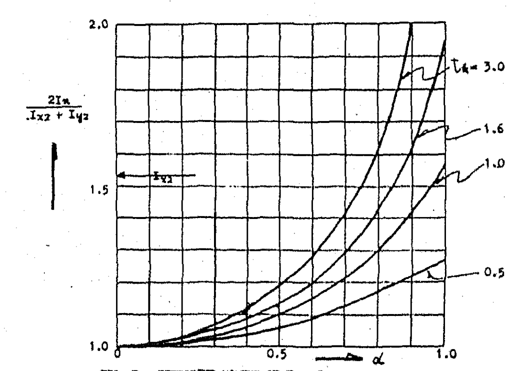


FIG. 7. EFFECTIVE MOMENT OF INERTIA

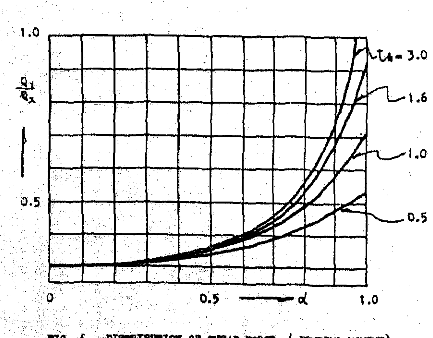


FIG. 6. DISTRIBUTION OF SHEAR FORCE, (=BENDING MOMENT)

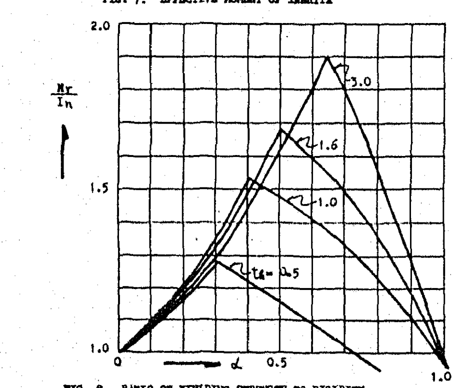


FIG. 8. RATIO OF YIELDING STRENGTH TO RIGIDITY

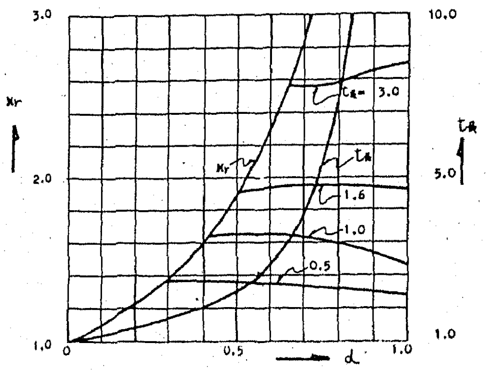


FIG. 9. RESISTING MOMENT AND THICKNESS OF STIFFENER THAT YIELDS AT THE SAME TIME IN ENDS OF STIFFENER AND COLUMN

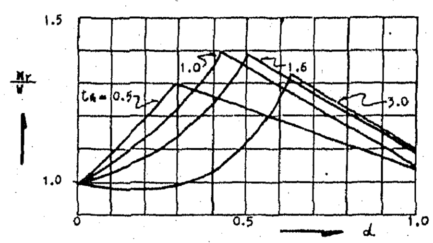
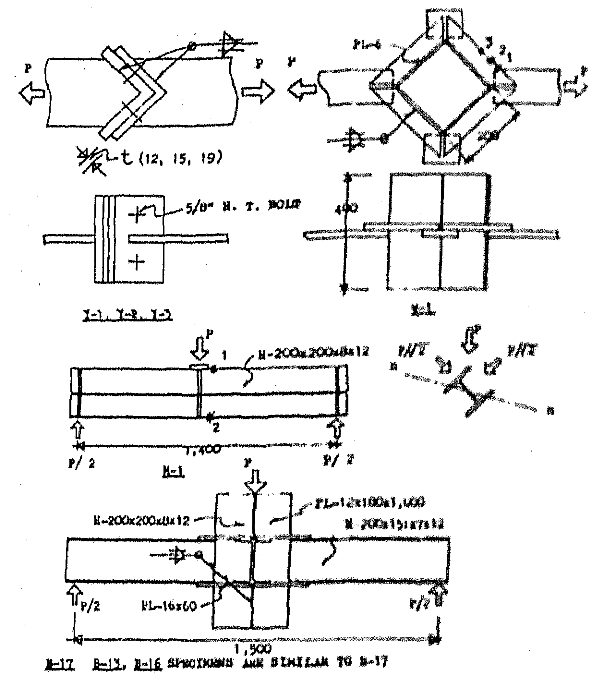


