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OF TROUGH GIRDER

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Elasticity, finite strips, local buckling, postbuckling,
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A finite strip method of analysis is used to study the linear local
buckling and nonlinear post-local buckling behaviour of
isolated trough girders in bending. The nonlinear stiffness
equations are given, and the accuracy of these is demonstrated
by a comparison with tests and independent theoretical
solutions. The variation of the local buckling coefficient
with cross-sectional parameters is demonstrated, and a design
equation is proposed. It is shown that local buckling can be
delayed by the provision of a longitudinal web stiffener, and
a design equation is proposed. The postbuckling of a typical
trough girder is then considered, with the moment-curvature
relationship, stress distribution and deformations being
addressed.
1. INTRODUCTION

Composite box girder bridges, consisting of a reinforced concrete deck above a steel trough, have been used as an important structural system for many years. A rectangular trough, such as that shown in Fig. 1, is usually first placed across the bridge span, and the system is then made composite after the concrete deck is poured and reaches its desired minimum compressive strength. In unpropped construction, the wet concrete and any construction loads render the slender trough girder liable to local instability. Local buckling in bending is not usually arrested by the provision of vertical shear stiffeners (Trahair and Bradford 1988), owing to the short wavelengths typical of this type of buckling. The local and post-local instability of trough plate assemblages in bending is considered in this paper.

In the early days of the design of thin-walled plate assemblages, local buckling was based on the simplified methods for plates in compression. In this approach, the local buckling of the plate assembly was analysed approximately by assuming that the plate elements were hinged along their common boundaries, so that each plate acted as if simply supported along its common boundary or boundaries and free along any unconnected boundary. The critical stress of each plate was determined using the appropriate buckling stress, and the lowest of these was used as an approximation for the critical stress $\sigma_{cr}$ of the member. This approximation is conservative, because the rigidity of the joints between the plate elements causes all plates to buckle simultaneously at a stress intermediate between the lowest and the highest of the critical stresses of the individual plate elements.

A comprehensive approach to some simple problems involving local buckling of open sections in uniform compression was given by Stowell and Lundquist (1939). However, credit for the first extensive analysis of the local stability of structural sections seems to belong to Lundquist, Stowell and Schuette (1943), who applied the method of moment distribution to the stability of structures composed of plates in uniform compression. Experimental work by Kollbrunner (1946), Heimerl (1947) and Chilver (1951) confirmed the analytical methods of Lundquist et al.

Between 1950 and 1970, the local buckling of structural sections had been studied by several investigators, mainly using the Rayleigh-Ritz energy formulation for the analysis. The importance of the analysis of local buckling of structural sections was stimulated by the widespread application of thin, cold formed members whose buckling mode is primarily local instability of the component flats. The determination of the critical load for local buckling of thin-walled sections under concentric compression has been studied by Chilver (1951), Bleich (1952), Van der Maas (1954), Becker (1957), Divakaran (1966), and Bulson (1967). A summary of the critical loads for a variety of structural sections is given by Bulson (1970) in his well-known text.
The matrix methods for the stability analysis of plates and structural sections have been studied extensively in recent years, and computer programs for the analysis have been developed. The finite element method of analysis is one such computer application, and has been used by Kapur and Hartz (1966), Przemieniecki (1968, 1972, 1973), Wittrick (1968) and many others to analyze the buckling of structural sections under uniform compression. A modification of the finite element method is the semi-analytical finite strip method (Cheung 1976, Graves Smith and Sridharan 1978a, Hancock 1978, Bradford 1983), which has been used to obtain the local buckling stresses of plates and sections. The finite strip method is used in this paper.

The analysis of the influence of uniform moment and moment gradient in promoting the local buckling of beams has received far less attention than the analysis of plates under uniform compression. As for the latter case, difficulty was encountered at first in the solution of this type of problem in determining the restraint imposed by adjacent plates on eachother. A series of papers by Rhodes and Harvey (1970, 1975, 1976) employ the Rayleigh-Ritz variational principle to obtain the elastic local buckling loads for thin-walled channel sections in combined uniform bending and compression. Their analysis makes no attempt to calculate the restraints on each plate, but allows the plates to deform in any way compatible with the deformations of the adjacent plates. Using a complex finite strip method, Plank and Wittrick (1974) obtained buckling stresses for plain channels in uniform bending about an axis of symmetry. Recently, Hancock (1985) studied the local buckling of I-beams in uniform bending, while Bradford (1985) considered box sections in bending and compression. Both of the latter studies employed the semi-analytical finite strip method.

