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BUCKLING CHECK OF STRUCTURES BEING LIFTED

Rafael S. de Macedo*, Luiz Francisco S. de Macedo*

* Emasa Engineering V.O.F.
e-mails: rafael@emasa.eu, luiz@emasa.eu

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Abstract. *The stability evaluation of structures being lifted by cranes poses a challenger for the analysts from the inherent hypostaticity that the problem presents. Usual buckling evaluation for standard structures supported by foundations are not possible and alternative modelling strategies are required. Two strategies can be defined for buckling evaluation: The Linearized Buckling Check and The Direct Method. This paper states the problem and describes both strategies along with the presentation of modelling good practices and a clear framework of codes and standards which bring light into aspects such as load factors; weight contingency; Centre of Gravity imprecision, etc. A comparative study for the results obtained from both methods for a typical truss being lifted is presented*

1 INTRODUCTION

Modularization strategies for steel structure`s erection is currently used as the preferred methodology for sound erection engineering practices. But some challenges exist while performing buckling checks of structural steel elements being lifted by cranes.

The structural system is inherently hypostatic (for small displacements, no lateral restriction is present as the part only "pendulates" below the crane). Figure 1 brings an example of a structure being lifted. All cables converge to a single node (the hook) which does not have a well-defined lateral restrain.

Figure 1. Example of a steel structure being lifted.

In practice, the element is stabilized by the lateral inertial forces which arise from its mass and the small inclination that the vertical cable gains when small lateral displacements exist. This generates a dynamic model and the equilibrium in practice is a dynamic equilibrium (as opposing to a static equilibrium) with small amplitudes on the horizontal displacements. From that, the structural models are not well posed for a static numerical analysis (which is often preferred in day-by-day engineering practice since it is much simpler than a dynamic analysis and requires much less computational efforts).

Also, being only vertically supported, the models are very sensitive to lateral effects. So, the Notional Loads which are the simplest and usually most efficient means to consider lateral imperfections in a model, cannot be used since they unavoidably produce spurious lateral reactions. Those can be neglected for most structures supported on ground, but from the conditions of support and geometric characteristics of a lifting analysis, they can lead to inexistent large internal forces and prediction errors.

Other issue is that the parts being lifted usually are only segments of the final structure that do not have a well-defined bracing system where braced nodes can be easily identified. Also, in many situations, structures being lifted have irregular geometry. This can make the identification of typical buckling factors to be considered on determining the equivalent buckling lengths (i.e. the "k' factors used in traditional buckling evaluation) extremely difficult or even not possible at all.

To deal with the ill posedness of the problem, modelling good strategies are available which simulate the actual condition of support at site during lifting. They are presented on chapter 2.1. To handle the instability check without the use of "k" factors, two possible buckling verification methodologies (both using the buckling eigen-modes as basis for the analysis) can be used. One is the Linearized Buckling Check (as for example described by DNV-GL [1]) with the proper correction on the buckling eigen-values by the suitable buckling curves (preferred method for plate or panel-like structures). The other, the Direct Method (e.g. from the AISC Design Guide 28 [2]), is a full-non-linear method performed upon the imperfect shape obtained from some eigen-modes calibrated to reach the prescribed possible fabrication and erection tolerances (this method is usually preferred for frame-like structures).

Comparative results from the application of both strategies on a typical structure are presented on chapter 3. The typical structure selected is a plane truss which, from not being braced as on its final configuration, often leads to erection problems or even buckling collapse during their lifting operations.

2 MODELLING APPROACHES FOR BUCKLING CHECK

2.1 Transformation into a well posed problem

The problem of an ill posed problem is solved by strategically adding fictious lateral restrains on the model at selected nodes. The quantity of lateral restrain must be minimal. The process for adding the restrains is a trial-and-error process where the experience from the analyst has great influence.

