ROZPRAWY INŻYNIERSKIE • ENGINEERING TRANSLATIONS • 36, 1, 167–179, 1988
Polska Akademia Nauk • Instytut Podstawowych Problemów Techniki

SOME ASPECTS OF MODE INTERACTION IN THIN-WALLED STIFFENED PLATE UNDER UNIFORM COMPRESSION

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Interaction of nearly simultaneous buckling modes in the presence of imperfections is studied. The investigation is concerned with infinitely wide plate with thin-walled stiffeners under uniform compression. The asymptotic expansion established by BYSKOV and HUTCHINSON [1] is also used here. The present paper is devoted to the improved study of equilibrium path in the initial post-buckling behaviour of imperfect structures. The results include effects of interaction of the „primary" local mode and a „secondary" local mode having the same wavelength as the primary one. In this paper the analysis of a few buckling modes interaction is presented.

NOTATION

l	length of the stiffened plate,
b_i	width of wall i of the plate,
h_i	thickness of wall i of the plate,
E	Young's modulus,
D_i	flexural rigidity of wall i,
u_i, v_i, w_i	displacements of middle surface,
$\overset{\circ}{u}_i, \overset{\circ}{v}_i, \overset{\circ}{w}_i$	prebuckling displacement fields,
$\underset{n}{u}_i, \underset{n}{v}_i, \underset{n}{w}_i$	buckling displacement fields,
Δ	measure of the applied pressure,
N_{ix}, N_{iy}, N_{ixy}	in-plane stress resultants for wall i,
n	number of mode,
m	number of axial half-waves of mode number n,
λ	scalar load parameter,
λ_n	value of λ at bifurcation mode number n,
λ_s	maximum value of λ for imperfect stiffened plate,
$\underline{\xi}_n$	amplitude of buckling mode number n,
ξ_n	imperfection amplitude corresponding to ξ_n,
a_{ijJ}	postbuckling coefficients (see BYSKOV and HUTCHINSON [1]),
$\sigma_n^* = 10^3\sigma_n/E$	dimensionless stress of mode number n,
σ_s^*	limit dimensionless stress.

1. INTRODUCTION

In compression members containing thin-walled plates local buckling of the plate elements and Euler type buckling of the whole structure can occur.

Interaction between the buckling modes may result in an imperfection-sensitive structure and is the principal cause of collapse of thin-walled structures.

In recent years many papers have been devoted to the analysis of the interaction of buckling modes as a factor that determines the construction sensitivity to imperfections at nearly the same magnitudes of bifurcational loads corresponding to different buckling modes and to the closely related problem of optimum structural desing.

KOITER and van der NEUT [2] have proposed a technique in which the interaction of an overall mode with two local modes having the same wavelength have been considered. Some examples of practical interest have been studied.

The fundamental mode is henceforth called „primary" and the nontrivial higher mode (having the same wavelength as the „primary" one) corresponding to the mode triggered by overall long-wave (bending) mode is called „secondary".

SRIDHARAN and ALI [3] have presented an analysis of 3-mode interaction using a finite strip method for thin-walled columns having doubly symmetric cross-sections as regards the secondary order solution.

Some works concerning only the interactions between the two independent buckling modes of thin-walled structures have been done by KOITER [4], BENITO and SRIDHARAN [5], SRIDHARAN [6], MANEVIČ [7, 8], KOLODJAŽNYJ [9], KOŁAKOWSKI [10].

In the present paper the initial post-buckling behaviour of wide plates with thin-walled stiffeners in the elastic range being under compression is examined on the basis of Byskov and Hutchinson's method with the co-operation between all the walls of the structures being taken into account. The solutions obtained include effects of interaction of some modes having the same wavelength problems of shear-lag and cross-sectional distributions.

2. STRUCTURAL PROBLEM

A large plate with thin-walled stiffeners at a distance l, simply supported at both ends is considered. Types of cross-sections of such structures consisting of a few flat plates with perpendicular axis of symmetry and local coordinate systems assumed are presented in Fig. 1. Materials of the stiffened plate obey Hooke's law.