FIG. 10. RESISTING MOMENT VERSUS WEIGHT OF COLUMN



NOTE: UNIT STRAIN MEASUREMENT AT POINT OF STARRED MARK
SCALE IN MM

FIG. 11-A. TEST SPECIMEN (Y, W, M & B SERIES)

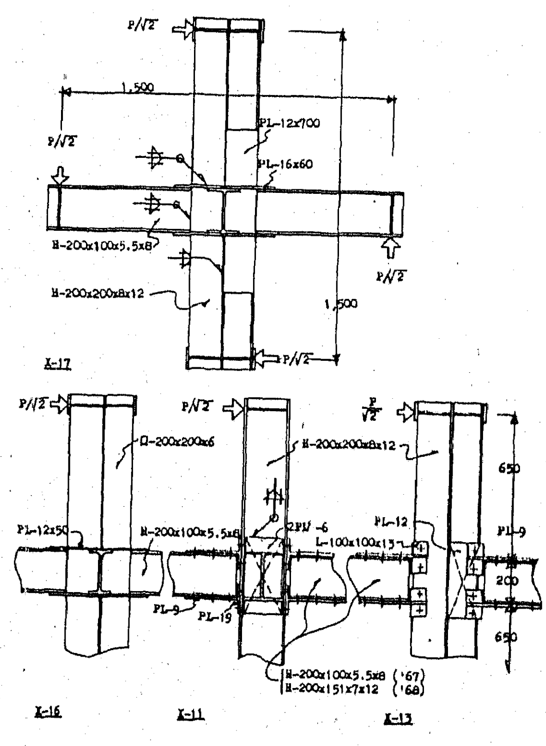


FIG. 11-B. TEST SPECIMEN (X-SERIES)

