Proceedings of the ASME 2011 30th International Conference on Ocean, Offshore and Arctic Engineering
OMAE2011
June 19-24, 2011, Rotterdam, The Netherlands

OMAE2011-49601

BUCKLING ANALYSIS OF OFFSHORE JACKETS IN REMOVAL OPERATIONS

Xia Wenjing
Structural Engineer

P.C.J. Hoogenboom
Assistant Professor

Delft University of Technology
Faculty of Civil Engineering and Geosciences
Delft, the Netherlands

ABSTRACT

In analysis of the removal of offshore jackets an important failure mode is buckling. In current practice, a buckling check involves manual determination of the buckling lengths of each frame member. It is estimated that 5 to 10% of the man-hours in structural analysis of removal projects is spend on checking and correcting buckling lengths. Fortunately, an alternative method is available that does not require determining buckling lengths. In this paper it is shown how this method can be derived from the NORSOK standard for tubular steel frame structures. The method is demonstrated in a removal analysis of an offshore jacket. It is concluded that this method can be successfully applied.

1 INTRODUCTION

Members of frame structures have many imperfections, such as a slight curvature or twist, welding stresses, rolling stresses, eccentric joints, eccentric loading and deviations in cross-section dimensions. All these imperfections make that members and structures buckle at smaller loads than predicted by models that do not include these imperfections. This has been studied in experimental programs in Europe and the USA which led to the well known buckling curves in the Eurocode and the AISC LRFD (Fig. 1 and 2) [1].

When applying the buckling curves the buckling lengths or effective length factors need to be determined. The codes provide comprehensive rules for determining the buckling lengths, however, not all situations are properly covered. As an example, Figure 3a shows part of a jacket leg that is modelled with 5 elements of 2 m length each. The subdivision into 5 elements is necessary to model the presence of a light tubular member parallel to the leg. One of the leg elements is checked for buckling. According to the code equations the adjacent members are modelled as rotational springs. The resulting buckling length is approximately 4 m. However, engineering judgment shows that the real buckling length is approximately 8 m (Fig. 3b). The latter buckling length leads to a much smaller buckling load. Therefore, blindly applying the code rules on buckling lengths – as a computer program does – can lead to a considerable overestimation of the buckling load.

