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ULTIMATE STRENGTH OF THIN-WALLED BOX STUB-COLUMNS

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This paper concerns the ultimate strength of thin-walled box stub-columns designed to fail by interactive plate buckling between the plate elements rather than by overall column buckling. An experimental study is described, conducted using nine stub-column specimens. The ultimate strength of stub-columns with initial imperfections and residual stresses is investigated using a finite element method (F. E. M.) based on elasto-plastic large deflection theory. Finally, two simplified design methods are proposed for evaluating the ultimate strength of stub-columns.

1. INTRODUCTION

Design codes such as JSHB[1], BS 5400 Part 3[2], DASt Ri012[3] and AASHTO[4] allow the use of non-compact steel plates which may fail prematurely by local buckling when used as components of columns with box cross-sections.

Several design methods[2),4)~8)] taking account of initial imperfections and residual stresses are available for calculating the ultimate strength of isolated unstiffened or stiffened plates under compression.

Theoretical and experimental studies have been conducted by Klöppel-Schmied-Schubert[9], Fukumoto-Usami-Aoki[10], Little[11], Frieze[12] and Mikami-Morisawa[13] on thin-walled columns with four plate elements. Even so, the ultimate strength of the plates, with the column proportioned so that overall buckling does not occur, can still not be predicted with sufficient confidence to formulate a rational design method. Therefore, presently, the ultimate load of thin-walled stub-columns is generally taken as a load which induces an average compressive stress in the cross-section equal to the ultimate stress of the weakest of all the plate elements taken as isolated simply supported plates. For this reason, stub-columns having one or two slender plate elements are likely to be uneconomically designed.

In this study, therefore, two alternative design methods are proposed for calculating the ultimate strength of thin-walled stub-columns. These methods are confirmed by not only the experimental results from this study and others[8),16),17)], but also the analytical results from the F. E. M. taking account of the elasto-plastic large deflection behaviour of each plate element and the interaction between the plate

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elements.

2. ELASTO-PLASTIC LARGE DEFLECTION ANALYSIS

The ultimate strength of thin-walled stub-columns with biaxially symmetrical cross-section, as shown in Fig. 1, is investigated in this study.

(1) Analytical model

In order to produce conservative results for design, it is assumed that both the initial deflected shape and the subsequent deflections due to loading are similar to the elastic buckling mode with a half wave length, a, in the longitudinal direction, as shown in Fig. 1. The length, a, needed to minimize this elastic buckling strength has been found to be different from the length needed to give the lowest ultimate strength. The latter length is examined in subsection 4 (1). Hereafter, the plate witdhs B and D are related to flange and web elements respectively.

Advantage is taken of symmetry to analyze only part of the column as shown in Fig. 2. The boundary conditions along the loaded and unloaded edges of the analytical model are as illustrated in Fig. 3. The flange and web plates at their intersections rotate so that the angle between them remains a right angle. Compression is introduced into the analytical model by applying compressive displacements to the nodal points on the loaded edge.

(2) Initial imperfection and residual stress

The maximum initial imperfections in the flange and web plates, v_0 and w_0, are taken respectively as $B/150$ and $D/150$ which correspond to the fabrication tolerance of JSHB[1].

The residual stress distribution adopted in the analysis is shown in Fig. 4. The compressive residual stress is taken as 0.3 times the yield stress, σ_y, based on the data measured in the experimental study (see 3. (3) a)).

(3) Non-dimensional plate slenderness R_f, R_w and R_{fw}

The plate slendernesses of flange and web plates, R_f and R_w, as defined by the following equations, are used as parameters to represent the sensitivity of each plate element to buckling:

$$R_f=\frac{B}{t_f}\cdot\sqrt{\frac{12(1-\mu^2)}{k\pi^2}}\cdot\sqrt{\frac{\sigma_{fy}}{E}} \quad (1)$$

$$R_w=\frac{D}{t_w}\cdot\sqrt{\frac{12(1-\mu^2)}{k\pi^2}}\cdot\sqrt{\frac{\sigma_{wy}}{E}} \quad (2)$$

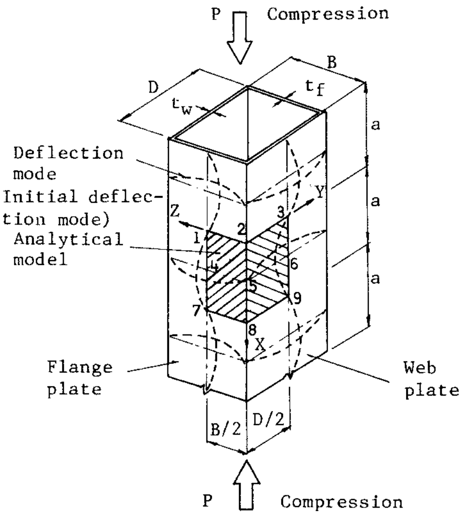

Fig. 1 Thin-walled stub-column.

