[PROC. OF JSCE Structural Eng. / Earthquake Eng. Vol. 2, No. 2 October 1985]

A DESIGN METHOD FOR ORTHOGONALLY STIFFENED PLATES WITH OR WITHOUT STRINGERS SUBJECTED TO UNIAXIAL COMPRESSION

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A design method based on the column approach is proposed for evaluating the ultimate strength of orthogonally stiffened wide plates with some heavy longitudinal stiffeners (stringers) in steel box girder bridges including the main box girders of cable stayed bridges. The method is verified by the elasto-plastic and large deflexion analysis (F. E. M.). It is shown that the dimensions of transverse stiffeners in orthogonally stiffened wide plates with small aspect ratio can be greatly reduced without significant variation of the ultimate strength either by increasing the size of the longitudinal stiffeners or by replacing some longitudinal stiffeners with stringers so proportioned that the elastic buckling strength remains constant. The required stiffness of the flange plate of the stringer to avoid lateral-torsional buckling is also investigated by the F. E. M.

1. INTRODUCTION

The following points should be taken into consideration when designing slender compression flanges having many longitudinal stiffeners.

1) The effect of a non-uniform stress distribution due to shear lag upon the ultimate strength of wide compression flanges may not be negligible, because the non-uniformity of stress distribution is much larger than that of ordinary box girders.
2) The ultimate stress of a longitudinally stiffened panel between transverse stiffeners is close to that of an effective stiffener[1] consisting of a longitudinal stiffener and associated effective plating, because the buckling mode of the multi-stiffened panel with small aspect ratio is flat in the transverse direction except for the small part of the plate panel near the junction edges of flange and web plates. Therefore, the ultimate strength can be somewhat different from the basic strength specified in JSHB[2], which is based on the experimental results of specimens with few longitudinal stiffeners.
3) Transverse stiffeners of a multi-stiffened plate of small aspect ratio designed according to JSHB may lead to impractically large dimensions. This is because JSHB assumes the transverse stiffeners to be at nodal lines of the elastic buckling mode and thus they are required to have a large stiffness.
4) Transverse bending of the box girder cannot be neglected. It is necessary to design the compression flange as a stiffened plate under biaxially applied in-plane stresses.
5) Considerable shearing stress due to bending, torsion and distortion of the main girder occurs in the

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flange plate because of the slender cross-section of main girder. The shearing stress may decrease the ultimate strength of compression flange.

According to the researches by Dowling[4], Lamas[5] and Jetteur-Maquoi-Massonnet[6], the effect of the elastic shear lag phenomenon on the ultimate strength can be generally neglected except for very wide box girders with a span length shorter than about 4 times of their width. However, ductility may be necessary both in the outer longitudinal stiffeners near the webs and in the outer plate panels. A survey of dimensions of flange plates in box girder bridges, constructed in Japan, showed that this effect can be neglected in bridge structures.

The first indication of the problem concerning 3) was reported by Mikami-Dogaki-Yonezawa[9),10], who suggested a design method based on the elastic and elasto-plastic buckling theories such that overall buckling occurs across the transverse stiffeners. Yoshida-Maegawa[11] also presented some figures for the design of orthogonally stiffened plates based on the elasto-plastic buckling theory. The ultimate strength of orthogonally stiffened plates was investigated by Komatsu-Kitada-Nara[12] by F. E. M. Some experimental results[13),14] are also available, but it is difficult to use them for developing a design method because of diffenrences between the boundary conditions of specimens and actual flanges of bridges.

The size of transverse stiffeners can also be reduced by replacing some longitudinal stiffeners with stringers[3]. An experimental study of such orthogonally stiffened plates with stringers under uniaxial compression was carried out by Smith[15], but information useful to this study cannot be obtained, because the specimens were designed against overall buckling.

A lot of researches on the column approach using the effective stiffener for calculating the ultimate strength of longitudinally stiffened plates have been executed, since the collapse of four box girder bridges around 1970. Among them, the methods of Chatterjee-Dowling[16] and Little[17] are adopted respectively in BS 5400 Part 3[7] and the design recommendation proposed for AASHTO[8].