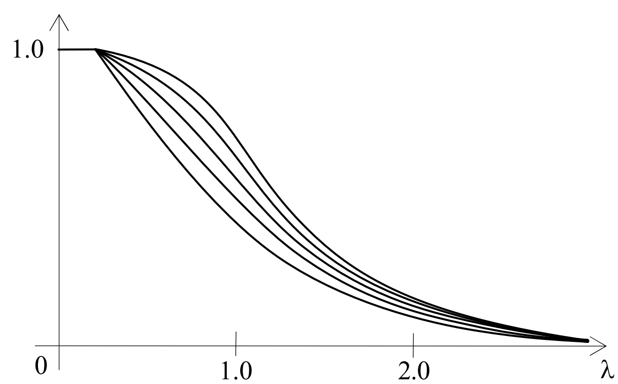

Figure 1. Buckling curves according to the Eurocode

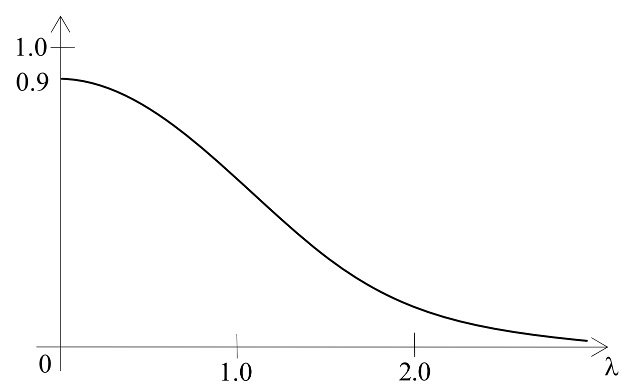

Figure 2. Buckling curve according to the AISC LRFD

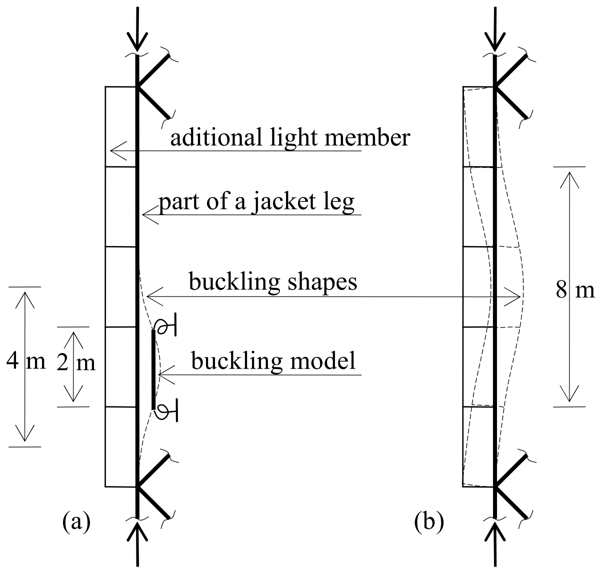

Figure 3. Automatically calculated buckling length (a) and the real buckling length (b)

Another example is the cross brace shown in Figure 4. One diagonal member is compressed and the other is tensioned. The compressed member can buckle in the out-of-plane direction. The buckling length thus calculated is approximately 14 m. However, the tensioned diagonal member will restrain this buckling deformation. The real buckling length is approximately 7 m. The latter buckling length leads to a much larger buckling load, which can make the difference between a complicated lifting operation and a simple one.[1]

Consequently, many member buckling length needs to be checked manually. In a typical jacket removal project approximately 2000 man-hours are spend on structural analysis. It is estimated that 5 to 10% of these man-hours are used for checking and correcting buckling lengths. This includes reruns made due to forgetting to adjust bucking lengths when design changes are processed.

An alternative analysis method is available that does not require manual determination of buckling lengths. It has been developed by W.F. Chen and co-workers for the AISC LRFD code [2, 3]. In this method the stability checks are included in a geometrical nonlinear frame analysis. The buckling lengths are determined computationally instead of by using code equations. This has the advantage that it is very accurate in any situation and buckling lengths do not need to be checked by hand. In this paper it is shown that also the NORSOK buckling requirements can be rewritten to obtain a straight forward implementation for a nonlinear analysis.

[1] The requirement that none of the members should buckle during lifting and transportation of a jacket is rather conservative. After all, in a statically indeterminate structure the buckling of a member does not need to cause collapse of the structure. A geometrical nonlinear analysis can show this, however, this issue is not considered in this paper.

For many frame structures buckling is not the decisive failure mechanism. For example, in most unbraced building structures the second order displacements under serviceability loading are decisive. In many offshore structures fatigue is the decisive failure mechanism. On the other hand, buckling is very likely to be decisive when a slender frame structure has no serviceability limit state and no fatigue restrictions. Examples are lifting and transportation of an offshore jacket in a removal operation.

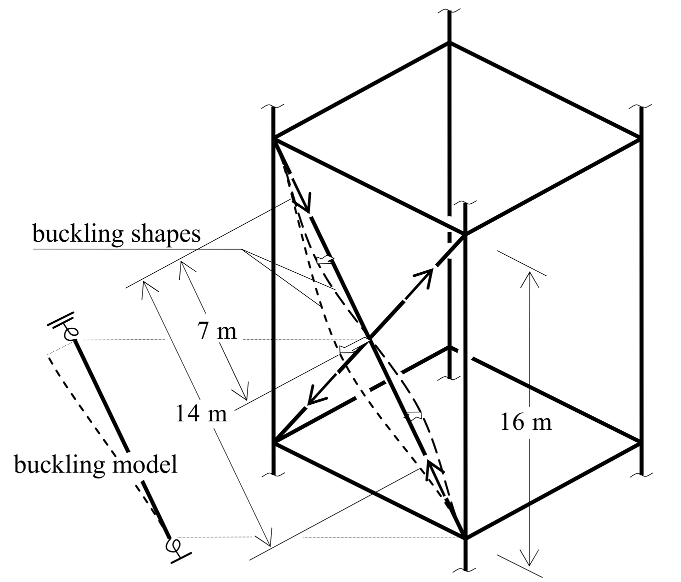

Figure 4. Out-of-plane buckling lengths of a cross brace

2 DERIVATION

In the NORSOK standard N-004, Eq. 6.5 the slenderness of tubular members is defined as