The membrane strains of wall i are

$$
\begin{aligned}
\varepsilon_{ix} &= u_{i,x} + 0.5(w_{i,x}^2 + v_{i,x}^2),\\
\varepsilon_{iy} &= v_{i,y} + 0.5(w_{i,y}^2 + u_{i,y}^2),\\
\gamma_{ixy} &= u_{i,y} + v_{i,x} + w_{i,x} w_{i,y}
\end{aligned}
\tag{2.1}
$$

and the bending strains are given by

$$
\kappa_{ix} = -w_{i,xx}, \quad \kappa_{iy} = -w_{i,yy}, \quad \kappa_{ixy} = -w_{i,xy}. \tag{2.2}
$$

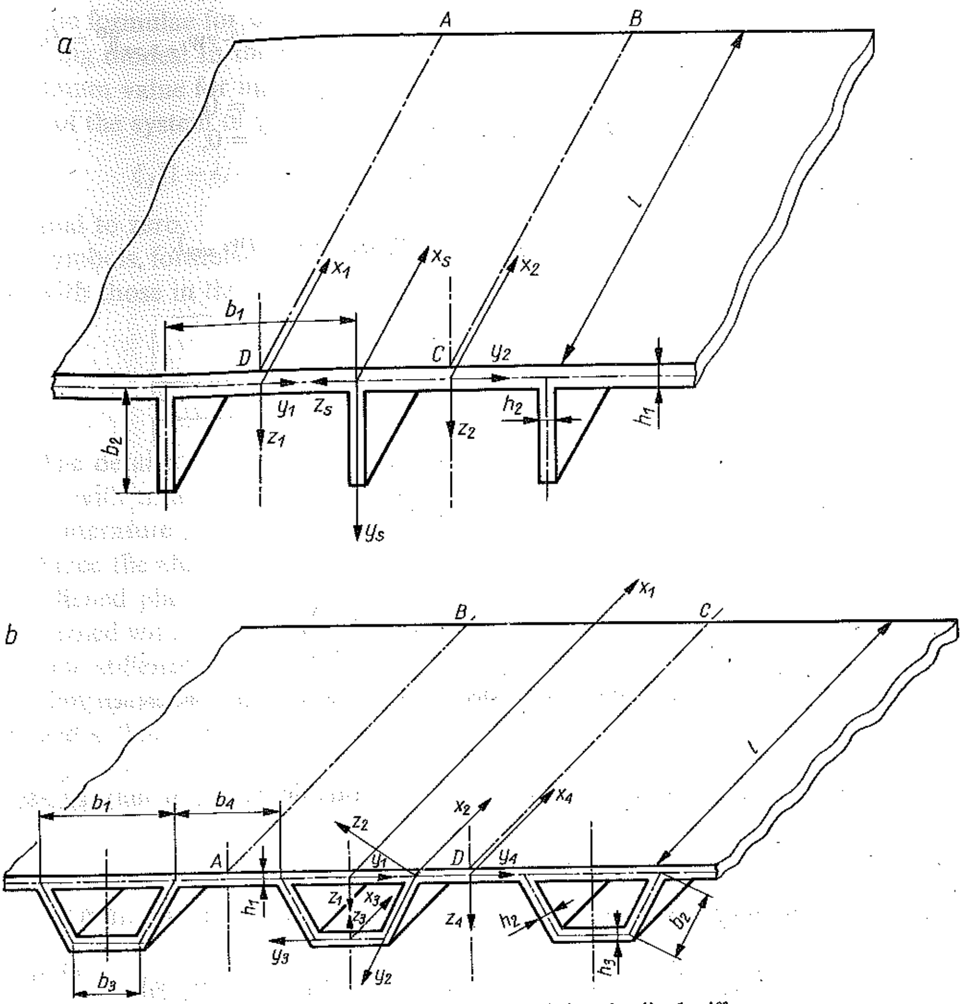

FIG. 1. Part of a wide plate with longitudinal stiffeners.