In this paper, a design method taking account of point 1) through 3) mentioned in this section, is proposed for orthogonally stiffened plates with or without stringers. It concerns the design of longitudinal stiffeners, transverse stiffeners and stringers, and also the proposition of both a modified column approach to predict the ultimate strength and a criterion for checking buckling instability. The column approach for longitudinally stiffened plates in Ref. 1) is modified in order to cope with the orthogonally stiffened plates with or without stringers.

Points 4) and 5) are still under research and are not discussed further here.

2. A DESIGN METHOD OF ORTHOGONALLY STIFFENED PLATES WITH STRINGERS

(1) Method for Stiffening Plates under Compression

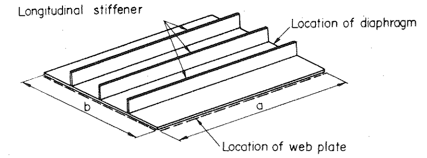
Three kinds of stiffening method in Fig. 1 can be considered in a plate panel with all the edges simply supported at the locations of web plates and diaphragms, where Fig. 1 (a) shows a longitudinally stiffened plate, Fig. 1 (b) an orthogonally stiffened plate whose transverse stiffeners are situated to reduce the size of longitudinal stiffeners and Fig. 1 (c) an orthogonally stiffened plate with a stringer arranged to decrease the dimensions of transverse stiffeners.

(2) Modified Column Approach

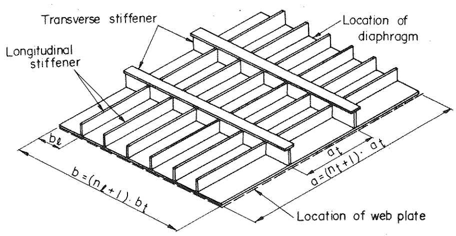
Although local buckling of the plate panels is considered, longitudinal stiffeners, transverse stiffeners and stringers are not subject to local buckling.

In the modified column approach, a pseudo-stiffened plate is used as illustrated in Fig. 2, which has features as follows:

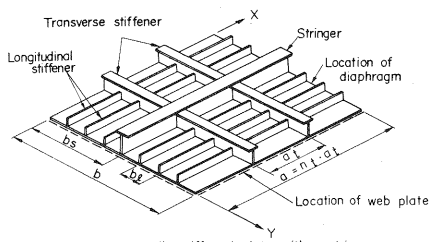
1) No transverse stiffeners or transverse stringers exist in the pseudo-stiffened plate.
2) The cross-section of pseudo-stiffened plate is the same as that of an orthogonally stiffened plate under consideration.



(a) Longitudinally stiffened plate

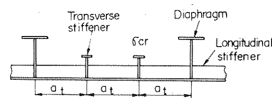


(b) Orthogonally stiffened plate

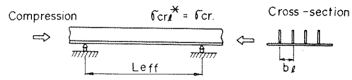


(c) Orthogonally stiffened plate with a stringer

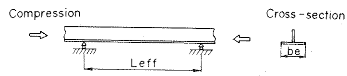
Fig. 1 Various Stiffened Plates.



(a) Original stiffened plate



(b) Pseudo-stiffened plate



(c) Effective stiffener

Fig. 2 Modified Column Approach.

3) The length of pseudo-stiffened plate, L_{eff}, is decided so as to equalize the elastic buckling stress of pseudo-stiffened plate, σ_{crl}^*, to that of the orthogonally stiffened plate, σ_{cr}. The elastic buckling stress, σ_{cr}, is discussed later in the paper. The column approach[1] can be applied to the pseudo-stiffened plate with the slight simplification by neglecting any special treatment for the plate panels beside the unloaded edges. This is justifiable because of the assumption that the effect of restraining deflexion along the unloaded edges upon the ultimate strength is negligible in case of the orthogonally stiffened plate with small aspect ratio.