Bifurcative local buckling does not usually represent a strength limit state, since it is well-known that plates and sections possess a substantial reserve of postbuckling strength (von Karman 1910). Although research into the post-local buckling behaviour of isolated plates has been plentiful, research into the postbuckling of plate assemblies has been limited. Most of the available solutions for the postbuckling of plates are based on solutions of the differential equations and semi-variational approaches. These methods tend to make the analysis of arbitrary cross-sections very complicated, as they involve satisfaction of difficult equilibrium and compatibility conditions along the junctions of the plates. Although the finite element approach provides a general framework for dealing with this problem, the large number of degrees of freedom required to analyse a plate assembly has limited its application for analysing the post-local buckling of structural sections.

The earliest solution for the postbuckling behaviour of compressed thin-walled members, consisting of flat plate elements, was obtained by Bentham (1959). Bentham analysed both compressed long steel tubes and channel sections, and obtained the ratio of the post-buckled stiffness to the pre-buckled stiffness.
An elastic and plastic postbuckling analysis of thin-walled rectangular columns was presented by Graves Smith (1968) as part of his studies of the ultimate strength of columns. Initial postbuckling stiffnesses were obtained for rectangular columns of various ratios of length to width of the cross-section. Since the deflection shape was not allowed to change in the postbuckling range, it was not possible to obtain the reduction in the stiffness which occurs in the post-locally buckled range of structural response at loads well above the critical load. In a subsequent paper, Graves Smith (1972) considered the post-local buckling behaviour of a square box beam subjected to end moments using a similar approach. Another significant contribution on the subject of post-local buckling behaviour of plate assemblies was made by Rhodes and Harvey (1970, 1976) who studied the problem of plain and lipped channels subject to eccentric compression, using a semi-energy method. Their theoretical results showed good agreement with experimental results published earlier by Winter (1968).

Desmond, Pekoz and Winter (1981) studied the postbuckling behaviour of thin-walled members with edge stiffeners. They employed a geometrically nonlinear finite element analysis to assess the performance of edge stiffeners in lipped channel members, and presented experimental results. Usami (1983) investigated the behaviour of eccentrically loaded thin-walled box sections in the post-locally buckled range. In his analysis, Usami proposed a design algorithm to calculate the post-local buckling response of box sections which was based on his effective width formula.

The finite strip method has been found to be a useful tool for analysing the postbuckling behaviour of plate assemblies. The method has been used by Graves Smith and Sridharan (1978b) to study the elastic postbuckling response of channels under uniform compression, while Hancock (1981) used the method to study thin-walled I-section columns. Based on the postbuckling theory of Bradford and Hancock (1984), Bradford (1985) studied the postbuckling of box beams whilst Hancock (1985) studied the postbuckling of I-beams in uniform bending. This analytical method forms the basis of the postbuckling results of trough girders presented herein.

In this paper, the finite strip method presented by Bradford and Hancock (1984) is used to study the linear bifurcation and nonlinear post-local buckling response of trough girders. Design charts for elastic local buckling are given, and the strengthening effect of the provision of a longitudinal stiffener is considered. The response of trough girders after the onset of local buckling is considered, and the postbuckled stiffnesses, deformations and stresses are examined.
2. **FINITE STRIP THEORY**

A geometrically imperfect finite strip method of analysis for determining the elastic nonlinear behaviour of plates and sections beyond the local buckling load has been described (Bradford and Hancock 1984). The basic steps in this nonlinear finite strip postbuckling analysis involve:

a. a definition of the displacement functions used to describe the plate membrane and flexural deformations;

b. selection of nonlinear strain-displacement relations suitable for the post-local buckling analysis;

c. selection of an appropriate plate theory to determine the membrane and flexural behaviour of the plate strips;

d. application of the principle of virtual displacements to determine the nonlinear stiffness equations, and

e. solution of the nonlinear stiffness equations by the Newton-Raphson procedure for a plate assembly with initial imperfections and subject to increasing compressive strain.