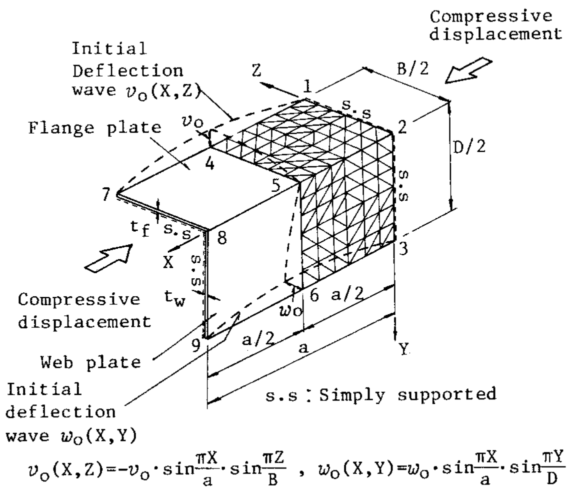

Fig. 2 Analytical model for stub-column.

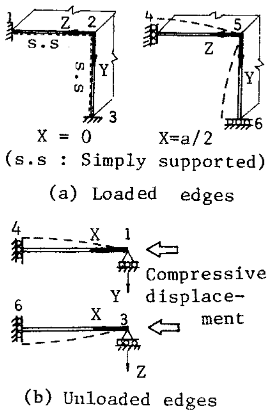

Fig. 3 Boundary conditions in analytical model.

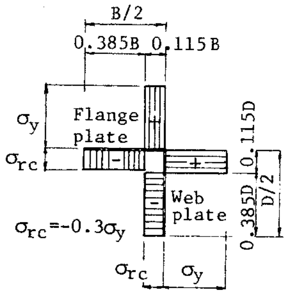

Fig. 4 Assumed residual stress distribution.

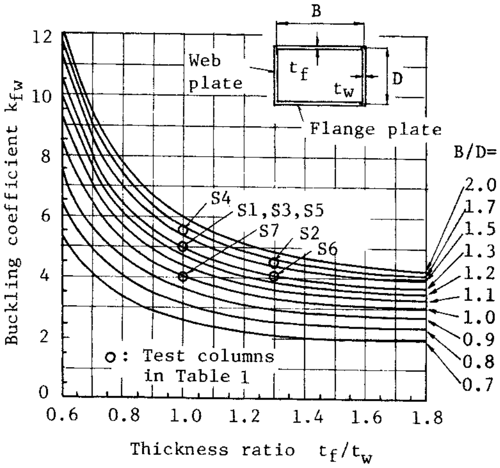

Fig. 5 Buckling coefficient k_{fw} of stub-columns.

where t_f, t_w : thicknesses of flange and web plates

σ_{fy}, σ_{wy} : yield stresses of flange and web plates

μ : Poisson's ratio (=0.3)

E : Young's modulus ($=2.06\times10^5$ MPa)

k : buckling coefficient of isolated plate (=4.0).

A non-dimensional plate slenderness, R_{fw}, is also defined as a single parameter to indicate the buckling sensitivity of the thin-walled stub-column by taking the square route of the reduced yield stress, σ_Y^*, divided by the elastic buckling stress, σ_{cr}, considering the interaction between the flange and web plates (see Appendix).

$$R_{fw}=\sqrt{\frac{\sigma_Y^*}{\sigma_{cr}}}=\frac{B}{t_f}\cdot\sqrt{\frac{12(1-\mu^2)}{k_{fw}\cdot\pi^2}}\cdot\sqrt{\frac{\sigma_Y^*}{E}} \quad (3)$$

$$\sigma_Y^*=\frac{A_f\cdot\sigma_{fy}+A_w\cdot\sigma_{wy}}{A} \quad (4)$$

where A_f, A_w : cross-sectional area of flange and web plates ($=B\cdot t_f$, $D\cdot t_w$)

A : half cross-sectional area of stub-column ($=A_f+A_w$)

k_{fw} : buckling coefficient of stub-column (see Eq. (A.2)).

k_{fw} is plotted as a parameter of the width ratio B/D versus the thickness ratio t_f/t_w in Fig. 5.

(4) Verification of the F. E. M. analysis

The computer program was checked for symmetry in the behaviour of flange and web plates for a symmetrical section column ($B=D$ and $t_f=t_w$) and the ultimate strength also confirmed compared with that calculated for the corresponding isolated plates in Ref. 5).

The mesh spacings used for the F. E. M. analyses were $a/12$ in the longitudinal direction, and $B/12$ or $D/12$ in the transverse direction.

The FACOM-M 180 computer of Osaka City University was used for the numerical analysis. The CPU-time taken for the calculation of a single case was 150 to 200 sec.

