LIMIT ANALYSIS OF BEAM-COLUMN CONNECTION (XI)

(Design Method of H-Beam to Box-Column Connection)

By Dr. HISASHI TANAKA*, TSUNEO SHIGENOBU**
Members of A.I.J.

§1. Introduction

In the previous papers⁽¹⁾, the authors have investigated the yield strength of H-beam-to-box-column connections, theoretically and experimentally. In this paper, they propose more simplified formulae of the yield strength of connections for facilities of design practice.

§2. Design Method

The cruciform connection shown in Fig. 1 may be designed according to the following design procedure;

(a) Calculate the effective thickness t of column plates :

$$t = \frac{1}{2} (t_1 + t_2) \dots\dots\dots (2.1)$$

where t_1 and t_2 are the plate thickness of upper and lower columns, respectively.

(b) Calculate the effective moment M and axial force N ²⁾ shown in Fig. 2 :

$$\begin{aligned} M &= \frac{1}{2} (M_2 + M_4) - \frac{1}{4} (V_1 + V_3)H \\ &= \frac{1}{2} (M_1 + M_3) - \frac{1}{4} (V_2 + V_4)D \dots\dots\dots (2.2) \end{aligned}$$

$$N = \frac{1}{2} (N_1 + N_3) \dots\dots\dots (2.3)$$

where M_i , V_i ($i=1, 2, 3, 4$) and N_i ($i=1, 3$) are the applied moments, shearing forces and axial forces, as shown in Fig. 1.

(c) Calculate the required thickness t_p of connection-web using the formulae shown in Table 1, where σ_0 denotes the yield strength of material.

Fig. 1 Cruciform Connection

For the T-type of connection shown in Fig. 3, let M_1 and

Fig. 2 Idealized Connection

Table 1

Conditions		Required thickness of connection-web : t_p
$M \leq D^2 t \sigma_0$	$\frac{N}{2} + \frac{M}{D} \leq D t \sigma_0$	$t_p \geq \frac{\sqrt{3} M}{H D \sigma_0}$
	$\frac{N}{2} + \frac{M}{D} \geq D t \sigma_0$	$t_p \geq \sqrt{3 \left(\frac{M}{H D \sigma_0} \right)^2 + \left(\frac{N}{2 D \sigma_0} + \frac{M}{D^2 \sigma_0} - t \right)^2}$
$M \geq D^2 t \sigma_0$		$t_p \geq \sqrt{3 \left(\frac{D t}{H} \right)^2 (2 - \xi)^2 + \left(\frac{N}{2 \xi D \sigma_0} \right)^2}$ $\xi = \sqrt{3 - 2 M / D^2 t \sigma_0}$

* Prof. Inst. of Indu. Sci., Univ. of Tokyo

** Assist. Inst. of Indu. Sci., Univ. of Tokyo

Fig. 3 I-Type Connection

V_4 be zero in the above procedure.

The T type of connection shown in Fig. 4 may be treated as follows;

(a) Take the thickness of column plates and the effective depth of beam as

$$t=t_2, H=2H' \dots\dots\dots(2.4)$$

(b) Calculate the effective moment M and axial force N :

$$\begin{aligned} M &= M_2 + M_4 - V_3 H' \\ &= M_3 - \frac{1}{2} (V_2 + V_4) D \dots\dots\dots(2.5) \end{aligned}$$

$$N = N_3 \dots\dots\dots(2.6)$$

where M_i, V_i ($i=2, 3, 4$) and N_3 are the applied loads as shown in Fig. 4.

(c) Calculate the required thickness t_p using the formulae shown in Table 1.

For the L type of connection shown in Fig. 5, let M_4 and V_4 be zero in the procedure for T type of connection.

§3. Derivation of Design Formulae

For the cruciform connection shown in Fig. 2, analyses are classified into three cases shown in Table 2, and for each case the stress fields are divided into several regions shown in Fig. 6 and 7.

Table 2

Conditions	$M \leq D^2 t \sigma_0$	$M \geq D^2 t \sigma_0$
$\frac{N}{2} + \frac{M}{D} \leq D t \sigma_0$	Case A	Case C
$\frac{N}{2} + \frac{M}{D} \geq D t \sigma_0$	Case B

Fig. 6 Stress field in Case A and B

Fig. 7 Stress field in Case C

In each region the stress distributions are assumed as shown in Table 3, 4 and 5.

