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LOCAL BUCKLING OF WELDED TUBULAR COLUMNS

by

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Alexis Ostapenko*

1. INTRODUCTION

The need for new research on local buckling of tubular columns fabricated by cold-rolling and welding arose from the fact that, as shown in Fig. 1, the few tests conducted on such columns gave results with very discouraging scatter and the design methods based on these test results also vary quite widely (1,2,3). For example, there is a considerable discrepancy between the DNV and the API curves in Fig. 1. Furthermore, all the tests shown in this figure were conducted on columns made from steel with a nominal yield stress of about 240 MPa (36 ksi) which is substantially weaker than the high-strength steels often preferred in offshore construction. This raises the question of reliability of the design curves shown when they are used in designing high-strength steel columns.

Described here are the local buckling tests conducted at Lehigh University and the conclusions and recommendations drawn from the obtained results.

2. LEHIGH UNIVERSITY TESTS

Ten axial load tests were carried out on short column specimens with the diameter varying from 0.7 m (28 in.) to 1.8 m (70 in.) and the nominal wall thickness being 6.5 to 8.3 mm (1/4 to 5/16 in.). The nominal yield stress was 345 MPa (50 ksi) for nine specimens and 690 MPa (100 ksi) for the tenth. The D/t ratio varied from 85 to 248 and parameter α from 1.4 to 7.5 (5,6,7).

\[ \alpha = \frac{F_y}{F} \cdot \frac{1}{D/t} \]  

where \( F_y \) is the yield stress.

A typical test specimen had the relative proportions as shown in Fig. 2. The length was 1.6 to 3 times the diameter and thus the effect of overall column buckling was completely excluded.

The longitudinal gage lines in the middle portion of the specimen in Fig. 2 were used for measuring the longitudinal residual stresses caused by welding. The measurements were made on both the outside and inside surfaces, and a typical distribution of the average residual stresses around the circumference is shown in Fig. 3 on the unfolded surface of a test specimen. The vertical line in the middle represents the weld seam and the horizontal coordinate indicates the circumferential distance from the weld in the right and left directions toward a line located diametrically opposite the weld. The most significant are the narrow high tension band at the weld seam and the 0.2-0.3 m wide compression bands adjoining it. The compressive stress was from about 60 to 110 MPa in different specimens, but no methodical relationship could be detected between the residual stress level and the other specimen parameters.

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The specimen ends were either machined flat and square for the smaller specimens (D = 0.7 and 1.2 m) or had plate bearing rings welded for the larger specimens (D = 1.5 and 1.8 m). During the test, the specimens were compressed flat ended between the machine head and the machine floor or the base pedestal as shown in Fig. 4. The buckles developed either in the middle portion, as shown in the figure, or, more frequently, near one of the ends.

The specimens with D/t less than 200 had the buckling process initiated with a ring bulge which later progressed into lobular buckles, whereas the specimens with the larger D/t had lobular buckles forming from the start.

The behavior typical of five of the specimens, prior and shortly after buckling, is shown in Fig. 5 as the ratio of the average stress to the yield stress vs. the axial shortening. One specimen, P1, had considerable amount of yielding deformation before buckling, whereas all the others, after reaching the buckling load, rapidly lost their capacity although at different rates dependent on their D/t values. The remaining five specimens had peaked load-deformation diagrams.

As shown in Fig. 6 for two specimens, the tests were continued to explore the post-buckling behavior and the ability of such tubular members to dissipate energy. As the specimens deformed axially, there was some increase in the load and the first set of buckles was followed by the second set. In most cases, a test was not stopped until fracture cracks appeared in the specimen wall, for example, as shown in Fig. 7.

The post-buckling load level was from 12 to 30 percent of the yield load, and approximately proportional to the D/t ratio of the specimens.

In addition to the eight "regular" 345 MPa (50 ksi) specimens described above, two special ones were tested (P8, P9). See Fig. 8. Both were 1.53 m (60 in.) in diameter, but one (P8), although made of 345 MPa steel, had a transverse weld, and the second (P9) was made of the nominally 690 MPa (100 ksi) steel. The purpose was to explore the effects of a transverse weld and of a yield stress twice as great as previously used in test specimens on local buckling.