$$\bar{\lambda} = \sqrt{\frac{f_{cl}}{f_E}} \tag{1}$$

where f_{cl} is the stress at which the tube wall yields or buckles and f_E is the theoretical mean stress at which the whole tubular member buckles if there would not be any imperfection. Stress f_{cl} can be determined from Eq. 6.6 to 6.8 in the NORSOK standard (See appendix). In hand calculations f_E is determined using the buckling length or the effective length factor, which strongly relies on sound engineering judgment. In computer calculations f_E is determined just as it is defined. For this a geometrical nonlinear computation is performed of the whole frame structure.

The effect of member imperfections is described by NORSOK equations 6.3 and 6.4.

$$f_c = [1.0 - 0.28\bar{\lambda}^2] f_y \quad \text{for} \quad \bar{\lambda} \le 1.34 \tag{2}$$

$$f_c = \frac{0.9}{\bar{\lambda}^2} f_y \quad \text{for} \quad \bar{\lambda} > 1.34 \tag{3}$$

where f_y is the material yield stress and f_c is the mean stress at which a tubular member buckles.

If member imperfections are included in a nonlinear computation a member buckles at a stress of f_c instead of f_E. The theoretical buckling stress f_E is linear in the bending stiffness, therefore, a computer can include imperfections by reducing the stiffness by a factor

$$\xi = f_c / f_E. \tag{4}$$

Note that f_c can be computed from f_E using Eq. (1) to (3). Unfortunately, f_E is not known before the nonlinear computation is finished. At first sight this seems an insurmountable problem but the solution is quite simple. During a nonlinear analysis the load is applied in increments and equilibrium is found in iterations. In each increment and iteration the stiffness reduction factor ξ is applied. In computing ξ, instead of f_E the actual member stress f over ξ is used. This is clearly incorrect, however, when the loading is such that a member is buckling, $f = f_c$ and $f / \xi = f_E$. Therefore, at the moment of buckling the correct reduction factor is applied, which takes imperfections correctly into account.

Substiting Eq. (1) (2) and (3) in (4), applying a material factor γ_M to f_c (NORSOK Eq. 6.2) and replacing f_E by f / ξ gives

$$\begin{aligned} \xi &= (1.0 - \frac{f}{f_y}) \frac{f}{0.28 \gamma_M f_{cl}} && \text{for} \quad f \ge 0.5 f_y \\ \xi &= \frac{0.9}{\gamma_M} \frac{f_y}{f_{cl}} && \text{for} \quad f < 0.5 f_y \end{aligned} \tag{5}$$

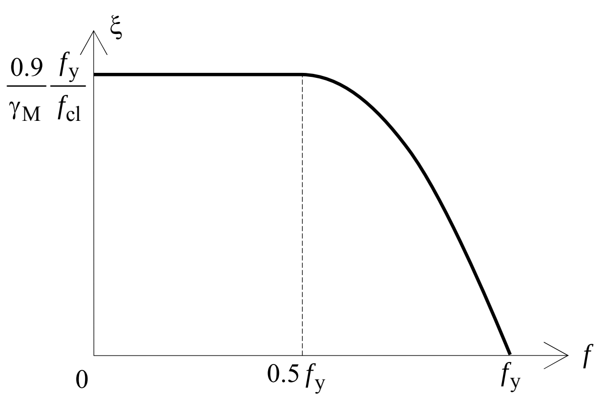

Figure 5. Bending stiffness reduction factor ξ as a function of the member stress f

where f is computed as the member axial force N over the member cross-section area A (Fig. 5).

$$f = -N / A. \tag{6}$$

Another way of explaining Eq. (5) is by an experiment of thought. Suppose that a buckling test is performed on a simply supported column. The column length l and the buckling load N_c are measured. The bending stiffness EI can be calculated from $N_c = \pi^2 EI / l^2$. The test is repeated for many columns with the same cross-section but different lengths. Subsequently, a graph is made with N_c on the horizontal axis and EI on the vertical axis. Clearly, this graph shows the bending stiffness as a function of the buckling load. It also shows the bending stiffness as a function of any normal force N, because it does not matter that the bending stiffness has been obtained from observing buckling and the buckling equation. In this way Figure 5 would have been obtained too.