The differential equilibrium equations resulting form the virtual work principle written for one wall are

$$
\begin{aligned}
&-N_{ix,x}-N_{ixy,y}-(N_{iy}u_{i,y})_{,y}=0,\\
(2.3)\qquad &-N_{iy,y}-N_{ixy,x}-(N_{ix}v_{i,x})_{,x}=0,\\
&D_i\nabla\nabla w_i-(N_{ix}w_{i,x})_{,x}-(N_{iy}w_{i,y})_{,y}-(N_{ixy}w_{i,x})_{,y}-(N_{ixy}w_{i,y})_{,x}=0.
\end{aligned}
$$

The geometrical and statical continuity conditions at the junctions of plates (Fig. 2) may be written as

$$
\begin{aligned}
w_i|^{+} &= w_s|^{0}\cos\alpha+v_s|^{0}\sin\alpha = w_{i+1}|^{0},\\
v_i|^{+} &= v_s|^{0}\cos\alpha-w_s|^{0}\sin\alpha = v_{i+1}|^{0},\\
w_{i,y}|^{+} &= w_{s,y}|^{0} = w_{i+1,y}|^{0},
\end{aligned}
$$

$$
\begin{aligned}
&D_i(w_{i,yy}+\nu w_{i,xx})|^{+}-D_s(w_{s,yy}+\nu w_{s,xx})|^{0}-D_{i+1}(w_{i+1,yy}+\nu w_{i+1,xx})|^{0}=0,\\
(2.4)\qquad &u_i|^{+}=u_s|^{0}=u_{i+1}|^{0},\\
&N_{iy}|^{+}-N_{sy}|^{0}\cos\alpha+Q_{sy}|^{0}\sin\alpha-N_{i+1}|^{0}=0,\\
&Q_{iy}|^{+}-N_{sy}|^{0}\sin\alpha-Q_{sy}|^{0}\cos\alpha-Q_{i+1}|^{0}=0,\\
&N_{ixy}|^{+}-N_{sxy}|^{0}-N_{i+1xy}|^{0}=0,
\end{aligned}
$$

where

$$Q_{iy}=N_{iy}w_{i,y}+N_{ixy}w_{i,x}-D_i(w_{i,yyy}+(2-\nu)w_{i,xxy}).$$

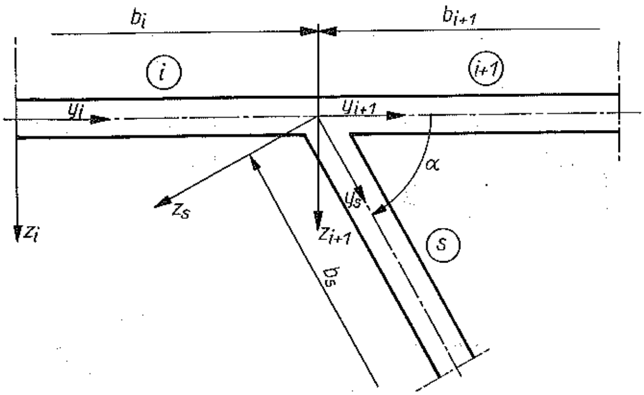

FIG. 2. Local coordinate systems for each plate meeting at the corner.

The prebuckling solution consists of homogeneous fields and it may be assumed that

$$(2.5)\qquad \mathring{u}_i=-x_i\Delta,\qquad \mathring{v}_i=\nu y_i\Delta,\qquad \mathring{w}_i=0.$$