In the first specimen (P8) the buckles initially developed near the transverse weld and at the bottom end. As the test progressed, the buckles joined together and the specimen deformed into a "stepped-on beer can" as shown in Fig. 9.

The second specimen (P9) buckled at the top into a circumferential string of lobular buckles.

3. TEST RESULTS

Longitudinal residual stresses introduced by the welding process appeared to have no detectable effect on local buckling. In Fig. 10, the residual stress pattern is shown together with the wave pattern of the buckles. Neither the wave length nor the amplitude nor the location
of the buckles correlate with the residual stress pattern. The only more or less consistent observation was that a larger inward buckle developed at the weld in many of the specimens. However, this can be as readily ascribed to the local inward out-of-roundness existing at the weld.

All the Lehigh test results are plotted as solid dots and labeled P1 to P9 in Fig. 11 in addition to the test points and design curves of Fig. 1. The striking feature of these new points is that they lie above all other test points and form a quite smooth curve. Only one point, labeled P9, is an odd one -- it is for the 690 MPa specimen; the others are for the 345 MPa specimens. Going from right to left, that is from P1 to P2, etc., the points are for specimens with an increasing value of D/t, from 85 to 248.

The first two, P1 and P2, both reached the yield stress. The next two, P3 and P3A, are on top of each other. The difference between them was that P3 had both ends machined and P3A had one end machined and the other, where the buckling took place, welded to a thick plate. Since there is no real difference in the buckling load, it can be concluded that the end conditions had no apparent effect on the local buckling stress.

The next to the left, P4, had very substantial out-of-roundness (46 mm) and longitudinal imperfections. Yet it falls only slightly below a smooth imaginary curve that can be drawn through the other Lehigh points. Thus, one may conclude that geometric imperfections do not reduce the local buckling stress in this range as significantly as is generally believed (8).

Of the remaining points, P6 and P8 are significant since they differed only by P6 having a transverse weld in addition to the longitudinal weld. Yet, the buckling stresses essentially coincide thus indicating that a transverse weld may have only insignificant deleterious effect on the local buckling stress.

4. DESIGN RECOMMENDATIONS
Approximation of the DNV Procedure

As can be observed in Fig. 11, the 345 MPa Lehigh test points, P1 to P8, emphatically support the DNV design curve (4). The following simple formula was derived to approximate the rather complex DNV procedure within 1% for D/t less than 400 and within 1.3% for D/t less than 600:

\[
\frac{F_c}{F_y} = 1 - \frac{1}{3} \left(1.5 + 0.001(D/t)\right)^2
\]

where \( \alpha \) is defined by Eq. 1.

Local Buckling Formula for Tubular Columns Fabricated from 345 MPa (50 ksi) Steel

The good consistency of the Lehigh tests (P1 to P8) is contrasted by the considerable scatter of the other tests shown in Fig. 11.
One source of the scatter is the accuracy and the method of determining the yield stress. The static yield stress (zero strain rate) was used in interpreting the Lehigh tests. The dynamic yield stress values determined at unspecified strain rates and used in reporting other tests may have contributed to the scatter of up to about 8% (5). Since no data on the actual yield strain rates were available, this effect could not be taken into account in analyzing the test results.

Another effect not reflected in Fig. 11 is the interaction between local and overall column buckling. An adjustment for this effect was made by computing what should have been the local buckling stress to cause column buckling at the experimental stress value. The SSRC column buckling formula with the yield stress replaced by the local buckling stress was used (1,2). The resultant formula for the effective local buckling stress became

\[ F_c = \frac{2E}{(KL/\pi r)^2} \left[ 1 - \sqrt{1 - \frac{F_{\text{exp}}(KL/E)}{F_{\text{exp}}}} \right] \]  

where, besides standard notation,

- \( K \) = effective column length factor
- \( F_{\text{exp}} \) = experimental buckling stress.