The idea is to simulate the lateral reactions which would arise during lifting. Such reactions mainly develop due to the force which construction personnel imposes on the structure by means of guide ropes, in order to restrain the pendulum action and the spinning of the part around the hook. Figure 2 below illustrates a part being restrained by construction personnel.

Figure 2. Part being restrained by construction personnel

The process for setting the lateral restrain follows these steps:

1 - The analyst selects a pair of nodes of the structure where most probably the restraining lines would be connected to.

2 - The structural model is solved and the horizontal reactions on those supports are captured.

3 - The reactions obtained are compared in terms of order of magnitude to the force that construction personnel could impose on the structure. The value of around 10kN is a good reference.

4 – If the reactions are large, then a new set of locations for the restrain must be selected.

Figure 3 below demonstrates the position used for the simulation of a module being lifted.

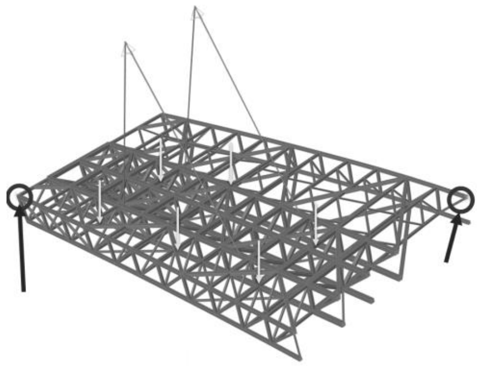

Figure 3. Position of fictious lateral restrain

The resulting reactions above are deeply influenced by the position of the theoretical of gravity of the model versus the position of the vertical support which simulates the hook. If the C.o.G. is not lying exactly below the support (or below the line that connect two supports when two hooks are used), large fictious lateral reactions will arise in the model. A calibration procedure on the position of the vertical supports, or on the position of the C.o.G. (by adding fictious vertical forces on the model, see the arrows pointing down on Figure 3) must be made in order to ensure the correct C.o.G. positioning.

For some models it is beneficial to have the fictious lateral restrains as springs in order to decrease the reaction on them. The spring stiffness must again be somehow related to the spring of the restraining lines used at site.

2.2 Buckling modes

After having the model in a well-posed condition, the buckling eigen-modes are captured using the self-weight as imposed loading. The buckling modes will be used in both proposed methods for buckling check. In the Direct Method, the imperfect shape for the buckling analysis will be derived from the mode shapes, and in the Linearized Buckling Check the eigenvalues will be threated in order to yield the buckling capacity of the system.

A sufficient quantity of modes must be captured in order to ensure that all shapes which can relate to how the structure could buckle during lifting are appreciated. For the Direct Method Analysis, only global modes are used. For the Linearized

Buckling Check both local and global modes must be explored. Figure 4 below illustrates the difference between local and global buckling modes.

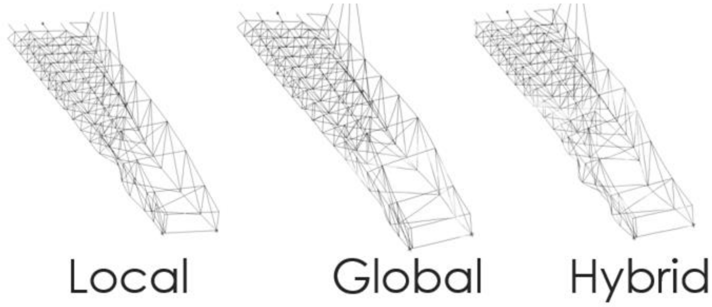

Figure 4. Buckling Modes of a Structure

2.3 Direct Method from AISC

The Direct Method here described is based on the AISC guidelines [2] on how to perform the geometrical non-linearities evaluation of a structure directly, by using software with non-linear capabilities. The method makes use of the structure modelled considering an imperfect shape as starting point for a loading being incrementally imposed. The initial imperfections "trigger" the instability process which naturally arises during the software incremental evaluation. Global instability is reached when the load-displacement curve of a characteristic location reaches unloading or when the incremental loading process does not converge anymore. Figure 5 brings plots which illustrates both behaviours.