The boundary conditions enable us to write the first order solution

$$
\begin{aligned}
&\overset{n}{w}=(\overset{n}{C}_{1i}\operatorname{ch}(\overset{n}{r}_{1i}y_i)+\overset{n}{C}_{2i}\operatorname{sh}(\overset{n}{r}_{1i}y_i)+\overset{n}{C}_{3i}\cos(\overset{n}{r}_{2i}y_i)+\overset{n}{C}_{4i}\sin(\overset{n}{r}_{2i}y_i))\sin\frac{m\pi x_i}{l},\\
(2.6)\quad &\overset{n}{u}_i=(\overset{n}{C}_{5i}\operatorname{ch}(\overset{n}{r}_3y_i)+\overset{n}{C}_{6i}\operatorname{sh}(\overset{n}{r}_3y_1)+\overset{n}{C}_{7i}\operatorname{ch}(\overset{n}{r}_4y_i)+\overset{n}{C}_{8i}\operatorname{sh}(\overset{n}{r}_4y_i))\cos\frac{m\pi x_i}{l},\\
&\overset{n}{v}_i=(-\overset{n}{C}_{5i}\overset{n}{b}_1\operatorname{sh}(\overset{n}{r}_3y_i)-\overset{n}{C}_{6i}\overset{n}{b}_1\operatorname{ch}(\overset{n}{r}_3y_i)-\overset{n}{C}_{7i}\overset{n}{b}_2\operatorname{sh}(\overset{n}{r}_4y_i)-\overset{n}{C}_{8i}\overset{n}{b}_2\operatorname{ch}(\overset{n}{r}_4y_i))\sin\frac{m\pi x_i}{l},
\end{aligned}
$$

where $\overset{n}{r}_{1i}$, $\overset{n}{r}_{2i}$, $\overset{n}{r}_3$, $\overset{n}{r}_4$, $\overset{n}{b}_1$, $\overset{n}{b}_2$ — see paper [10].

For some values of the load parameter the trigonometric functions $(2.6)_1$ have to be transformed into suitable hyperbolic functions. The bifurcation load λ_n is the smallest value of the parameter for any integer m for which the determinant of the coefficients of conditions (Eqs. (2.4)) vanishes.

The global buckling mode occurs at $m=1$ and the local modes occur at $m\neq 1$. All the modes are normalized so that the maximum normal displacement is equal to the skin plate h_1.

The formulae for the postbuckling coefficients a_{ijJ} [1, 10] involve only the buckling modes. In the points where the scalar load parameter λ_s reaches the maximum value for imperfect structure (bifurcation or limit points), the Jacobian of the system of nonlinear equations [10]

$$\xi_J(1-\lambda/\lambda_J)+\xi_i\xi_j a_{ijJ}+\ldots = \lambda/\lambda_J\bar{\xi}_J \qquad \text{at} \qquad J = 1, \ldots, n \tag{2.7}$$

is equal to zero.

Symbols, formulae and methods of solution applied in this paper are identical with those in the paper [10] (see Appendix).

3. Results

The detailed numerical calculations have been performed for several wide plates with thin-walled longitudinal stiffeners, the geometry of which is known from literature [5, 7, 10].

Since the effect of shear lag is more pronounced in stiffened plates than in unstiffened plates of the same extensional rigidity, the designer is even more concerned with the interaction of shear lag and collapse by buckling in the case of wide stiffened flanges.

Simply supported plates of infinite width with longitudinal, regularly arranged stiffeners are considered in the present paper.

Types of cross-sections, basic dimensions of the plates and the local coordinate systems assumed are presented in Figs. 1a—b.

Due to the symmetry with respect to the longitudinal centre lines of each skin plate, only action of a typical panel contained between two succesive centre lines is considered.

For a wide plates reinforced by equispaced narrow rectangular stiffeners ofthe following dimensions [7] (Fig. 1a):

$b_1/b_2 = 4.226, \quad h_1/h_2 = 2.367, \quad l/b_1 = 2.591, \quad b_1/h_1 = 35.714,$

the values of dimensionless stress $\sigma_n^*(\nu = 0.3)$, the number of half-waves m for the local buckling modes are shown in Table 1.

Table 1. Dimensionless stresses σ_n^* for different buckling modes of wide plate with rectengular stiffeners.

m	1	2	3	4	5	6	7	8	9	10
σ_n^*	2.109	2.940	2.794	3.116	3.003	2.845	2.843	2.953	3.147	3.407

In Fig. 3a the first two overall buckling modes are shown and the first three local modes at half-waves $m = 3$ and $m = 7$ are presented in Fig. 3b and Fig. 3c, respectively.