Therefore, the ultimate stress of stiffened plate, $\bar{\sigma}_{spm}$, is given by :

$$\bar{\sigma}_{spm}=\bar{\sigma}_{esm}\cdot\frac{A_l+b_{eff}\cdot t}{A_l+b_l\cdot t} \quad\cdots\cdots (1)$$

in which $\bar{\sigma}_{esm}$ is the ultimate stress of effective stiffener as follows[1] ;

$$\left.\begin{aligned}\bar{\sigma}_{esm}/\sigma_Y&=1.0, \quad (\lambda_{eff}\leq 0.2)\\ &=-\alpha\cdot\lambda_{eff}^3+\beta\cdot\lambda_{eff}^2-\gamma\cdot\lambda_{eff}+\delta, \quad (0.2<\lambda_{eff}\leq 1.2)\end{aligned}\right\} \quad\cdots\cdots (2)_{a,b}$$

where λ_{eff} is the slenderness parameter of effective stiffener given by :

$$\lambda_{eff}=\frac{1}{\pi}\cdot\sqrt{\frac{\sigma_Y}{E}}\cdot\frac{L_{eff}}{r_{eff}} \quad\cdots\cdots (3)$$

and L_{eff} : effective length of pseudo-stiffened plate, which can be calculated by σ_{cr} in Eq. (10),

A_l : cross-sectional area of longitudinal stiffener,

σ_Y : yield stress,

E : young's modulus,

r_{eff} : radius of gyration of effective stiffener.

The parameter α, β, γ and δ in Eq. $(2)_b$ for some steel grades specified in JSHB are shown in Table 1 together with the nominal yield stress, σ_Y^*. According to Ref. 1), the effective width, b_{eff}, in Eq. (1) is given by :

$$\frac{b_{eff}}{b_l}=\frac{\bar{\sigma}_p}{\sigma_Y}=0.702\cdot R_p^3-1.640\cdot R_p^2+0.654\cdot R_p+0.926 \quad\cdots\cdots (4)$$

where

Table 1 Coefficients α, β, γ and δ.

Steel grade	σ_Y^* (MPa) [2]	σ_{rc}/σ_Y^*	α	β	γ	δ
SS41	235	0.30	0.223	0.550	0.914	1.136
SM50	314	0.25	0.294	0.638	0.886	1.135
SM53	353	0.23	0.299	0.618	0.852	1.135
SM58	451	0.20	0.425	0.820	0.903	1.142

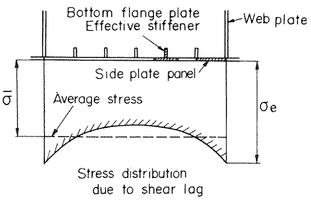


Fig. 3 Stress Distribution in Compression Flange.

$$R_p = 0.526 \cdot \sqrt{\bar{\sigma}_{esm}/E} \cdot (b_l/t) \quad (5)$$

Iterative calculation is necessary in predicting the ultimate stress, because the calculation of Eq. (1) needs b_{eff} of Eq. (4). Good convergence can be obtained by two iterations starting from substituting σ_Y into Eq. (5) instead of $\bar{\sigma}_{esm}$.

The effective width, b_{eff}, obtained from Ref. 26) may be used for biaxial loads, if the transverse load is not predominant.

(3) Criteria for Buckling Instability

As mentioned in the INTRODUCTION, the effect of shear lag phenomenon upon the ultimate strength is considered to be negligible in compression flanges used in bridge girders. Therefore, a criterion given by the following equation can be used to check the buckling instability of orthogonally stiffened plates with or without stringers :

$$\frac{1}{\rho} \cdot \bar{\sigma} \leqq \frac{\bar{\sigma}_{spm}}{\nu} \quad (6)$$

where $\bar{\sigma}$ is the average applied stress as illustrated in Fig. 3 and ρ is a load factor which varies with the combinations of applied loads specified in JSHB, ν being the safety factor expected in JSHB ($\fallingdotseq 1.7$).