Applying the von Mises' yield condition

$$\sigma_y^2 + 3\tau^2 \leq \sigma_0^2 \quad (3.1)$$

to the stresses in the region (4), the required thickness of connection-web can be obtained as follows:

$$\text{Case A : } t_p \geq \frac{\sqrt{3} M}{HD \sigma_0} \quad (3.2 a)$$

Case B :

$$t_p \geq \sqrt{3 \left(\frac{M}{HD \sigma_0} \right)^2 + \left(\frac{N}{2D \sigma_0} + \frac{M}{D^2 \sigma_0} - t \right)^2} \quad (3.2 b)$$

Table 3 Case A

	σ_y	τ
(1)	$\frac{M}{D^2 t} - \frac{N}{2Dt}$	0
(2)	$\frac{M}{D^2 t_p} - \frac{2y}{H} - \frac{N}{2Dt_p}$	$\frac{2M}{HDt_p} \left(1 - \frac{x}{D}\right)$
(3)	0	0
(4)	0	$\frac{M}{HDt_p}$
(5)	$-\frac{M}{D^2 t} - \frac{N}{2Dt}$	0
(6)	$-\frac{M}{D^2 t_p} - \frac{2y}{H} - \frac{N}{2Dt_p}$	$\frac{2M}{HDt_p} \left(1 + \frac{x}{D}\right)$

Table 4 Case B

	σ_y	τ
(1)	$\frac{D^2 t}{2M} - \sigma_0$	0
(2)	$\frac{M}{D^2 t_p} \left(1 + \frac{2y}{H}\right) - \frac{t}{t_p} \sigma_0$	$\frac{2M}{HDt_p} \left(1 - \frac{x}{D}\right)$
(3)	$-\frac{N}{2Dt} - \frac{M}{D^2 t} + \sigma_0$	0
(4)	$-\frac{N}{2Dt_p} - \frac{M}{D^2 t_p} + \frac{t}{t_p} \sigma_0$	$\frac{M}{HDt_p}$
(5)	$-\sigma_0$	0
(6)	$\frac{M}{D^2 t_p} \left(1 - \frac{2y}{H}\right) - \frac{t}{t_p} \sigma_0$	$\frac{2M}{HDt_p} \left(1 + \frac{x}{D}\right)$

Table 5 Case C

	σ_y	τ
(1)	σ_0	0
(2)	$\frac{t \sigma_0}{t_p} - \frac{2y}{H}$	$\frac{2Dt \sigma_0}{Ht_p} \left(1 - \frac{x}{D}\right)$
(3)	$-\frac{N}{2\xi Dt}$	0
(4)	$-\frac{N}{2\xi Dt_p}$	$\frac{Dt \sigma_0}{Ht_p} (2 - \xi)$
(5)	$-\sigma_0$	0
(6)	$-\frac{t \sigma_0}{t_p} - \frac{2y}{H}$	$\frac{2Dt \sigma_0}{Ht_p} \left(1 + \frac{x}{D}\right)$

Fig. 8 (a) Comparison of Design Formulae and Limit Analysis

Fig. 8 (b) Comparison of Design Formulae and Limit Analysis

Fig. 8 (c) Comparison of Design Formulae and Limit Analysis

$$\text{Case C : } t_p \geq \sqrt{3 \left(\frac{Dt}{H} \right)^2 (2 - \xi)^2 + \left(\frac{N}{2 \xi D \sigma_0} \right)^2} \dots\dots\dots (3.2 c)$$

where

$$\xi = \sqrt{3 - 2 M / D^2 t \sigma_0} \dots\dots\dots (3.3)$$

§4. Accuracy of the design Formulae

Figs. 8 (a), (b) and (c) show the comparisons between the proposed design formulae and the theoretical results¹⁾. As seen in the figures, good agreement of both results assures the proposed design formulae are acceptable in design practice.

Reference :

- 1) H. TANAKA et al. : Limit Analysis of Beam-Column Connections,
 (V) Connections of Box-Column to Wide-Flange Beam Subjected to Anti-symmetrical Bending, Trans. A.I.J. Sept., 1969
 (VII) Connections of Wide-Flange Beam to Box-Column Subjected to Bending and Thrust, Trans. A.I.J. No. 165, Nov. and No. 166, Dec., 1969
 (IX) Experimental Verification, Trans. A.I.J. No. 169, March, 1970
 - 2) H. TANAKA et al. : Limit Analysis of Beam-Column Connections,
 (X) Design Method of H-Beam-to H-Column Connections, Trans. A.I.J. No. 173, July, 1970
-

柱はり接合部の極限解析 (XI) (梗概)

(H形はりと箱形柱の接合部の設計法)

正会員 田 中 尚*
" 重 信 恒 雄**

§1. はしき

本シリーズ (V), (VII), (X)¹⁾において, H形はり
と箱形柱の接合部強度の解析を行ないその結果を実験的に
確かめた。ここではこのような接合部の実際設計のため
の設計式を提案する。