At the free ends of the stiffeners, conditions corresponding to a completely free edge have been assumed $M_y = Q_y = N_y = N_{xy} = 0$.

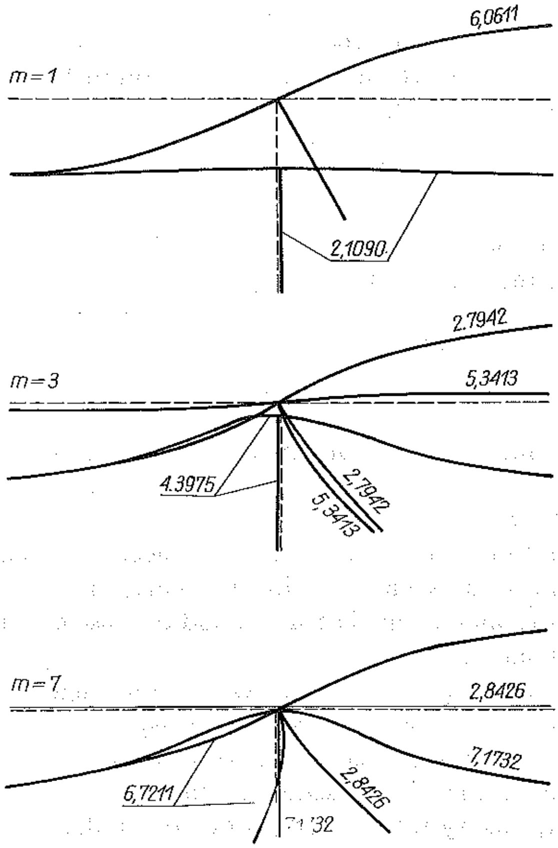

FIG. 3. Two global and several local modes for the wide plate with rectangular stiffeners.

It can easily be noticed that for such dimensions of the stiffened plate two local buckling modes at different numbers of half-waves occur almost simultaneously. In this case the primary local mode at $m = 3$ may be called „local mode of skin plate" and in the case $m = 7$ the primary local mode „local mode of stiffeners".

Table 2 shows the values of the ration of dimensionless limit stress σ_s^* to dimensionless global stress σ_1^* for various imperfection values and for some possible combinations of buckling modes, presented in Fig. 3. The bottom index at the imperfection denotes, respectively: *1* — basic global buckling mode ($\sigma_1^* = 2.109$); *2* — the first local buckling mode at $m = 3$; *3* — the first local buckling mode at $m = 7$. Furthermore, the following code has been used in Table 2, in order to index the buckling mode: the first figure denotes the mode of buckling in the same fashion as the bottom index does in the case of imperfections; the second figure denotes the first, second or third local buckling mode for a previously fixed number of half-waves (e.g. 23-refers to the third local mode ($\sigma^* = 5.341$) at $m = 3$).

Table 2. Limit dimensionless stresses carried by the wide plate with rectengular stiffeners

	Interaction of n modes	σ_s^*/σ_1^*					
		$\bar{\xi}_1 = 1.0$, $\bar{\xi}_2 = 0.05$, $\bar{\xi}_3 = 0.05$	$\bar{\xi}_1 = 1.0$, $\bar{\xi}_2 = 0.2$, $\bar{\xi}_3 = 0.2$	$\bar{\xi}_1 = 1.0$, $\bar{\xi}_2 = 0.05$, $\bar{\xi}_3 = 0$	$\bar{\xi}_1 = 1.0$, $\bar{\xi}_2 = 0.2$, $\bar{\xi}_3 = 0$	$\bar{\xi}_1 = 1.0$, $\bar{\xi}_2 = 0$, $\bar{\xi}_3 = 0.05$	$\bar{\xi}_1 = 1.0$, $\bar{\xi}_2 = 0$, $\bar{\xi}_3 = 0.2$
1	5—11, 21, 23, 31, 32	0.3224	0.2736	0.3587	0.3566	0.3223	0.2738
2	5—11, 12, 31, 32, 33	—	—	—	—	0.3223	0.2738
3	4—11, 12, 31, 32	—	—	—	—	0.3224	0.2738
4	3—11, 31, 32	—	—	—	—	0.3224	0.2738
5	3—11, 12, 31	—	—	—	—	0.3225	0.2740
6	5—11, 12, 21, 22, 23	—	—	0.4793	0.4350	—	—
7	4—11, 12, 21, 23	—	—	0.4793	0.4351	—	—
8	3—11, 21, 23	—	—	0.4782	0.4340	—	—
9	3—11, 12, 21	—	—	0.7306	0.7309	—	—
10	2—11, 31	—	—	—	—	0.3225	0.2740
11	2—11, 21	—	—	0.7306	0.7309	—	—