The following criterion should also be checked in the side plate panels between web plates and outer longitudinal stiffeners in case of compression flanges with significant shear lag phenomenon, because it seems that ductility is necessary in the side plate panels :

$$\frac{1}{\rho} \cdot \sigma_e \leqq \frac{\bar{\sigma}_p}{\nu} \quad (7)$$

where σ_e is the maximum applied stress in a flange plate and $\bar{\sigma}_p$ is the ultimate stress of a simply supported plate under compression given by substituting the following plate slenderness, R_R, into Eq. (4) instead of R_P.

$$R_R = 0.526 \cdot \sqrt{\sigma_Y/E} \cdot (b_l/t) \quad (8)$$

(4) Elastic Buckling Stress of Orthogonally Stiffened Plates with Stringers

The elastic buckling stress, σ_{cr}, is required not only to find L_{eff} but also to design the stiffeners and stringers.

It can be assumed that a buckling mode of the stiffened plate, illustrated in Fig. 1 (c), is represented by

$$w = w_{mn} \sin \frac{m\pi}{a} x \cdot \sin \frac{n\pi}{a} y \quad (9)$$

Subsequently, σ_{cr} is deduced by applying the Ritz's method to an orthogonally anisotropic plate with discrete stringers on which longitudinal and transverse stiffeners are smeared. Thus, σ_{cr} can be approximated by :

$$\sigma_{cr} = \frac{\pi}{b^2 \cdot t^*} \cdot \left\{ D \cdot \left(\frac{m}{\alpha} + \frac{\alpha}{m} \right)^2 + \left(\frac{m}{\alpha} \right)^2 \cdot \frac{E \cdot I_l}{b_l} + \left(\frac{\alpha}{m} \right)^2 \cdot \frac{E \cdot I_t}{a_t} + \left(\frac{m}{\alpha} \right)^2 \cdot \frac{E \cdot I_s}{b_s} \right\} \quad (10)$$

where

$$t^* = \bar{t} + A_s/b_s = t + A_l/b_l + A_s/b_s \quad (11)$$

$\alpha = a/b$: aspect ratio,

t : thickness of plate panel.,

A_s : cross-sectional area of stringer,

I_l, I_s, I_t : geometrical moment of inertia of longitudinal stiffener, stringer and transverse stiffener with respect to surface to which stiffeners are welded.

The value of σ_{cr} should be minimized with respect to the number of half waves, m, which may vary from 1 to the number of transverse stiffeners, n_t. Simple and practical formulae for calculating σ_{cr} of orthogonally stiffened plates without stringers are detailed in Ref. 9).

(5) Dimensioning Stiffeners and Stringers

A reasonable design method of dimensioning stiffeners and stringers can be proposed, if the ordinary design method adopted in Japan is taken into account as follows :

1) First, the longitudinal stiffeners are so designed that their relative stiffness to plate, γ_l, coincides with the minimum required stiffness specified in JSHB, $\gamma_{l,req}$.
2) Then, the transverse stiffeners are designed according to JSHB so that their geometrical moment of inertia, I_t, is equal to the minimum required stiffness in JSHB, $I_{t,req}$.
3) Finally, if the transverse stiffeners obtained in the above design are too heavey, their size should be reduced either by increasing the stiffness of longitudinal stiffeners or by situating stringers such that the elastic buckling strength remains constant.

In the case of an orthogonally stiffened plate where the longitudinal stiffeners have γ_l greater than $\gamma_{l,req}$, the required geometrical moment of intertia of transverse stiffener, I_t^*, is approximated by :

$$I_t^* \simeq \frac{n_l+1}{4\cdot\eta\cdot\alpha_t^3}\cdot\gamma_{l,req} \qquad (12)$$

which is derived by neglecting the first term in the right hand side of Eq. (10) and equalizing the elastic buckling strength of the orthogonally stiffened panel and the longitudinally stiffened panel and where

$$\eta=\gamma_l/\gamma_{l,req}, \quad \alpha_t=a_t/b \qquad (13)_{a,b}$$

and n_l is the number of longitudinal stiffeners. In a similar way to Eq. (12), the required geometrical moment of inertia of a stringer, I_s^*, is approximately given by :

$$I_s^* \simeq \left\{\frac{t^*}{t}\cdot(n_t+1)^2\cdot\frac{1}{m^2}-\eta\right\}\cdot(n_l'+1)\cdot I_{l,req}-\frac{\alpha^3}{m^4}\cdot\frac{n_t+1}{n_s+1}\cdot I_t \qquad (14)$$

where n_l' : number of longitudinal stiffeners between stringers,

n_s : number of stringers.