3. EXPERIMENTS

(1) Specimens

The dimensions and non-dimensional plate slendernesses of the specimens are shown in Fig. 6 and Table 1. The specimens Nos. S 1 to S 7 were used for buckling tests and the others used for measuring residual stress. All specimens were made of mild steel of grade SS 41, the nominal yield stress of which is 235

Table 1 Dimensions, cross-sectional properties and plate slendernesses of test columns.

Items / Test columns	Dimensions and cross-sectional properties								Plate slenderness		
	B(B') (mm)	D(D') (mm)	tf (mm)	t_w (mm)	2·A (mm^2)	L (mm)	r_y (mm)	L/r_y	R_f	R_w	R_{fw}
S1	145(160)	111(105)	5.85	5.85	3,092	500	47.1	10.6	0.549	0.360	0.446
S2	154(170)	95(89)	5.85	4.49	2,382	420	42.4	9.9	0.583	0.391	0.490
S3	185(199)	141(135)	5.85	5.85	3,917	500	59.7	8.4	0.686	0.466	0.569
S4	162(178)	99(94)	4.49	4.49	2,442	420	42.9	9.8	0.783	0.416	0.593
S5	245(260)	185(179)	5.85	5.85	5,135	580	78.4	7.4	0.893	0.617	0.751
S6	244(260)	185(179)	5.85	4.49	6,649	580	80.7	7.2	0.894	0.790	0.819
S7	447(476)	447(438)	9.06	9.06	16,551	1,440	183.5	7.8	1.007	0.991	1.007
RS1	154(170)	94(89)	5.85	4.49	2,382	500	42.4	—	0.583	0.391	0.490
RS2	162(178)	99(94)	4.49	4.49	2,442	500	42.4	—	0.783	0.416	0.593

Notes: 2·A:Cross-sectional area, r_y:Radius of gyration, R_f and R_w:Eqs.(1) and (2)
R_{fw}:Eq.(3)

MPa. Plates with nominal thicknesses of 4.5, 6 and 9 mm are used to vary the non-dimensional slenderness R_{fw} within the range from 0.45 to 1.0.

The ends of the specimens used for buckling tests were filled with cement mortar so that uniform compression was applied to the specimens.

Tensile tests carried out on the steel plates produced the average Young's modulus, average Poisson's ratio and static yield stresses as listed in Table 2.

(2) Buckling tests

Buckling tests were performed under load control in an Amsler type machine with a 200 tonf (1.96 MN) capacity and a compression testing machine[15] of horizontal type developed in Osaka City University.

Table 2 Test results of mechanical properties. (SS41)

Young's modulus E (MPa)	Poisson's ratio μ	Yield stress σ_y (MPa)		
		t=4.5mm	t=6mm	t=9mm
2.1 x 10^5	0.3	292	302	313

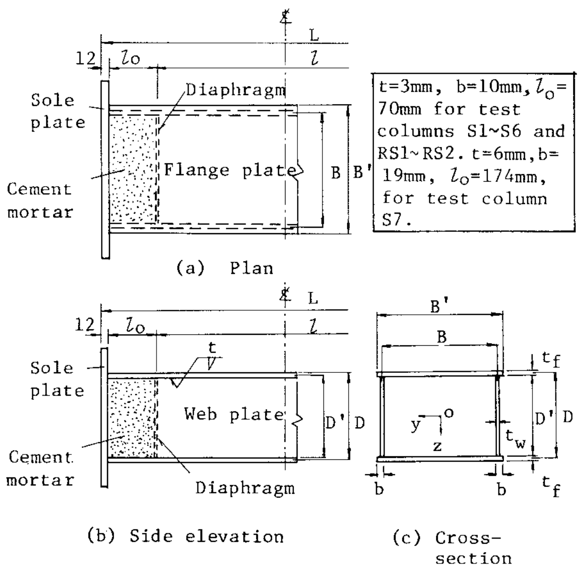

Fig. 6 Details of test columns.

(3) Test results

a) Initial imperfection and residual stress

Initial imperfections in the flange and web plates of each specimen used for the buckling tests were measured after the specimen positioned in the testing machine. The maximum value of initial imperfection in each plate element was within the range from 0.16 to 0.56 times the width divided by 150.

Residual stress distributions measured for

Table 3 Test results of ultimate strength P_u and P_u/P_Y.