Table 3. Limit dimensionless stresses carried by the wide plate with closed section longitu dinal stiffeners.

	Interaction of n modes	σ_s^*/σ_1^*											
		$\bar{\xi}_1$	$\bar{\xi}_2$	$\bar{\xi}_3$	$\bar{\xi}_1$	$\bar{\xi}_2$	$\bar{\xi}_3$	$\bar{\xi}_1$	$\bar{\xi}_2$	$\bar{\xi}_3$	$\bar{\xi}_1$	$\bar{\xi}_2$	$\bar{\xi}_3$
		−1.0	0.05	0.0	−1.0	0.2	0.0	1.0	0.0	0.05	1.0	0.0	0.2
1	3—11, 21, 22	0.7112			0.6025			—			—		
2	2—11, 21	0.7607			0.6650			—			—		
3	3—11, 31, 32	—			—			0.5123			0.3110		
4	2—11, 31	—			—			0.5266			0.3253		

In each case a set of $n+1$ nonlinear equations have been solved for n buckling modes under the condition that Jacobian of this set of Egs. (2.7) must be equal to zero. Numerical calculations have proved that the interaction of local modes having considerable different wavelengths is either very weak or does not occur at all. Therefore the interaction of the global mode(s) and a few local buckling modes may be considered. According to the assumptions mode in Byskov and Hutchinson's theory [1], local buckling modes (and the secondary global modes) do not interact explicitly. However, the interaction occurs through the interaction of each of them with the (primary) global mode.

In paper [10], in the case of a wide plate with closed section longitudinal stiffeners of dimensions (Fig. 1b)

$$b_2/b_1 = 0.7166, \quad b_3/b_1 = 0.4933, \quad b_4/b_1 = 1.0,$$
$$l/b_1 = 12.0 \quad h_2/h_1 = 0.6, \quad h_3/h_1 = 0.6, \quad b_1/h_1 = 30.0;$$

the following results were obtained:

$$\sigma_1^* = 3.9449, \quad \sigma_2^* = 3.9271(m = 13), \quad \sigma_3^* = 3.9349(m = 20).$$

Due to a very high torsional rigidity of closed stiffeners, a vertical symmetry axis was assumed for the segment under consideration. All the buckling modes are normalized so that the maximum normal displacement of the skin plate is equal to the plate thickness h_1.

Indices of the buckling modes applied here, are identical with those used for the plate considered above.

Table 3 contains the values of the ratio of limit stress σ_s^* to global stress σ_1^*, the 2- and 3-mode interaction approach for various imperfection values being taken into account.

On the basis of the results obtained for wide stiffened plate it is possible to conclude that the problem of interaction of the overall buckling mode with the primary and secondary local modes having the same shape as the global one is of great significance (for example compare cases 8 and 11 in Table 2). This effect is contained in the term $\sigma_1 \cdot l_{11}(u_l, u_k)$ (where $l, k = 2, 3$) in coefficients a_{ijJ} of the equations (2.7). In other cases it is moderate (cases in Table 3) ar absent- -„local mode stiffeners" (cases 4 and 10 in Table 2).