It was also found that the popular nondimensional parameter \( \alpha \) used as abscissa was not the best for plotting the test results. After some trials, parameter \( c \) (Eq. 4) was selected, and the test data, incorporating the correction by Eq. 3, was replotted as shown in Fig. 12.

\[ c = \sqrt{\frac{E}{F_y}} \cdot \frac{1}{D/t} \]  

The scatter is significantly narrowed down and a smooth curve can be passed through the principal band of the points shown in Fig. 12. A cubic least-squares fit for this curve gives Eq. 5.

\[ \frac{F_c}{F_y} = 38c - 480c^2 + 2000c^3 \]  

for \( c < 0.07 \)

and

\[ \frac{F_c}{F_y} = 1.0 \]  

for \( c \geq 0.07 \)

Since the point for specimen P9 with \( F_c = 690 \text{ MPa} \) (labeled 90 ksi in Fig. 12) and a number of points for specimens with \( F_c = 240 \text{ MPa} \) (36 ksi) are some distance from this curve, Eq. 5 should be limited to tubular columns made of steel with the yield stress approximately equal to 345 MPa (50 ksi).
5. SUMMARY AND CONCLUSIONS

Ten axial load tests were conducted on short cold-rolled and then welded tubular columns to investigate local buckling in the stress range between the proportional limit and the yield stress. Nine specimens were made of steel with $F = 345$ MPa (50 ksi) and one of steel with $F = 690$ MPa (100 ksi). Analysis of the obtained test results and a comparison with the results of other tests and with the current design rules lead to the following conclusions:

1) The effect of geometric imperfections on local buckling is much less pronounced than generally assumed.

2) One test indicated that a transverse weld has no noticeable detrimental effect on local buckling.

3) A test on a specimen made of $F = 690$ MPa (100 ksi) steel shows that the yield stress may not be nondimensionalized by the popular $\alpha$-parameter and more well controlled tests are needed on 690 and 240 MPa (100 and 36 ksi) specimens in order to develop a comprehensive design approach.

4) A formula (Eq. 5) is recommended for computing the local buckling stress of fabricated tubular columns made from 345 MPa (50 ksi) steel.

6. ACKNOWLEDGMENTS

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Appendix I - REFERENCES


Appendix II - NOTATION

\[
c = \sqrt[3]{\frac{E}{F_y}} (t/D) = \text{nondimensional wall slenderness parameter};
\]
\[
D = \text{mean diameter of tubular column};
\]
\[
E = \text{modulus of elasticity};
\]
\[
F_c = \text{local buckling stress};
\]
\[
F_{exp} = \text{experimental column buckling stress affected by local buckling};
\]
\[
F_y = \text{yield stress};
\]
\[
K = \text{effective column length factor};
\]
\[
L = \text{column length};
\]
\[
P = \text{axial load};
\]
\[
r = \text{radius of gyration};
\]
\[
t = \text{wall thickness of tubular column};
\]
\[
\alpha = \frac{E}{F_y} (t/D) = \text{nondimensional wall slenderness parameter};
\]
\[
\Delta = \text{axial shortening of tubular column}.
\]

ABSTRACT

Tests were conducted on short high-strength tubular columns fabricated by cold-rolling and welding. The results indicated an apparently negligible effect of initial imperfections and of a transverse weld on local buckling above the proportional limit. A formula was proposed for computing the local buckling stress of tubular columns made of steel with the yield stress of 345 MPa (50 ksi).
Fig. 1 Test Results and Present Design Curves for Local Buckling of Tubular Columns
Fig. 2 Test Specimen with Gage Lines for Measuring Residual Stresses

Fig. 3 Longitudinal Residual Stresses due to Welding
Fig. 4 Test Specimen in Testing Machine after Initial Buckling

Fig. 7 Fracture of Specimen Wall
Fig. 5 Buckling Behavior of Five Test Specimens
Fig. 6 Post-Buckling Behavior of Two Test Specimens
Fig. 8 Test Specimens P8 and P9
Fig. 9 Specimen P8 after Testing
Developed Plate Width

\[ \approx F_y \text{ (Tension)} \]

30 MPa (4.4 ksi)

Weld Seam

2m (78.74 in.)

WELDING RESIDUAL STRESS PATTERN

-90

120 MPa (-17.4 ksi)

PATTERN OF LOBULAR BUCKLES

Outward Displ.

Inward Displ.

Fig. 10 Residual Stresses and Pattern of Buckles in 1.78 Diameter Specimen (P5)
Fig. 11  Lehigh Tests in Comparison with other Tests and Design Curves
Fig. 12 New Plot of Test Results and Proposed Design Curve