Items / Test columns	Plate slenderness R_{fw}	Squash load $P_Y=2A\cdot\sigma_Y^*$ (MN)	Test results	
			P_u (MN)	P_u/P_Y
S1	0.446	0.993	1.109	1.092
S2	0.490	0.832	0.886	1.065
S3	0.569	1.182	1.259	1.066
S4	0.593	0.713	0.742	1.040
S5	0.751	1.550	1.428	0.921
S6	0.819	1.388	1.212	0.874
S7	1.007	5.714	3.938	0.761

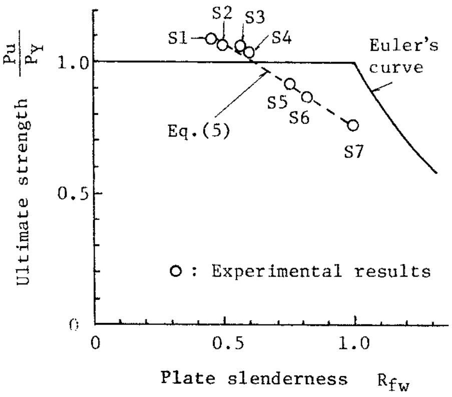

Fig. 7 Relationship between ultimate strength and plate slenderness of test columns.

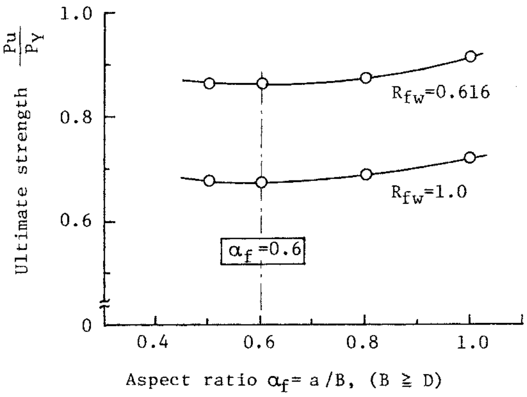

Fig. 8 Variation of ultimate strength with aspect ratio.

specimens Nos. RS 1 and RS 2 illustrated uniform values in the compression region as expected. The compressive residual stress was $0.35 \cdot \sigma_y$ at maximum with an average of $0.27 \cdot \sigma_y$.

b) Ultimate strength

The results of the buckling tests are shown in Table 3 and Fig. 7, where the ultimate load, P_u, is non-dimensionalized by the squash load $P_Y(=2 \cdot A \cdot \sigma_Y^*)$. The values of P_u/P_Y in specimens Nos. S 1 to S 4 are greater than 1.0. This may be due to the effect of strain hardening and the fact that the yield stress is generally higher in compression than in tension.

P_u/P_Y can be represented as a function of R_{fw} by a least squares fit :

$$P_u/P_Y = 1.384 - 0.614 \cdot R_{fw} \qquad (5)$$

4. PARAMETRIC STUDY BY F. E. M.

(1) Influence of length, a, on ultimate strength

In order to examine the variation of ultimate strength with the length, a, stub-column models with a width ratio $D/B=0.75$ were used, and the length, a, needed to minimize the elastic buckling strength of the models found to be approximately $0.84 \cdot B$.

The calculated ultimate strengths are plotted in Fig. 8 against the aspect ratio of the flange plate, $\alpha_f(=a/B)$. For both $R_{fw}=0.616$ and 1.0, the ultimate strengths are minimum at an aspect ratio of about 0.6. This value is smaller than 0.84 obtained from the elastic buckling analyses.

Stub-column models with $\alpha_f=0.6$ are dealt with later in the parametric study.

(2) Interactive behaviour between the plate elements and their ultimate strength

Two models with $R_f=1.3$ and $R_w=0.3$ as well as $R_f=1.3$ and $R_w=0.6$ were analyzed to examine the interactive behaviour between flange and web plates. The results are shown in Table 4 and Figs. 9 and 10, where

P : load applying to model $(P=P_f+P_w)$

P_f, P_w : loads carried respectively by flange and web plates

P_{fu}, P_{wu} : values of P_f and P_w at ultimate state of model

P_{fu}^u, P_{wu}^u : ultimate loads of flange and web plates taken as simply supported plates.

According to Ref. 5), P_{fu}^u and P_{wu}^u can be calculated by substituting R_f and R_w into the following equation :

$$\left.\begin{aligned} P_{fu}/P_{fY} \text{ or } P_{wu}/P_{wY} &= 1.0, \ (R \leqq 0.3) \\ &= 0.542 \cdot R^3 - 1.249 \cdot R^2 + 0.412 \cdot R + 0.968, \ (0.3 < R \leqq 1.3) \end{aligned}\right\} \qquad (6)_{a,b}$$

where

Table 4 Ultimate strength of stub-columns.