In the case of thin-walled open structures of low flexural-torsional rigidity, where stresses corresponding to higher global modes are of the same order as those corresponding to local modes, it may be necessary to take into account interaction of the „secondary" global mode with the others (4-mode approach).

SRIDHARAN [4] considered the interaction of two buckling modes of wide plate with rectangular stiffeners for which the overall mode is assumed to consist of two half-waves to model the „clamped" end conditions, and having the following dimensions (Fig. 1a):

$$b_1/b_2 = 2.0, \quad h_1/h_2 = 1.0, \quad b_1/h_1 = 50, \quad l/b_1 = 26.4.$$

In this case the postbuckling equilibrium paths are governed by the secondary order solution only.

Having assumed those geometric dimensions of the plate, calculations have been carried out in terms of the three-mode approach and it has been found that all components of the first approximation, i. e. coefficients a_{ijJ} of the set of equations (2.7) are equal to zero.

Next, the same dimensions of a wide stiffened plate cross-section and identical wave-length of global buckling have been assumed (the global buckling mode corresponds to one half-wave, i. e. $l/b_1 = 13.2$). Results of the calculations are

$$\sigma_1^* = 1.187, \qquad \sigma_2^* = 1.157(m = 11), \qquad \sigma_3^* = 2.793(m = 11),$$

where indices 1, 2, 3 denote the global, the primary and secondary local modes, respectively.

Stress values σ_1^* and σ_2^*, obtained here, are identical with those obtained by SRIDHARAN [4] for two-mode approach, at the same buckling length. However, following major terms of the first order solution (i. e. $\xi_1\xi_2^2$, $\xi_1\xi_3^2$, $\xi_1\xi_2\xi_3$), have turned out to be non-zero.

Table 4. Limit dimensionless stresses carried by the plate with rectengular stiffeners.

Imperfection amplitudes			2-mode approach	3-mode approach	$\left\|\frac{2-3}{3}\right\|$
$\bar{\xi}_1$	$\bar{\xi}_2$	$\bar{\xi}_3$	σ_s^*/σ_1^*	σ_s^*/σ_1^*	%
	1		2	3	4
0.1	0.0	0.0	0.8986	0.8942	0.5
0.1	0.05	0.0	0.7738	0.7605	1.7
0.1	0.2	0.0	0.6363	0.6112	4.1
0.5	0.0	0.0	0.8017	0.7843	2.2
0.5	0.05	0.0	0.7160	0.6915	3.5
0.5	0.2	0.0	0.6050	0.5715	5.8
1.0	0.0	0.0	0.7361	0.7079	4.0
1.0	0.05	0.0	0.6670	0.6334	5.3
1.0	0.2	0.0	0.5736	0.5345	7.3

In Table 4 the ratios of limit stress σ_s^* to the global stress σ_1^* are presented for different imperfections, two- and three-mode approaches being taken into account. As the imperfection becomes more significant, the difference between these two approaches increases, too.

It is seen that also in this case, local mode imperfections promote an interaction between the local mode (s) and the global mode.

Therefore an *a priori* application of the results obtained for the same buckling length but a different number of half-waves m, may bear even a qualitative error resulting in the appearance of a catastrophic failure.

4. Conclusions

The initial post-buckling behaviour of a thin-walled infinitely wide plate with longitudinal stiffeners has been presented on the basis of Byskov and Hutchinson's method, co-operation of all walls of the structure being taken into account.

The solutions given here are valid in the case of uniform compression. The present approach takes into account the secondary local mode activated by the interaction of the overall mode with the primary local mode.

In the case when a few buckling modes are comparable, disregarding the mode interaction may lead to overestimation of the load carrying capacity of the structure.

The paper is aimed at drawing attention to the question associated with disregarding the key terms of the first order solution.

Appendix

The asymptotic method

The method outlined in the following was developed by Byskov and Hutchinson in [1] where a complete derivation is given. This method is suitable for structures with M simultaneous or nearly simultaneous buckling modes.