The previous steps have been set out by Bradford and Hancock (1984). The nonlinear stiffness equations for the complete fabricated plate member can then be obtained using an axis transformation, together with a simple consideration of equilibrium and compatibility along the nodal lines, as described by Cheung (1976). These can be written as

\[
([K_0]+[G(\varepsilon_H)]+[K_1(\Delta)]+[K_2(\Delta^2)]-[K_2(\Delta^2)]) \{ \Delta \} \\
= \{ W(\varepsilon_H) \} +([K_0]+\frac{1}{2}[K_1(\Delta_0)]) \{ \Delta_0 \}
\]

(1)

where \([K_0]\) is the linear component of the stiffness matrix of the fabricated plate assembly; \([G(\varepsilon_H)]\) is the stability matrix which is a function of the longitudinal bending strains \(\{ \varepsilon_H \}\); \([K_1(\Delta)]\) is the nonlinear component of the stiffness matrix which is a linear function of the nodal line displacements \(\{ \Delta \}\); \([K_2(\Delta^2)]\) is the nonlinear component of the stiffness matrix which is a quadratic function of the nodal line displacements; \(W(\varepsilon_H)\) is the load vector resulting from the longitudinal strains; and \(\{ \Delta_0 \}\) is the initial value of the nodal line displacements.

The set of nonlinear stiffness relations represented by Eq. 1 can be solved for \(\{ \Delta \}\) at any value of the strain vector \(\{ \varepsilon_H \}\) using a tangent stiffness approach. The nature of Eq. 1 requires that the vector \(\{ \Delta_0 \}\) must be nonzero if there is to be a solution at values of the
longitudinal strains less than the vector of strains for local buckling \( \{ \varepsilon_{cr} \} \). Equations 1 are solved by the simple Newton-Raphson scheme presented by Gallagher et al. (1971).

Bifurcation due to local buckling can be obtained from Eq. 1 as (Bradford 1983)

\[
| [K_0] + [G(e_H)] | = 0
\]  

(2)

The matrices \([K_0]\) and \([G]\) are identical to those presented by Hancock (1978) in his study of local, distortional, and lateral buckling of I-section beams in uniform bending.

3. **VERIFICATION OF THEORY**

3.1 *Local Buckling*

The accuracy of the finite strip analysis in predicting the local buckling load of eccentrically loaded channel struts has been assessed by comparing the buckling loads with experimental values given by Rhodes and Harvey (1975). Because the channels were loaded eccentrically, the struts were subjected to axial and bending stresses. The cross-sectional dimensions were chosen so that buckling occurred in the purely local mode. The length of the struts (610 mm) was long enough to allow several buckling half-wavelengths to be obtained, but short enough to preclude any interaction of local buckling with overall buckling.

Figure 2 shows a comparison of theoretical and experimental buckling loads for lipped channels with loads applied at various eccentricities. It can be seen that the agreement between theory and experiment is good, although the experimental loads fall slightly below the theoretical curve for larger values of the ratio \(d/bf\). It was reported by Rhodes and Harvey that the experimental loads were not clearly defined, since the combination of axial and bending stresses caused the initial imperfections to grow large enough to mask the local buckling load.

The theoretical and experimental results for a plain channel strut are shown in Fig. 3 for a condition of "constant load eccentricity" (Bradford 1983). The results are seen to be in fairly good agreement. The comparisons in Figs. 2 and 3 illustrate that Eq. 2 leads to reliable results for local buckling of sections under bending actions.

3.2 *Post-local Buckling*

The postbuckling theory represented by Eq. 1 was used to compare the deflections of a flat rectangular plate subjected to a linearly varying end compressive load action with the experimental results reported by Walker (1967) for this problem. The experimental results are compared in Fig. 4 with the theoretical solutions corresponding to a "constant load
eccentricity". While the agreement between theory and experiment is good, any disparity between the two is probably a result of adopting a slightly conservative estimate of the initial plate imperfections.