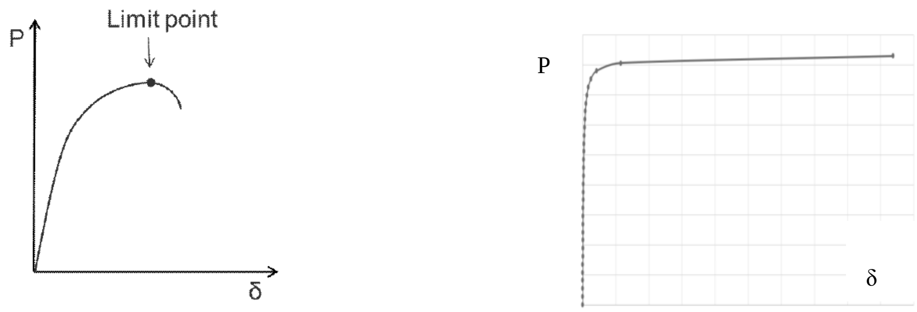

Figure 5. Buckling limit reached by unloading (Left). Buckling limit reached by convergence criterion (Right)

The imperfect shape is obtained by scaling a buckling eigen-mode shape which is elected as characteristic for the problem. The scaling is made in a manner that the deformed shape`s peaks match standardized possible misplacements from the fabrication process (e.g. the misalignments proposed by AISC COSP [3]).

The direct method requires that a softening of the structure is considered (e.g. by reducing its elasticity modulus) and that inner nodes are included in frame elements to correctly capture the non-linear effects within the frame limits. This method greatly simplifies the analysis, since all buckling factors ("k" factors) can be considered as one.

If the model has converged for the properly factored loading, and has not reached the unloading limits as shown above, the structure can be considered stable from a global perspective. The possible member instabilities are checked using the traditional analytical formulae for column buckling for each element.

2.4 Linearized Buckling Check from DNV

The Linearized Buckling Check is a traditional method to access the buckling capacity of plate-like structures (e.g. maritime stiffened panels) which can also be used to access the stability of frame-like structures. In this method the buckling eigenvalues serve as input to, along with considerations for construction imperfections and residual stresses, determine a buckling capacity for the structure. This procedure includes both local and global verifications.

The buckling eigenvalues represent theoretical numerical load multipliers which make the second-order stiffness matrix to be singular. It is important to notice that they are obtained from a theoretical numerical analysis which does not include the deleterious effects from imperfections or residual stresses. Therefore, they do not directly represent how far the system is to buckle in reality (a dangerous non-conservative assumption often made). To properly assess the buckling capacity, the analysis must include the deleterious effects mentioned above.

According to DNV-GL [1] The analysis goes as follows (with small editions by the author):

i) Build the model.

ii) Perform a linear analysis for the dead load case.

iii) Determine the buckling eigenvalues and the eigenmodes (buckling modes) by means of FE analysis.

iv) Select the governing buckling mode (usually the lowest buckling mode) and the point for determining the buckling representative stress. The point for reading the representative stress is the point in the model that will first reach yield stress when the structure is loaded to its buckling resistance.

v) Determine the von-Mises stress at the point for the representative stress from step ii).

vi) Determine the critical buckling stress as the eigenvalue for the governing buckling mode times the representative stress. Determine the reduced slenderness as function of the representative stress and the structure`s yield stress.

vii) Select empirically based buckling curve to be used based on the sensitivity of the problem with respect to imperfections, residual stresses and post buckling behaviour to obtain proper buckling capacity reduction factors.

viii) Determine the buckling resistance as function of the reduction factor, the imposed loading, the yield and representative stresses and material factors.

If the buckling resistance is larger than the imposed dead loading with the proper load factors, then the structure is safe for lifting.