In this equation, the same m as in Eq. (10) should be used.

(6) Design Examples

The relationship between I_t^* and I_l are shown in Fig. 4 for the stiffened plate of $b=6.9$ m, $a=7.25$ m, $b_l=40$ cm and $n_t=3$.

It can be seen from Fig. 4 that the variation of I_t^* due to increasing I_l from the minimum required value in

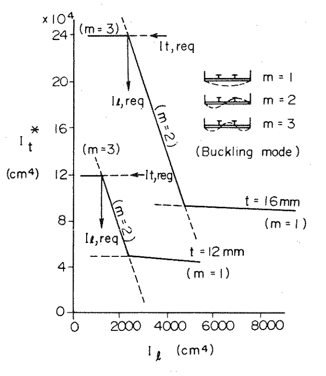


Fig. 4 Relationships between I_t^* and I_l.

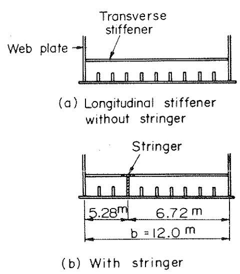


Fig. 5 Orthogonally Stiffened Plates. ($a=9$ m, $n_t=2$).

Table 2 Dimensions of Transverse Stiffeners and Stringer.

(mm)

	Transverse stiffener	Stringer
Without stringer	280 × 14 flange, 9 × 1100 web	
With stringer	240 × 12 flange, 9 × 500 web	300 × 15 flange, 9 × 500 web

JSHB, $I_{l,req}$, is considerable in the case of $m=2$, but slight when $m=1$. Significant reduction in the size of transverse stiffeners can be achieved by designing the orthogonally stiffened plate so that overall buckling occurs.

The minimum required cross-sections of transverse stiffeners and stringer in orthogonally stiffened plates with or without a stringer, illustrated in Fig. 5, are shown in Table 2. As the stringer is not located at the center, it is designed by using the elastic buckling stress[3),21)] derived for an orthogonally stiffened plate with asymmetrical cross-section. Table 2 shows that the size of transverse stiffeners can be considerably reduced by using a stringer.

3. VERIFICATION OF MODIFIED COLUMN APPROACH

(1) Longitudinally Stiffened Plates

The ultimate strength of a longitudinally stiffened plate under uniaxial compression, shown in Fig. 6, was analyzed by F. E. M.[12),23)] in order to verify the column approach in Ref. 1), and to compare with the methods in BS 5400 Part 3[7)], AASHTO[8)], JSHB[2)] and DASt Ri 012[22)].

For this study, only a part of the stiffened plate need be analyzed (see Fig. 6), because all the longitudinal stiffeners are assumed to have the same initial deflexions, and the effect of boundary condition along the unloaded edges upon the ultimate strength of the stiffened plates of $n_l \geq 4$ is to be negligible as reported in Ref. 24). Triangular plate elements and column elements are adopted respectively for the plate and longitudinal stiffeners of F. E. M. model.