The nonlinear finite strip analysis was also used to derive the moment-curvature relationship for a square box section in uniform bending in order to compare the results with those of an elasto-plastic analysis presented by Graves Smith (1972). The results are shown in Fig. 5 for a maximum initial plate imperfection of 0.1 times the thickness of the box with the same shape as the initial buckling mode. It can be seen from this figure that the postbuckled stiffness of the box beam compares well with Graves Smith’s elastic results, with a postbuckled stiffness of approximately 0.745 times the initial stiffness.

4. LOCAL BUCKLING BEHAVIOUR

The local buckling behaviour of the trough girder in uniform bending with the stress distribution shown in Fig. 1 was analysed using the bifurcation representation of Eq. 2. The compressive critical stress in the top flanges $\sigma_{ol}$ was expressed as a function of the web local buckling coefficient $k_w$ (Bulson 1970) as

$$\sigma_{ol} = k_w \frac{\pi^2 E}{12(1-\nu^2)} \left( \frac{t}{d} \right)^2$$

(3)

All of the studies fixed the web slenderness $d/t$ at 200. This was found not to influence the solutions for $k_w$. The value of the Young’s modulus $E$ and the Poisson’s ratio $\nu$ were taken as 200000 MPa and 0.3 respectively.

In order to obtain the minimum value of $k_w$, a plot of the elastic critical stress $\sigma_{ol}$ against the buckling half-wavelength was made. This plot exhibited the familiar garland shaped curve. A quadratic interpolating function was fitted through three computed points close to the local nadir, and the minimum of this interpolating function was used to calculate $k_w$. The finite strip subdivision had two strips in each top flange, six strips in the webs and two strips in the bottom (tension) flange.

Figure 6 shows the effect of varying the trough width $b$ on the web local buckling coefficient $k_w$. The width $b_F$ and thickness $T_F$ of the top flange were fixed, and the thickness of the bottom (tension) flange was varied. It can be seen that for increasing bottom flange thicknesses that the web buckling coefficient decreases. This occurs because the neutral axis is lowered as $T/t$ increases, so that more of the web is placed into compression. Being in tension, the bottom flange provides near complete restraint against local buckling regardless of its thickness. As $b/d$ increases, the neutral axis is also lowered, resulting in a reduction in the local buckling coefficient $k_w$. 
The effect of top flange width and thickness on the local buckling coefficient is shown in Fig 7 for a trough girder with \( b/d = 0.5 \). For a given top flange width \( b_F \) and bottom flange thickness \( T \), increasing the top flange thickness significantly enhances the local buckling coefficient \( k_w \). For very narrow values of the top flange width \( b_F \), the top flange was not stiff enough to restrain the flange-web junction from deflecting laterally. Because of this, the line junction was not straight, so that the buckling mode was not local and no minimum could be found. This is akin to the stiffener buckling of unrestrained channel sections reported by Bradford and Trahair (1982).

Based on the results presented in the previous figures and on other studies made but not reported herein, an approximate expression for the local buckling coefficient \( k_w \) was sought for use in design. A generally conservative approximation is given by

\[
k_w = 10(T_F/r)(1.5-b/d)[1-30(b_F/d-0.15)^2]
\]

for \( b/d \leq 1 \) and \( 0.05 \leq b_F/d \leq 0.15 \). For clarity, Eq. 4 is not shown on the graphs, but it may be used as an alternative to Figs. 6 and 7 with interpolation.

The local buckling modes of the trough girders studied above were found to be characterised by flexure of the web in the compressive zone, so it was decided to use the finite strip method to investigate the effects of placing a horizontal web stiffener on the web so as to minimise this flexure. The results are shown in Fig. 8, where the web buckling coefficient \( k_w \) is plotted against the position of the web stiffener for various ratios of the second moment of area of the top flange \( I_F \) to that of the stiffener \( I_s \), where \( I_s \) is taken about the mid-depth of the stiffener. The horizontal lines for \( I_F/I_s=\infty \) represent the buckling coefficient for no web stiffener. For a very stiff stiffener \( (I_F/I_s = 40) \), the maximum local buckling coefficient \( k_w \) is increased by about 65 percent. This was found to be typical of other stiffened trough geometries studied. It may also be observed that the web stiffener enhanced the value of \( k_w \) greatest when it was placed about fifteen percent of the web depth below the top flange \( (2d_s/d = 0.3) \). This is in reasonable agreement with the results of Bulson (1970) for isolated plates in bending. When the web stiffener is placed below the web mid-height \( (2d_s/d > 1.0) \), it can be seen that its effect on \( k_w \) is negligible.