Items	F.E.M.			Ref. 5)		Results by Eq.(9)			Ratios		
	Flange plate	Web plate	Stub-column	Flange plate	Web plate	Flange plate	Web plate	Stub-column	Flange plate	Web plate	Stub-column
R_{fw}	(1) $\frac{P_{fu}}{P_{fY}}$	(2) $\frac{P_{wu}}{P_{wY}}$	(3) $\frac{P_u}{P_Y}$	(4) $\frac{P^u_{fu}}{P_{fY}}$	(5) $\frac{P^u_{wu}}{P_{wY}}$	$\frac{P_{fY}}{P_Y}$	$\frac{P_{wY}}{P_Y}$	(6) $\frac{P_u}{P_Y}$	$\frac{(1)-(4)}{(1)}$	$\frac{(2)-(5)}{(2)}$	$\frac{(3)-(6)}{(3)}$
0.3	0.671	0.989	0.896	0.580	0.994	0.291	0.709	0.874	0.136	-0.005	0.025
0.6	0.628	0.855	0.753	0.580	0.883	0.451	0.549	0.746	0.076	-0.033	0.009

Notes: t_w=24.63 mm, R_{fw}=0.388 for Rw=0.3, t_w=12.32 mm, R_{fw}=0.820 for R_{fw}=0.6, B=480 mm, D=360 mm, t_f=7.58 mm, R_f=1.3, σ_Y=314 MPa.

$$P_{fY}=A_f\cdot\sigma_{fy} \text{ and } P_{wY}=A_w\cdot\sigma_{wy} \quad \cdots\cdots (7)_{a,b}$$

Eq. (6) was proposed on the basis of the numerical results generated by introducing an initial imperfection of 1/150 times the plate width and a residual stress of $0.4\cdot\sigma_y$ into simply supported plates restrained so that the unloaded edge displacement remained straight[5].

The relationships between the applied load non-dimensionalized by the squash loads and a mean longitudinal strain are shown in Figs. 9 and 10, where u is the compressive dislacement given as input-data at the loaded edge. It can be seen from Fig. 9 (R_f=1.3, R_w=0.3) that the slender flange plate has already lost in-plane stiffness, while the stocky web plate maintains stiffness when the stub-column model reaches the ultimate state (point P). In the case of Fig. 10 (R_f=1.3, R_w=0.6), the stub-column model loses stiffness due to the simultaneous failure of flange and web plates. However, the phenomenon encountered in Fig. 9 does not occur in practice for structural members, because the R_f and R_w values are generally less than 1.0 and the plate slenderness ratio R_f/R_w is not normally far from 1.0.

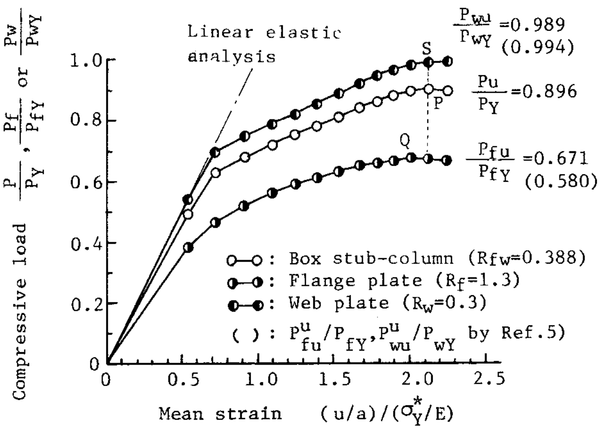

Fig. 9 Compressive load-mean strain curve (R_w=0.3).

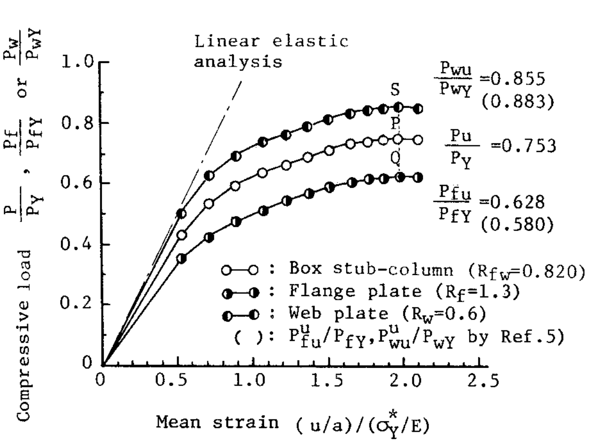

Fig. 10 Compressive load-mean strain curve (R_w=0.6).

It should be noted from Table 4 that the ultimate strengths of slender flange plates (P_{fu}/P_{fY}) in the models with R_w=0.3 and 0.6 are greater than the ultimate strengths of the corresponding isolated plates (P^u_{fu}/P_{fY}) by 14 % and 8 %, respectively, due to the stiffening action of the stocky web plates, while the P_{wu}/P_{wY} of the web plates are less than P^u_{wu}/P_{wY} by 1~3 % because of the destabilizing effect of the slender flange plates.