§2. 設 計 法

Fig. 1 に示すような十字形接合部では, まず (a), 式
(2.1) によって柱標準板厚 t を求め, (b), 式 (2.2),
(2.3) によって等価モーメントおよび軸力を求める。
(c), Table 1 の式を用いて, 接合部板厚 t_p を求める。
Fig. 3 に示すI形の場合は, 式 (2.2) の中で $M_i = V_i$
 $= 0$ とおけば, 上の手順がそのまま使える。

Fig. 4 に示すT形接合部の場合は, (a), 柱標準板厚

およびはりせいを式 (2.4) のようにとり, (b), 等価モー
メントおよび軸力を式 (2.5), (2.6) で求め, (c), Table
1 の各式によって t_p を求める。Fig. 3 のL形の場合も
同様である。但し式 (2.5) で $M_i = V_i = 0$ とおく。

§3. 設計式の誘導

Fig. 2 に示す十字形接合部で考えると, 解析は Table
2 に示す3つの場合に分類できる。各場合に対し, 応力
場を Fig. 6, 7 のように, 各領域における応力分布を
Table 3, 4, 5 のように仮定する。これらの応力場で接
合部中央の領域 (4) に式 (3.1) の降伏条件を適用する
と, 接合部の所要板厚は式 (3.2 a, b, c) のように得ら
れる。

§4. 設計式の精度

Fig. 8 (a), (b), (c) に本略算解と精算解¹⁾の比較を
示す。図に見られる通り, 両者の結果は良く一致し, 略
算解を設計式として提案した。

* 京京大学教授, 生産技術研究所 工博

** 東京大学助手, 生産技術研究所

(昭和45年3月26日日本稿受理・討論期限昭和45年11月末日)

U.D.C. 624.014.5 : 539.4

FLANGE BUCKLING AND ULTIMATE LOAD OF BEAMS UNDER MOMENT GRADIENT

(Part III—Ultimate Load & Rotation Capacity)

by Dr. YASUNAGA FUKUCHI, Assist. Prof. of Tokyo Electric
Engineering Collage, Member of A.I.J.

Experiments and analyses on flange buckling of beams under moment gradient have been reported in my before reports (Part I & II). In this report, the ultimate loads and rotation capacities are analyzed and these analytical values are compared with the experimental values. And these values agree with each other.

Thus, the restriction values of the width-thickness ratio (b/t) of the flange plates of beams under moment gradient are proposed as followings according to the results of analyses on the rotation capacities.

for SS 41 $b/t \leq 12.3$

for SM 50 $b/t \leq 9.1$

for SM 50 Y $b/t \leq 8.7$

And it is shown that the ultimate loads of these beams can be over the full plastic moment loads.

U.D.C. 624.014.2.043 : 539.3

ELASTO-PLASTIC ANALYSIS OF THIN-WALLED OPEN SECTIONS

by Dr. TOSHIRO SUZUKI, Associate Professor, Nagoya Institute of
Technology; MAMORU KIMURA, Research Officer, Takenaka
Technical Research Laboratory and KAZUO ISHIDA, Research
Assistant, Aichi Institute of Technology. Members of A.I.J.

This paper describes the elasto-plastic analysis of arbitrary sections under arbitrary loads. Strain hardening and the effect of torsion are taken into account in the yield criterion. This numerical procedure is not based on the incremental-load procedure but on the method that directly solves the equilibrium equation under secant modulus theory. The assumptions for this analysis are as follows:

- (1) Normal strain is given by the value of the middle line of the cross section.
- (2) Total strain theory is applied, but strain-reversal is considered.
- (3) The behaviour of the strain-distribution beyond the elastic is similar that in the elastic range.
- (4) Axial and Shear Deformation of the members are considered to be neglected.

U.D.C. 624.078.072.2 : 624.043

LIMIT ANALYSIS OF BEAM-COLUMN CONNECTION (XI)

(Design Method of H-Beam to Box-Column Connection)

(See Page 33)

by Dr. HISAHSI TANAKA, Prof. Inst. of Indu. Sci. Univ. of Tokyo
and TSUNEO SHIGENOBA, Assist. Inst. of Indu. Sci., Univ. of
Tokyo, Members of A.I.J.

U.D.C. 333 : 331

SEASONAL FLUCTUATION OF CONSTRUCTION LABOR (II)

—The Change of Seasonal Patterns of Number of Construction Workers—

by RYOICHI SHIMADA, Assistant of Tokyo Metropolitan Univ.,
Member of A.I.J.

By examining the figure 1 and other figures, we can distinguish three stages in the period after the end of world war II with respect to the seasonal fluctuation of the construction labor in Japan.

- (I) The end of war (1944)—1950

In this stage, the ordinary seasonal pattern of construction workers does not appear, as seen in the figure

1. Economic conditions have not yet recovered enough just after the perfect destruction of the cities by