When the horizontal stiffener is placed at a distance \( d_s/d = 0.15 \) from the top flange, the local buckling coefficient \( k_w \) in Eq. 4 may be increased to \( k'_w \) by

\[
k'_w = \left[ 1 + \frac{4}{\sqrt{I_F/I_s}} \right] k_w
\]

for \( I_F/I_s \geq 20 \). Although for clarity these values are not shown in Fig. 7, they were found to be a good approximation to the maxima of the curves in this figure for varying values of \( I_F/I_s \).
5. POST-LOCAL BUCKLING BEHAVIOUR

The nonlinear finite strip analysis has been used to study the post-local buckling of a trough girder of typical dimensions in bending. The geometry of the beam is shown in Fig. 9. The initial imperfection vector \( \{ \Delta_0 \} \) in Eq. 1 was chosen to be of the same shape as the local buckling eigenmode, with a maximum imperfection of 0.2 times the web thickness \( t \). Geometric imperfections of this type produced by the fabrication process result in the largest deformations \( \{ \Delta \} \) in the nonlinear analysis. The same finite strip subdivision that was used for the previous bifurcation study was employed for the postbuckling analysis.

A set of bending strains \( \varepsilon \) were applied at each node corresponding to a curvature \( \rho \). Because of the stress redistribution in the nonlinear range of structural response, an axial force was found to be present in the trough section as the curvature was increased monotonically, and Eq. 1 was solved. A constant strain \( \varepsilon_0 \) had to be applied at each node in order that the axial force vanished and a condition of pure bending was maintained. This loading condition is termed "constant load eccentricity" (Bradford 1983) and results in a shifting of the neutral axis similar to that experienced by a beam with residual stresses in bending. The axial strain \( \varepsilon_0 \) was determined by trial.

Figure 9 shows the moment-curvature relationship for the trough girder studied, non-dimensionalised with respect to the local buckling curvature \( \rho_{0l} \) and elastic critical moment \( M_{0l} \). Also shown in this figure is the relationship for a box beam determined by Graves Smith (1972), which has a postbuckled to prebuckled stiffness ratio of 0.745. It can be seen from Fig. 9 that there is very little reduction in stiffness in the postbuckled range for the trough girder considered. This can be explained in terms of Fig. 10, which shows the distribution of postbuckled stresses in the nonlinear domain. Because the top flange is stocky \((b_F/T_F = 10)\), the postbuckling effects in it are minimal, and the reduction in stresses due to postbuckling are greatest close to the centroid of the section. When these reduced stresses are integrated to calculate the moment \( M \), their lever arms are small so that the reduction in the moment below that for a fully effective section is small. For the section considered, the postbuckled stiffness was 90 percent of the prebuckled stiffness.

While the reduction in flexural rigidity after the onset of local buckling is only slight, the deflections \( \{ \Delta \} \) shown in Fig. 11 for the beam studied in Figs. 9 and 10 indicate that the deformations can be reasonably large in the postbuckled range. Since the flange is stocky, the deformations are generally confined to flexure of the web, with these deformations being largest in the region of the web where the reductions in stress below those assuming a fully effective section are greatest.
6. CONCLUSIONS

A brief historical survey was made of analyses of local buckling and post-local buckling with particular reference to plate assemblages in bending. The nonlinear stiffness equations that represent local behaviour were given, the derivation of which is given elsewhere (Bradford and Hancock 1984). The equations predict results that agree well with experiments and other independent analyses.