The yield stress of the model is 2 400 kg/cm^2 (235 MPa). According to Ref. 25), a residual stress field, shown in Fig. 7, is adopted, where the average compressive stresses are taken as 0.3 σ_Y in the plate panel and 0.2 σ_Y in the stiffeners. The following initial deflection is used in this analysis.

$$w_0=\frac{a_t}{1\,000}\cdot\sin\frac{\pi x}{a_t}+\frac{b_l}{150}\cdot\sin\frac{5\pi x}{a_t}\cdot\sin\frac{\pi y}{b_l} \quad \cdots\cdots (15)$$

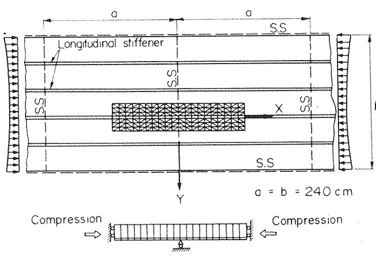


Fig. 6 Analytical Model for Longitudinally Stiffened Plates.

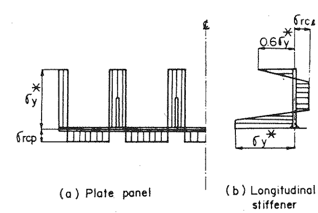


Fig. 7 Residual Stress Distribution.

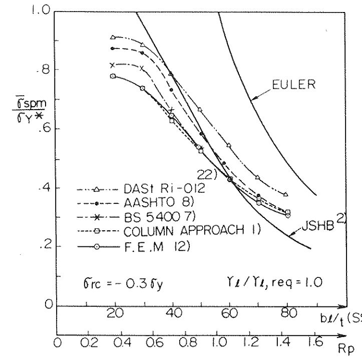


Fig. 8 Ultimate Strength Curves of Longitudinally Stiffened Plate ($\gamma_l/\gamma_{l,req}=1.0$).

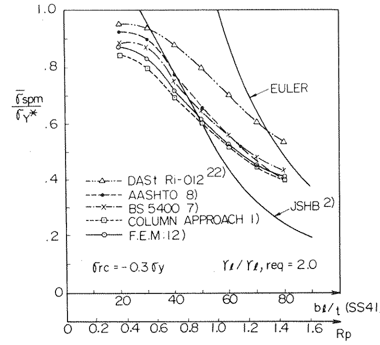


Fig. 9 Ultimate Strength Curves of Longitudinally Stiffened Plate ($\gamma_l/\gamma_{l,req}=2.0$).

These numerical results are plotted in Figs. 8 and 9, where the results given by the column approach agree well with those by the F. E. M. All the specifications except for JSHB also seem to predict good results, if one considers that the magnitudes of relevant initial imperfections are different for each country.

Since $\gamma_{l,req}$ is reduced as a function of $(t_0/t)^2$, where t_0 is the thickness of plate corresponding to $R_R=0.5$, the slope of strength curves in these figures is not steep in the region of stocky plate (say $R_R\leq 0.5$).

(2) Orthogonally Stiffened Plates

In order to examine the accuracy of the modified culumn approach, the ultimate strength of an orthogonally stiffened plate, illustrated in Fig. 10, was analyzed by the F. E. M.[12] and the numerical results are compared with the predicted strength by the modified column approach. Advantage is taken of symmetry to analyse only the portion as shown in Fig. 10.

The parameters in the analysis are as follows:

$R_R=0.71$ and 1.0

$\gamma_l/\gamma_{l,req}=1, 2.5, 5, 7.5$ and $\gamma_l=\gamma^*_{l,req}$

where $\gamma^*_{l,req}$ is the minimum required relative stiffness of longitudinal stiffener in the plate without transverse stiffeners. The pseudo-stiffened plate corresponding to each orthogonally stiffened plate is also analyzed.

The yield stress of the plates is taken as $\sigma_Y=2\,400$ kg/cm² (235 MPa). The same residual stress distribution as in the previous subsection is used. The initial deflexion is given as the superposition of two or three of the following modes:

$$\frac{a}{1\,000}\cdot\sin\frac{\pi x}{a}\cdot\sin\frac{\pi y}{b},\ \left(\text{or}\ \frac{L_{eff}}{1\,000}\cdot\sin\frac{\pi x}{L_{eff}}\cdot\sin\frac{\pi y}{b}\right) \quad \cdots\cdots (16)$$

$$\frac{a_t}{1\,000}\cdot\sin\frac{\pi x}{a_t}\cdot\sin\frac{\pi y}{b},\ \frac{b_l}{150}\cdot\sin\frac{\pi x}{a_t}\cdot\sin\frac{\pi y}{b_l} \quad \cdots\cdots (17, 18)$$