From the explanation above it follows that Eq. (5) can also be used to include imperfections in the computation of second order displacements. This has not been used in the paper at hand but in many situations it is a very useful property. Clearly, for the serviceability limit state $\gamma_M = 1$.

3 IMPLEMENTATION

A geometrical nonlinear computation can use either the secant algorithm or the Newton-Raphson algorithm (Fig. 6 and 7). In the secant algorithm the following member stiffness matrix is used [4].

$$\begin{bmatrix} F_1 \\ T_1 \\ F_2 \\ T_2 \end{bmatrix} = \frac{\xi EI}{l^3} \begin{bmatrix} 2(\beta+\gamma)-\varepsilon^2 & -(\beta+\gamma)l & 2(\beta+\gamma)+\varepsilon^2 & -(\beta+\gamma)l \\ & \gamma l^2 & (\beta+\gamma)l & \beta l^2 \\ & & 2(\beta+\gamma)-\varepsilon^2 & (\beta+\gamma)l \\ \text{sym.} & & & \gamma l^2 \end{bmatrix} \begin{bmatrix} w_1 \\ \phi_1 \\ w_2 \\ \phi_2 \end{bmatrix}$$

where EI is the member bending stiffness and l is the member length. The matrix depends on the member normal force N. For compression N is smaller than zero and the matrix elements are computed with

$$\varepsilon = \sqrt{\frac{-Nl^2}{\xi EI}} \tag{8}$$

$$\beta = \frac{\varepsilon(\varepsilon - \sin\varepsilon)}{2(1-\cos\varepsilon) - \varepsilon\sin\varepsilon} \tag{9}$$

$$\gamma = \frac{\varepsilon(\sin\varepsilon - \varepsilon\cos\varepsilon)}{2(1-\cos\varepsilon) - \varepsilon\sin\varepsilon} \tag{10}$$

For tension N is larger than zero and the matrix elements are computed with

$$\varepsilon = \sqrt{\frac{Nl^2}{EI}} \quad (11)$$

$$\beta = \frac{\varepsilon(\varepsilon - \sinh\varepsilon)}{-2(1-\cosh\varepsilon) - \varepsilon\sinh\varepsilon} \quad (12)$$

$$\gamma = \frac{\varepsilon(\sinh\varepsilon - \varepsilon\cosh\varepsilon)}{-2(1-\cosh\varepsilon) - \varepsilon\sinh\varepsilon} \quad (13)$$

Figure 8 shows the member node forces and node displacements.

Imperfections are included by multiplying the member stiffness matrix by ξ as shown in Eq. 7 and 8. Note that ξ has different values for different members.

In the Newton-Raphson algorithm the factor ξ needs to be applied to the member contribution of the internal force vector. It also needs to be applied to the member tangent stiffness matrices. However, in this paper the secant algorithm has been applied.