According to the definition of P_{fu} and P_{wu}, the ultimate strength of a stub-column, P_u/P_Y, is given by:

$$P_u/P_Y=(P_{fY}/P_Y)\cdot(P_{fu}/P_{fY}) + (P_{wY}/P_Y)\cdot(P_{wu}/P_{wY}) \quad \cdots\cdots (8)$$

However, the ultimate strength of a stub-column may be, on average, expressed by the following cumulative equation in terms of the ultimate strengths of the isolated plate elements, if it is considered that the slender plate element in a stub-column is strengthened by the stocky plate and the latter is weakened by the former :

$$P_u/P_Y \cong (P_{fY}/P_Y)\cdot(P^u_{fu}/P_{fY})+(P_{wY}/P_Y)\cdot(P^u_{wu}/P_{wY}) \quad (9)$$

It can be seen that Eq. (9) produces good agreement with the analysis as listed in Table 4.

(3) Relationship between single plate slenderness R_{fw} and ultimate strength

In order to examine the variation of the ultimate strength P_u/P_Y with the plate slenderness R_{fw} and the plate slenderness ratio R_f/R_w, numerical results were generated for 16 stub-column models listed in Table 5. These results are shown in Fig. 11, compared with the results from Eq. (9). The results from the F. E. M. show that the variation of P_u/P_Y with R_f/R_w is less than 2 % if R_{fw} remains constant. The results verifies the appropriateness of the use of R_{fw} as a single parameter representing the buckling sensitivity of thin-walled stub-columns.

As shown in Fig. 11, Eq. (9) predicts the ultimate strengths within 2 % error compared with the results from F. E. M.

(4) Ultimate strength curve for thin-walled stub-columns

Some of the F. E. M. results are replotted against plate slenderness R_{fw} in Fig. 12, together with the following ultimate strength curve with R replaced by R_{fw} in Eq. (6) :

$$\left.\begin{aligned} P_u/P_Y &= 1.0,\ (R_{fw}\leqq 0.3) \\ &= 0.542\cdot R_{fw}^3-1.249\cdot R_{fw}^2+0.412\cdot R_{fw}+0.968,\ (0.3<R_{fw}\leqq 1.3) \end{aligned}\right\} \quad (10)_{a,b}$$

The good agreement of the results from Eq. (10) with the F. E. M. results indicates the appropriateness of adopting Eq. (10) as a design method for calculating the ultimate strength of thin-walled stub-columns.

(5) Alternative design methods for calculating ultimate strength

From the parametric study, the following alternative design methods can be proposed for calculating the

Table 5 Dimensions of analytical models (Unit : mm).

Items / Rf/Rw	Rfw=0.411		Rfw=0.616		Rfw=0.822		Rfw= 1.0	
	tf	tw	tf	tw	tf	tw	tf	tw
1	18.0*	18.0*	12.0*	12.0*	9.0*	9.0*	9.8	7.3
$\sqrt{2}$	20.7	22.0	13.8	14.7	10.4	11.0	8.5	9.1
$\sqrt{3}$	18.2	23.6	12.1	15.8	9.1	11.8	7.5	9.7
2	16.3	24.4	10.9	16.3	8.1	12.2	6.7	10.0

Notes: B=480mm, D=360mm, B=D=360mm for super script *, σ_y=314MPa

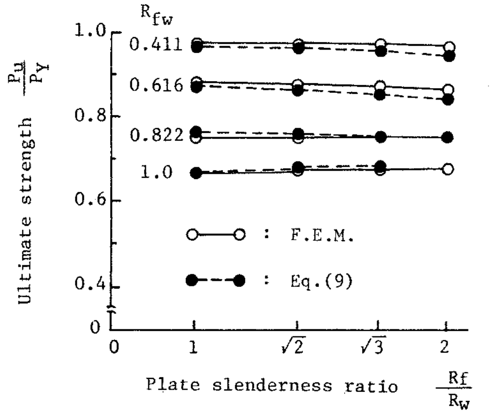

Fig. 11 Variations of ultimate strength $\frac{P_u}{P_Y}$ with R_f/R_w.

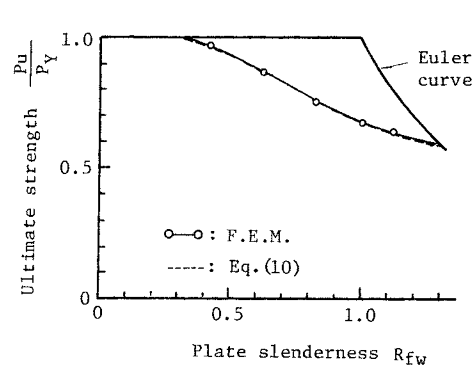

Fig. 12 Ultimate strength curve of thin-walled stub-column.

ultimate strength of thin-walled stub-columns.

a) Method 1 using Eq. (9)

The ultimate strength can be predicted by calculating the ultimate strengths of the isolated flange and web plates using Eq. (6) and then substituting them into Eq. (9). This method should be used in the ranges of $R_f \leqq 1.3$, $R_w \leqq 1.3$ and $0.5 \leqq R_f/R_w \leqq 2.0$.