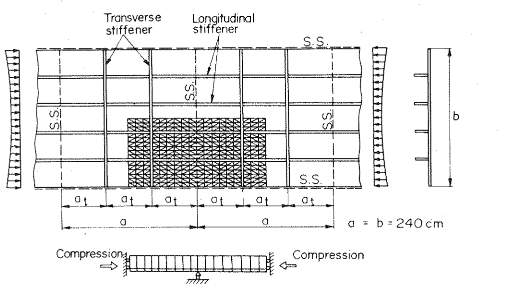


Fig. 10 Analytical Model for Orthogonally Stiffened Plates.

The numerical results are plotted in Fig. 11 together with the results by the modified column approach. The column approach strength is a little conservative, but its variation with $\gamma_l/\gamma_{l,req}$ is similar to that given by the F. E. M. The conservative result in the modified column approach is due to the use of more severe initial deflexion, i. e. it is given by the first term in Eq. (15) rather than that given by Eqs. (16) and (17) used in the F. E. M.

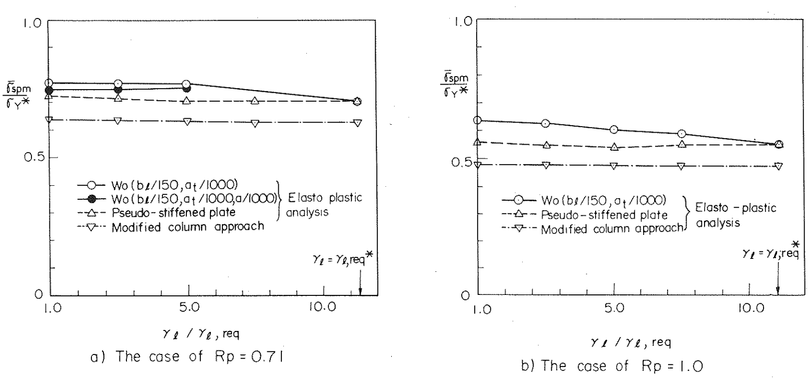


Fig. 11 Variation of Ultimate Strength of Orthogonally Stiffened Plates with Stiffness of Longitudinal Stiffener.

(3) Orthogonally Stiffened Plates with a Stringer

The applicability of the modified column approach to an orthogonally stiffened plate with a stringer is verified by analyzing the elasto-plastic and finite displacement behaviour of two orthogonally stiffened plates with or without a stringer in Fig. 12 by means of the F. E. M.[12] The orthogonally stiffened plate without a stringer has the longitudinal and transverse stiffeners with the minimum required stiffness specified in JSHB. In order to reduce the stiffness of transverse stiffeners to half that in the case without stringer, a stringer is situated on another orthogonally stiffened plate to keep the elastic buckling stress constant.

The material properties and the residual stress distribution are the same as the previous subsections, except that only the global initial deflexion given by Eq. (16) is adopted in this analysis.

The numerical results are shown in Fig. 13. It can be seen that the ultimate strength and the mean compressive stress-centre deflexion curves are not significantly changed by situating the stringer to reduce the size of transverse stiffeners.

The predicted strength by the modified column approach is also plotted in the figure. The conservative result does not deny the applicability of the modified column approach, because it may be caused by the difference in initial deflexions adopted in these methods.

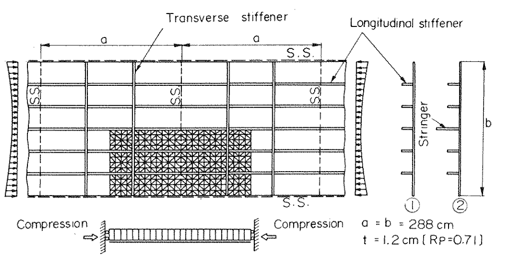


Fig. 12 Analytical Model for Orthogonally Stiffened Plates with/without Stringer.