The initial local buckling of an isolated trough girder, such as that used in a composite bridge with a concrete deck, was studied. Graphs were presented of the local buckling coefficient as a function of the trough geometry. Based on these studies, an approximate expression for the local buckling coefficient was given. It was shown that local buckling could be delayed significantly by the provision of a longitudinal stiffener about fifteen percent of the web depth from the compression flange.

The local nonlinear behaviour of a trough girder was studied. For the girder of typical proportions considered, there is little redistribution of stress in the flange in the postbuckling domain. Because of this, the effective flexural rigidity after local buckling is reduced by only ten percent from that prior to local buckling. This enables approximate deflection calculations to be made up to first yield. It was shown, however, that deflections in the web are quite substantial in the post-local buckling range of structural response.

7. ACKNOWLEDGEMENT

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APPENDIX I - REFERENCES


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APPENDIX II - NOTATION

The dimensions of the trough girder are shown in Fig. 1. Other principal notation is as below.

$b_f, d$ dimensions of lipped channel in Fig. 2;

$E$ Young’s modulus of elasticity;

$[G]$ stability matrix;

$I_F$ second moment of area of top flange;

$I_s$ second moment of area of stiffener;

$[K_0],[K_1]$ linear stiffness matrices;

$[K_2]$ nonlinear stiffness matrix;

$k_w$ web local buckling coefficient;

$k'_w$ web local buckling coefficient with horizontal stiffener;

$M$ bending moment;

$M_{ol}$ bending moment at local buckling;

$v$ displacement in Fig. 4;

$\{W\}$ load vector;

$\{\Delta\}$ vector of displacements;

$\{\Delta_0\}$ vector of initial displacements;

$\varepsilon_{cr}$ local buckling strain;

$\{\varepsilon_{II}\}$ longitudinal bending strains;

$\nu$ Poisson’s ratio;
\( \rho \) curvature;

\( \rho_{ol} \) curvature at local buckling;

\( \sigma \) stress;

\( \sigma_{ol} \) stress at local buckling.
FIGURE 1  TRough GIRDER

Dimensions

Stresses

Dimensions

Stresses
FIGURE 2  LOCAL BUCKLING COMPARISON OF LIPPED CHANNEL STRUT

- Finite strip solution
- Tests

- $h = 203$ mm
- $b_f = 102$ mm
- $d_s = 38$ mm
- $t = 1.22$ mm
- $\nu = 0.3$
- $D = \frac{Et^3}{12(1-\nu^2)}$
Figure 3: Local Buckling Comparison of Plain Channel Strut

- $h = 203$ mm
- $t = 1.22$ mm
- $\nu = 0.3$
- $D = \frac{Et^3}{12 (1-\nu^2)}$
\[ \varepsilon_{cr} = 5.65 \times 10^{-4} \]

Finite strip solution

- Tests

\[ \frac{Pb}{\pi^2 D} \]

Dimensionless load

- \( a = 508 \text{ mm} \)
- \( b = 254 \text{ mm} \)
- \( t = 1.7 \text{ mm} \)

- \( E = 227 \text{ 000 MPa} \)
- \( \nu = 0.3 \)
- \( D = \frac{Et^3}{12 (1-\nu^2)} \)

Maximum displacement

\( v/t \)

**Figure 4** Load-Deflection Comparison for a Simply Supported Plate
Figure 5: Moment-Curvature Comparison for Box Beam

Graves Smith (1972)

Finite strip solution

Initial local imperfection = 0.1t

Dimensionless moment $M/M_{0l}$

Dimensionless curvature $\rho/\rho_{0l}$
Figure 6: Effect of Trough Width on Local Buckling Coefficient
FIGURE 7  EFFECT OF TOP FLANGE WIDTH AND THICKNESS ON LOCAL BUCKLING COEFFICIENT
FIGURE 9  MOMENT-CURVATURE RELATIONSHIP

This study

Graves Smith (1972)

Full stiffness

\[ \frac{b_F}{d} = 0.1 \quad T/t = 2 \]

\[ b/d = 0.5 \quad d/t = 200 \]

\[ T_F/t = 2 \quad \nu = 0.3 \]
FIGURE 10 STRESS DISTRIBUTION
FIGURE 11  POSTBUCKLING DEFORMATIONS