Two parameters R_f and R_w are needed to represent the ultimate strengths of thin-walled stub-columns.

b) Method 2 using single plate slenderness R_{fw}

This method predicts the ultimate strength using Eq. (10), where the plate slenderness R_{fw} is calculated by substitution of the buckling coefficient k_{fw} obtained from Fig. 5 or Eq. (A. 2). The method should be used in the ranges of $R_{fw} \leqq 1.3$ and $0.5 \leqq R_f/R_w \leqq 2.0$.

This method is slightly more complicated than method 1 but has the advantage that the ultimate strength can be expressed in terms of a single parameter R_{fw}.

5. VERIFICATION OF DESIGN METHODS BY EXPERIMENTAL RESULTS

(1) Method 1

The ultimate strengths predicted by method 1 are compared in Fig. 13 with the experimental results from this experimental study (7 specimens), Fukumoto-Usami-Aoki (7)[10], Fukumoto-Usami (6)[16] and Okumura-Nishino-Hasegawa (26)[17]. The average error between the predicted and experimental results is 14.9 %. Fig. 13 shows that method 1 predicts results which are both accurate and conservative, and is useful for practical design.

(2) Method 2

The ultimate strength curve produced using Eq. (10) is shown in Fig. 14, compared with the experimental results mentioned above. The experimental strength line obtained from the authors' test results (Eq. (5)) is also plotted in this figure. This experimental line is close to an average line through all the test results. The ultimate strength curve gives a lower bound to the test results. This seems to be caused by the simultaneous combination of the large initial imperfection ($B/150$, $D/150$) and large residual stress ($0.4 \cdot \sigma_y$) when producing the ultimate strength curve.

Method 2 is also useful as an alternative to method 1 because of its accuracy and conservatism.

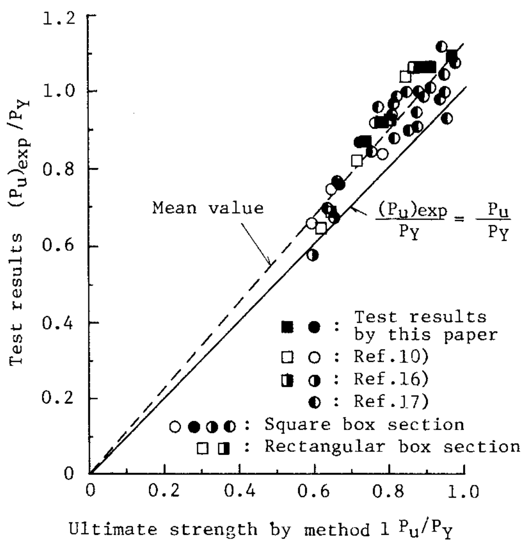

Fig. 13 Comparison of test results $\frac{(P_u)_{exp}}{P_Y}$ with ultimate strength $\frac{P_u}{P_Y}$ by Method 1.

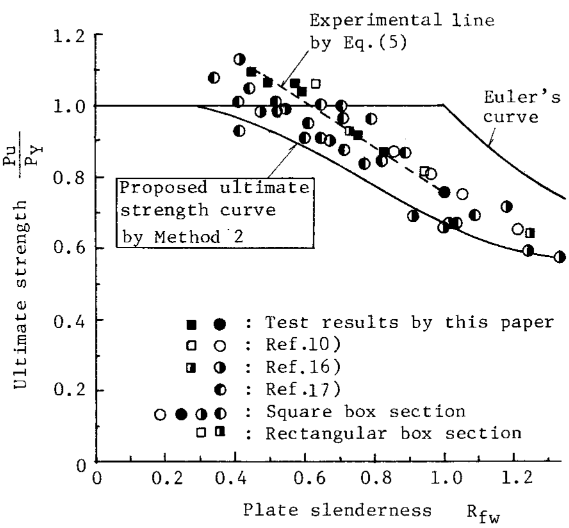

Fig. 14 Comparison of test results with ultimate strength curve for Method 2.

6. COMPARISON OF PROPOSED METHODS WITH OTHER DESIGN METHODS

In order to compare the two methods proposed here with the design methods of JSHB[1], BS 5400 Part 3[2], DASt Ri 012[3], AASHTO[4] and Ref. 10), the ultimate strengths of four stub-columns with R_{fw}=0.616 in Table 5 were calculated and are indicated in Table 6. The results are also shown in Fig. 15, where

γ= (Ultimate strength by F. E. M.) / (Ultimate strength from design methods) ······················ (11)

This figure shows that the values of γ in the design methods except methods 1 and 2 increase with an increase in the plate slenderness ratio R_f/R_w. The variation of γ in Fukumoto's method is rather smaller than in the other four methods, because the former also takes account of the difference between the plate slandernesses of the plate elements by using the same equation as method 1 (Eq. (9)). Fukumoto's method, however, gives unsafe results compared with the F. E. M. results. Their method uses an empirical formula for predicting the strengths of isolated plate elements based on their test results from 8 specimens having smaller initial imperfections than those used in this study.