SCALE IN MM

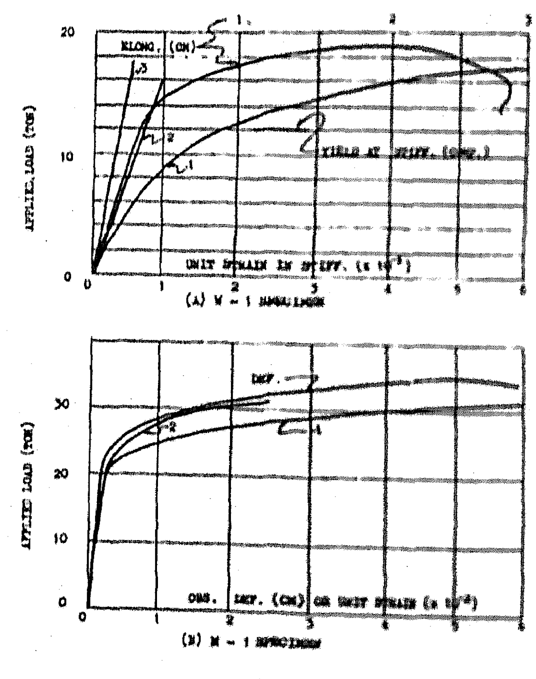


FIG. 12. OBSERVED UNIT STRAIN OR TOTAL DEFORMATION VERSUS APPLIED LOAD

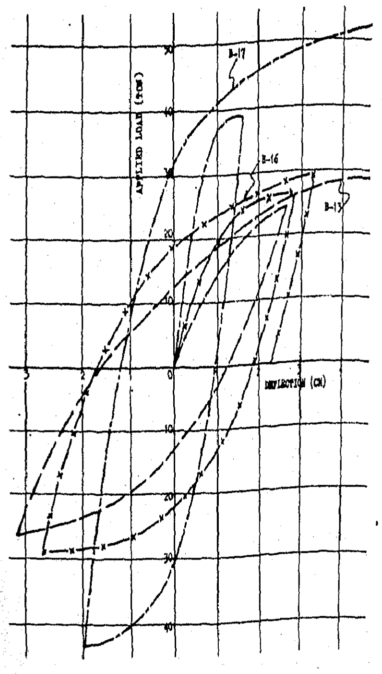


FIG. 13. LOAD-DEFLECTION CURVE

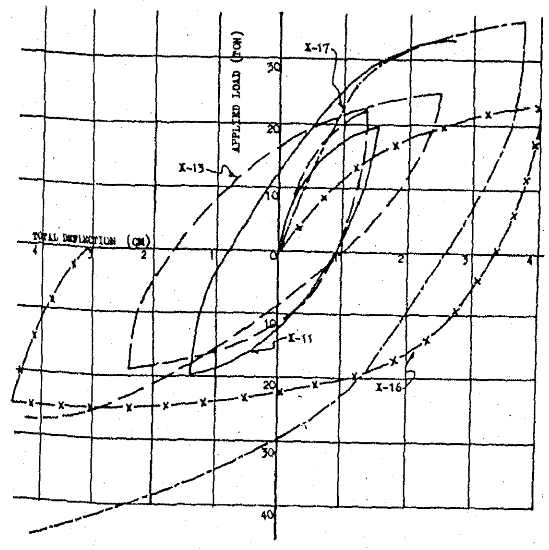


FIG. 14. LOAD-DEFLECTION CURVE

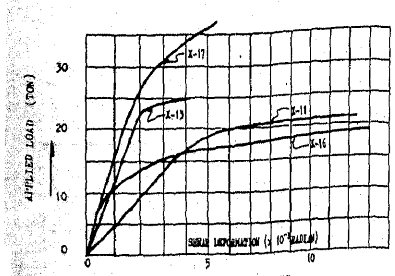


FIG. 15. SHEAR DEFORMATION IN PANEL ZONE

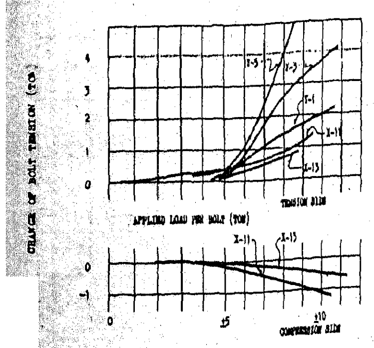


FIG. 16. BOLT FORCE VERSUS APPLIED LOAD

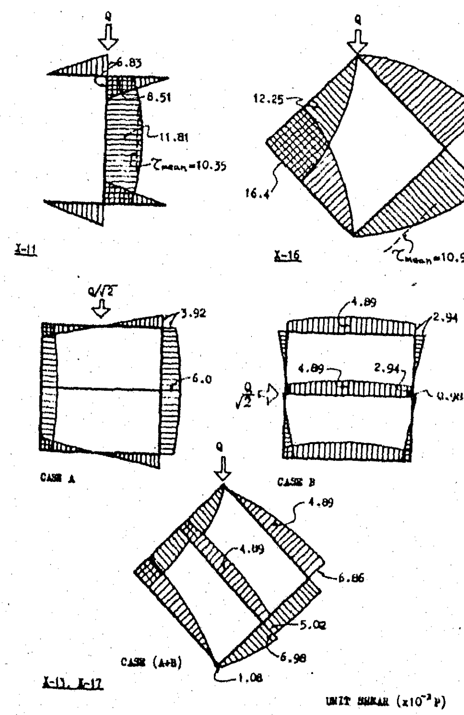


FIG. 17. COMPUTED UNIT SHEAR IN PANEL ZONE

TABLE 1. PROPERTIES OF MATERIALS USED

MATERIAL	σ_y	σ_t	ELONG.	E	E_{st}	E/E_{st}	$\varepsilon_{st}/\varepsilon_y$
PL-6	2.69	4.29	31.0	2,200	43.6	17.4	14.4
9	2.81	4.59	28.5	2,160	45.1	18.0	14.1
12	2.67	4.58	28.8	2,040	47.1	11.5	8.8
AVE.	2.72	4.49	29.5	2,130	45.3	15.6	11.2
WEB	3.53	4.93	34.3	2,070	55.9	38.3	11.5
FLANGE	3.02	4.63	41.1	2,060	49.6	41.5	18.6
AVE.	3.28	4.78	37.7	2,065	52.8	39.9	15.1

σ_y, σ_t, E, E_{st} : t/sq·cm, ELONG. : %

TABLE 2. COMPUTED AND OBSERVED VALUES OF X-SERIES

COMPUTED VALUE

SPECI-MEN	TEST	P_y/P_t (TON) BEAM	COL.	PANEL	BOLT TEN.	BOLT SHEAR	CONN. PIECE TEN.	CONN. PIECE BEND.	MIN. P_y/P_t	PANEL τ_{max}	PANEL κ	PANEL δ_P