Assume that the structure is perfect and that the prebuckling state is linear with respect to the scalar load parameter λ. The displacement field is expanded in the following fashion

$$u = \lambda u_0 + \xi_i u_i + \xi_i \xi_j u_{ij} + \ldots, \tag{A.1}$$

where the prebuckling displacement field is described by λu_0, the amplitude ξ_i measures the influence of buckling mode u_i, and u_{ij} is the second order field associated with u_i and u_j. The stress and strain fields are expanded in a fashion similar to (A.1).

The material is assumed to be linearly elastic. The dot used in the following denotes integration over the entire structure

$$\sigma \cdot \varepsilon = \int_v \sigma_{ij} \varepsilon_{ij} dv. \tag{A.2}$$

The eigenvalue problems determining the buckling modes and their associated eigenvalues λ_J are found from the variational equation

$$\text{(A.3)} \qquad \sigma_J \cdot l_1(\sigma u) + \lambda_J \sigma_0 \cdot l_{11}(u_J, \delta u) = 0, \qquad J = 1, ..., M,$$

where δu denotes all kinematically admissible variations of u. The buckling modes are taken to be mutually orthogonal in the following sense

$$\text{(A.4)} \qquad \sigma_0 \cdot l_{11}(u_i, u_j), \qquad i \neq j.$$

If the structure contains geometric imperfections $\bar{u}$ given by $\bar{u} = \bar{\xi}_i u_i$, the following M nonlinear equations determine the equilibrium paths

$$\text{(A.5)} \qquad \xi_J\left(1 - \frac{\lambda}{\lambda_J}\right) + \xi_i \xi_j a_{ijJ} + ... = \frac{\lambda}{\lambda_J}\bar{\xi}_J, \qquad J = 1,...,M.$$

The formula for the coefficients are

$$\text{(A.6)} \qquad a_{ijJ} = [\sigma_J \cdot l_{11}(u_i, u_j) + 2\sigma_i \cdot l_{11}(u_j, u_J)]/(2\sigma_J \varepsilon_J).$$

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STRESZCZENIE

PEWNE ASPEKTY WSPÓŁDZIAŁANIA RÓŻNYCH POSTACI WYBOCZENIA PŁYTY WZMOCNIONEJ CIENKOŚCIENNYMI ŻEBRAMI I PODDANEJ RÓWNOMIERNEMU ŚCISKANIU

Rozważono współdziałanie prawie równoczesnych postaci wyboczenia nieskończenie szerokiej płyty wzmocnionej cienkościennymi żebrami w obecności imperfekcji konstrukcyjnej. Zastosowano również rozwinięcia asymptotyczne BYSKOVA i HUTCHINSONA [1]. Praca poświęcona jest pogłębionej analizie stanu równowagi konstrukcji z imperfekcjami w początkowym zakresie pokrytycznym. Otrzymane wyniki wykazują efekt współdziałania „podstawowej", lokalnej postaci wyboczenia oraz „drugorzędnej" postaci lokalnej o tej samej liczbie półfal. W pracy przedstawiono analizę współdziałania kilku postaci wyboczenia.

Резюме

НЕКОТОРЫЕ АСПЕКТЫ ВЗАИМОДЕЙСТВИЯ РАЗНЫХ ВИДОВ ПРОДОЛЬНОГО ИЗГИБА ПЛИТЫ УПРОЧНЕННОЙ ТОНКОСТЕННЫМИ РЕБРАМИ И ПОДВЕРГНУТОЙ РАВНОМЕРНОМУ СЖАТИЮ

Рассмотрено взаимодействие почти одновременных видов продольного изгиба бесконечной плиты, упрочненной тонкостенными ребрами, в присутствии конструкционных дефектов. Применены тоже асимптотические разложения Быскова и Гатчинсона [1]. Работа посвящена углубленному анализу состояния равновесия конструкции с дефектами в начальной стадии после поетри устойчивости. Полученные результаты указывают на эффект взаимодействия „основного", локального вида продольного изгиба и „второстепенного", локального вида с этой же самой длиной волны. В работе представлен анализ взаимодействия нескольких таких видов продольного изгиба.

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Received May 7, 1987.