7. CONCLUSION

(1) In a thin-walled stub-column comprising plate elements with different plate slendernesses, the slender plate elements are strengthened by the stocky plate elements. Therefore, design methods in which the ultimate strength is defined on the basis of the ultimate strength of the weakest plate element regarded as an isolated plate, predict increasingly conservative results as the difference between the slendernesses of the plate elements increases.

(2) The ultimate strength of thin-walled stub-columns can be represented as a function of a single plate slenderness, R_{fw}.

(3) Two alternative design methods have been proposed for evaluating the ultimate strength of thin-walled stub-column. One of them uses an ultimate strength curve in terms of R_{fw}. In the other, the ultimate load is defined as the sum of the ultimate loads of all the plate elements taken as isolated plates.

(4) The two methods are useful for the design of thin-walled stub-columns, because they predict accu-

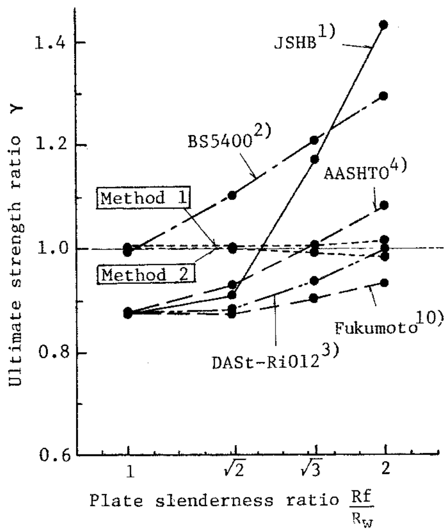

Fig. 15 Variation of ultimate strength ratio with R_f/R_w.

Table 6 Comparison of ultimate strength calculated by various design methods.

Methods \ Items R_f/R_w	Ultimate strength P_u/P_Y				Ultimate strength ratio γ by Eq.(11)			
	1	√2	√3	2	1	√2	√3	2
JSHB1)	1.000	0.961	0.740	0.600	0.880	0.912	1.172	1.432
BS5400 2)	0.886	0.793	0.718	0.664	0.993	1.104	1.208	1.294
DASt-Ri012 3)	1.0	0.991	0.923	0.862	0.880	0.883	0.939	0.997
AASHTO4)	1.0	0.944	0.861	0.793	0.880	0.928	1.007	1.083
Fukumoto 10)	1.0	1.0	0.960	0.920	0.880	0.876	0.903	0.934
F.E.M.	0.880	0.876	0.867	0.859	1.0	1.0	1.0	1.0
Method 1	0.875	0.870	0.855	0.845	1.006	1.007	1.014	1.017
Method 2	0.875	0.875	0.875	0.875	1.006	1.001	0.991	0.982

rate and conservative ultimate strengths.

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APPENDIX—ELASTIC BUCKLING ANALYSIS OF THIN-WALLED STUB-COLUMNS

It is assumed that each plate element in a box column shown in Fig. 1 has an elastic buckling mode with a half wave in the transverse direction and with infinitely continuous waves (half wave length a) in the longitudinal direction, and that the angle between the two neighbouring buckling modes at a corner of the stub-column remain a right angle.

The elastic buckling stress of the stub-column, σ_{cr}, is given by using the Ritz method as follows :

$$\sigma_{cr}=k_{fw}\cdot\frac{E\pi^2}{12(1-\mu^2)}\cdot\left(\frac{t_f}{B}\right)^2 \quad \text{(A. 1)}$$

where k_{fw} is a buckling coefficient defined by taking the elastic buckling stress of a flange plate of width B as a reference value, which gives :

$$k_{fw}=\frac{(\alpha_f+1/\alpha_f)^2+(t_w/t_f)^3\cdot(D/B)\cdot(\alpha_w+1/\alpha_w)^2}{1+(D/B)^3\cdot(t_w/t_f)} \quad \text{(A. 2)}$$

where

$$\alpha_f=a/B \text{ and } \alpha_w=a/D \quad \text{(A. 3)}_{a,b}$$

The half wave length, a, corresponding to the buckling stress is given by :

$$a=B\cdot\left\{\frac{1+(D/B)^3\cdot(t_w/t_f)^3}{1+(B/D)\cdot(t_w/t_f)^3}\right\}^{\frac{1}{4}} \quad \text{(A. 4)}$$

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