X-11	'67	12.1 13.8	31.2 34.6	15.6 26.7	10.4 27.5	7.3 29.0	8.95 13.5	9.3 14.0	7.3 13.5	11.8	2.62 (1.14)	2.27 1.08
	'68	24.4 27.4	31.2 34.6	15.6 26.7	10.4 27.5			13.9 21.0	10.4 27.5			
X-13	'67	13.9 15.8	12.3	24.9 48.7	10.9 28.8	8.2 32.6	10.0 14.9	50.4	8.2 14.9	6.98	1.57	1.64 0.82
	'68	27.0 29.4	12.3	24.9 48.7	10.9 28.8			50.4	10.9 28.8			
X-16	'67	12.8 14.7	13.5 20.2	10.5 24.5					10.5 14.7	16.4	1.5	2.7 1.3
X-17	'68	25.4 28.2	17.9	24.9 48.7					17.9 28.2	6.98	1.57	1.64 0.82

OBSERVED VALUE

SPECI-MEN	TEST	P_y/P_t (TON) BEAM	COL.	PANEL	BOLT TEN.	CONN. TEN.	P_y/P_t	FAILURE
X-11	'67	16 >	> >	15.5 >	11 >	19.8	11 19.8	TENSION OF CONNECTING PIECE
	'68	24 27	32 >	17 >	11 32.6		11 32.6	TENSION OF BOLTS
X-13	'67	16 >	11.5 >	23 >	11 >	23.9	11 23.9	TENSION OF CONNECTING PIECE
	'68	24 >	12 >	25 >	11 27.6		11 27.6	TENSION OF BOLTS
X-16	'67	13 26.7	14 >	10.5 >			10.5 26.7	LATERAL BUCKLING OF BEAM
X-17	'68	26 >	22 >	25 >			22 48	LOCAL BUCK'G OF BEAM AND COL.

NOTE

τ_{max} : x 10^{-2} P t/sq·cm, δ_P : x 10^{-2} P·cm/x 10^{-4} P radian,

ASSUMED MATERIAL PROPERTIES σ_y = 3.0 t/sq·cm, σ_t = 4.5 t/sq·cm,

B_y = 0.6 x 9.5 = 5.7 t, B_t = 15.1 t, B_{sf} = 0.4 x 9.5 = 3.8 t, B_{st} = B_t