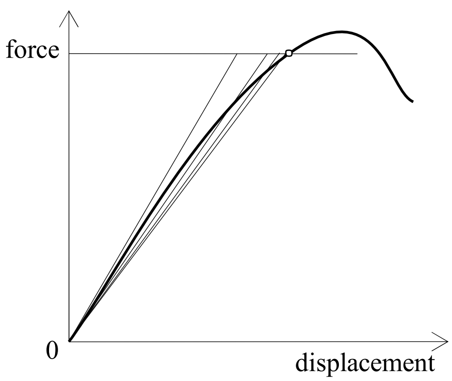

Figure 6. Equilibrium iterations of the secant algorithm

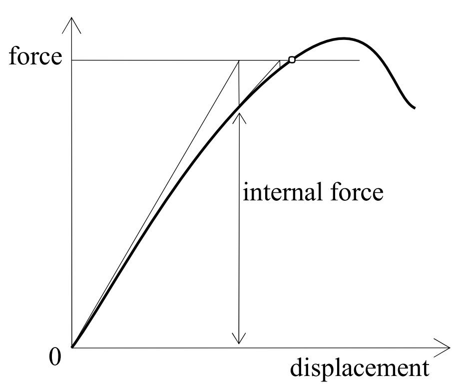

Figure 7. Equilibrium iterations of the Newton-Rahpson algorithm

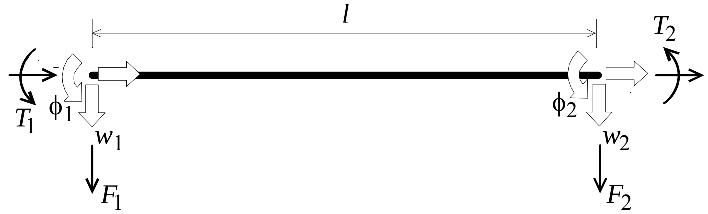

Figure 8. Frame member node forces, node moments, node displacements and node rotations

4 ANALYSIS EXAMPLES

Figure 9 shows a frame structure, which seems very usual at first sight. However, this example has an unusually large sensitivity to imperfections [5]. All members have a bending stiffness $EI = 127\times10^{12}$ Nmm^2. The response has been analyzed by a linear and a nonlinear frame program.

The horizontal displacement is recorded at joint D. The first order displacement is 11.3 mm. The second order displacement without imperfections is 11.4 mm, which is 1% larger. The second order displacement including imperfections (bending stiffness reduced by factor ξ) is 14.1 mm which is 25% larger than the first order displacement.

The elastic buckling load without imperfections is 75000 kN. The elastic buckling load including imperfections is 6450 kN, which is a factor 12 smaller. The second order displacements can also be obtained by the commonly used formula $u_2 = u_1\, n\, /\, (n - 1)$ where $n = N_c\, /\, N$. For performing this hand calculation it is important to estimate the buckling loads correctly. The plastic collapse load is just 1800 kN, therefore, buckling is not decisive for the ultimate limit state of this frame. This example shows the general importance of using a reduced bending stiffness in nonlinear structural analyses.

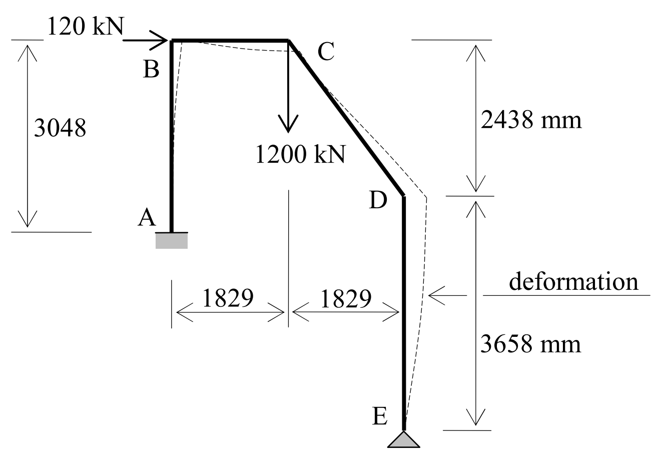

Figure 9. Frame structure with a remarkeble influence of the reduced bending stiffness

Lifting and transportation of a typical large jacket in a removal operation has been analyzed (Fig. 10). During lifting the jacket is loaded by self weight only. During transportation the jacket is loaded by self weight and inertia forces due to sea motions. A structural analysis program has been used to perform the linear analyses and the NORSOK buckling checks with manually updated buckling lengths. It is noted that these checks are a post processing operation to the linear analysis. It was shown that each member fulfils the NORSOK requirements.

A structural analysis program has been used for nonlinear analysis of the jacked loaded by one of the transportation loads. This loading has been selected because the linear analysis showed large compressive forces in members that are sensitive to buckling failure. The load has been applied in one increment and several iterations were necessary to find equilibrium. After every iteration the member stiffnesses were manually reduced by factor ξ to account for imperfections. None of the members buckled in the nonlinear analyses.