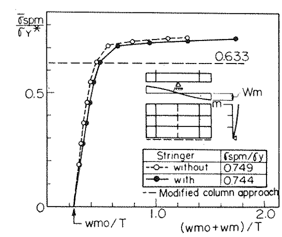


Fig. 13 Mean Compressive Stress-Centre Deflexion Curves.

4. DESIGN METHOD OF STRINGER AGAINST LATERAL TOSIONAL BUCKLING

A method for designing a stringer against lateral-torsional buckling is investigated. As shown in Fig. 14, a continuous compression plate with one longitudinal edge simply supported and the other longitudinal edge stiffened is used as an analytical model for the F. E. M. Although there may be interaction between the stringer and the stiffened plate, it is not considered here because :

1) Analysis is easy.
2) The simple design method can be obtained.
3) The analytical model under uniaxial compression is conservative, because the compression stress at the top of the stringer is smaller than at the bottom due to the existence of the global bending moment of the main girder under consideration.

A required moment of inertia of top stiffener, I_f^*, is proposed by Bleich[18] based on the elastic buckling theory. The verification of I_f^* is examined by the F. E. M.[19,20]

Use is made of triangular plate elements and column elements in the F. E. M. model. The residual stress and initial deflextion, shown in Fig. 14, are adopted. A local initial deflexion having the highest sensitivity to the local buckling

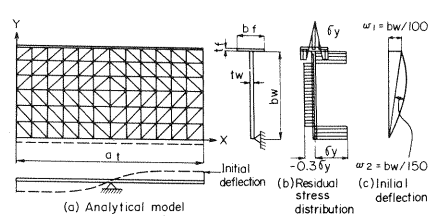


Fig. 14 Analytical Model and Initial Imperfections for Stringer.

strength of the web panel is neglected, because our attention is mainly directed to the ultimate strength of a lateral-torsional buckling mode.

The numerical results are shown in **Fig. 15** for the plates of web slenderness $b_w/t_w=25$ and 35 with the geometrical moment of inertia of top stiffener $I_f=(0.5\sim4.0)\times I_f^*$. In the figure, the ultimate strength of the web panels taken as simply supported plate are also plotted for comparison. It can be seen that the ultimate strength of lateral-torsional buckling mode is greater than the local ultimate strength of the web panel in the stiffness range :

$$I_f/I_f^* \geqq 2.0 \quad \cdots\cdots (19)$$

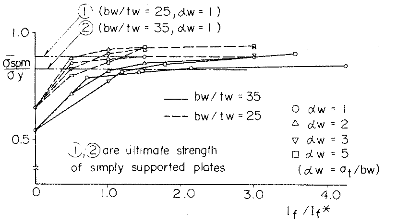


Fig. 15 Relationships between σ_{spm}/σ_Y and I_f/I_f^*.

5. CONCLUSION

(1) A modified column approach for calculating the ultimate strength of orthogonally stiffened plates with or without stringers under uniaxial compression is proposed by a natural extension of the ordinary column approach for longitudinally stiffened plates.

(2) The ultimate strength does not significantly change, when the size of transverse stiffeners is reduced if either the stiffness of longitudinal stiffeners is increased or stringers are situated such that the elastic buckling strength remains constant.

(3) The required moment of inertia of the top stiffener of the stringer should be taken as twice the value proposed by Bleich.

6. ACKNOWLEDGEMENTS

This study was financially supported by the Ministry of Education of Japan and the Hanshin Expressway Public Corporation. The authors are indebted to Mr. J. Noguchi of Sogo Engineering Inc., and Mr. A. Nohara of Kobe Steel Ltd. (a formerly post-graduated Student of Osaka City University) for their helps in the numerical calculations. Thanks are also due to Dr. A. F. Dier of Imperial College for his helpful advices in finishing this paper.

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(Received April 23 1985)