It is noted that software developers can simply implement the reduction factor ξ. This would make the duration of the nonlinear analysis just a few seconds and without any need for manual intervention.

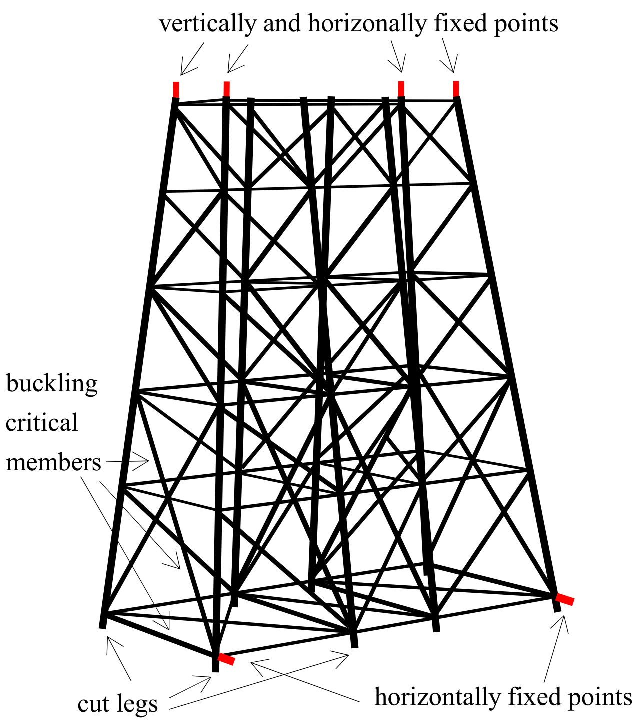

Figure 10. Transportation of a typical eigth leg jacket[2]

[2] The computation results are as expected, nevertheless, details of the structural model are not shown to respect confidentiality of the contractor, owner and software developer.

5 CONCLUSIONS

The considered alternative method can replace the traditional method of buckling checking. In the alternative method the NORSOK buckling requirements are correctly applied too. The important difference between the methods is that in the alternative method the buckling lengths are determined computationally without interpretations or approximations. The advantages and disadvantages of the alternative method compared to the traditional method are summarized below.

Advantages

- No manual calculation of buckling lengths needed
- According the NORSOK code of practice
- Easy to implement in frame analysis software
- Both member buckling and structural buckling included
- More accurate

Disadvantages

- Nonlinear analysis necessary
- Software developers need to implement Eq. (5)
- Some training of the analysis engineer is needed

REFERENCES

[1] European Convention for Constructional Steelwork, “Manual on Stability of Steel Structures”, June 1976.
[2] J.Y.R. Liew, “Advanced Analysis for Frame Design”, PhD dissertation, Purdue University, School of Civil Engineering, West Lafayette, 1992.
[3] W.F. Chen, Y. Goto en J.Y.R. Liew, “Stability Design of Semi-Rigid Frames”, John Wiley & Sons, 1996.
[4] J. Blaauwendraad, “Elastic Stability of Frame Structures”, Lecture book, Delft University of Technology, Aug. 1992.
[5] G. van der Helm, “Case studie niet-lineaire mechanica”, Cursus BV 2010/2011, Feb. 2011 (in Dutch).

APPENDIX

NORSOK Eq. 6.6 to 6.8 are related to this paper. For completeness they are provided in this appendix

$$f_{cl} = f_y \quad \text{for} \quad \frac{f_y}{f_{cle}} \leq 0.170 \qquad (6.6)$$

$$f_{cl} = (1.047 - 0.274\frac{f_y}{f_{cle}})f_y \quad \text{for} \quad 0.170 < \frac{f_y}{f_{cle}} \leq 1.911 \qquad (6.7)$$

$$f_{cl} = f_{cle} \quad \text{for} \quad \frac{f_y}{f_{cle}} > 1.911 \qquad (6.8)$$

and

$$f_{cle} = 2C_e E \frac{t}{D}$$

where

f_{cle} is the characteristic elastic local buckling strength
C_e is the critical elastic buckling coefficient = 0.3
D is the outside diameter
t is the wall thickness