2.5 Load Factors

Various codes address the proper load factors to be used in order to ensure that the lifting analysis is within a minimum reliability level. Most will deal with the possible load uncertainties by first classifying them and later obtaining partial load factors for each uncertainty type present in the lifting operation. Typical load factors are the weight contingency factor, the dynamic amplification factor and the sling arrangement indeterminacy factor. The reader is referenced to [4] and [5] for more information on the proper load factors to be used in the analysis.

After the loads are factored, the two modelling approaches described above can be applied.

3 CASE STUDY

A case study is performed in order to compare the results obtained from both methodologies. The capacity obtained will be expressed in terms of a maximum possible fictious weight that the structure presented could have until it reaches buckle collapse. This paper's analysis focuses on comparing the abovementioned methodologies, therefore no load factor or material factor is applied. For a proper analysis, load factors should be obtained as described in the previous chapter.

3.1 Characteristics of the problem

The geometry modelled is a truss having a total length of 60m and 3m height (similar to a truss used in a stadium roofing or in a hangar). The hook elevation is set as to be 14m above the centre of the truss. Figure 6 demonstrates the arrangement.

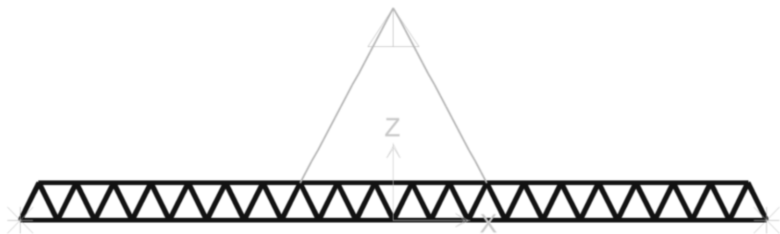

Figure 6. Geometry for Case Study

At both lower extremities, out-of-plane lateral restrains are provided simulating restrain lines. The node on the left also restrains horizontal in-plane displacements. Frames are modelled with a HP14x73 section (A572Gr50) without end-releases. The in-plane orientation of the elements makes the profile's major axis to be directed in the vertical orientation. Cables are modelled with catenary cable elements with area of 645mm^2.

The only load applied is the self-weight of the structural steel elements. No load factor is considered.

3.2 Results from Linearized Buckling Check from DNV

The Self weight load case is solved without imperfections considered. Below is the relevant information for the analysis captured from the model:

- Dead weight of the structure (reaction at vertical support) = 268.49kN
- Reactions at horizontal supports (to validate the restraining line similarity): -0.03kN (ok)
- Buckling Mode Load multipliers (eigenvalues) are shown in Table 1

Table 1. Buckling Eigenvectors

Mode Number	Load Factor (Eigenvalue)
1	-0.41
2	-1.00
3	2.82
4	-4.20
5	6.16
6	-7.12

In this case only the positive values are of interest and the smallest is selected for the analysis. In real situations, other modes should also be investigated to ensure that all possible local and global buckling possibilities are explored. The buckling shape for the 3rd mode is shown on Figure 7.

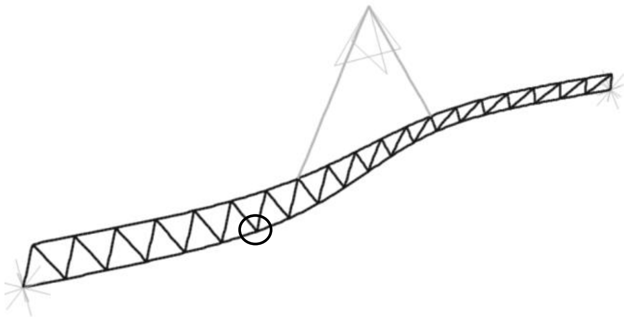

Figure 7. Shape of the 3rd Buckling Eigenmode. Circle marks the location with maximum displacement.

- The buckling representative VM stress is selected at the location shown on Figure 8 as the maximum stress for the 3rd mode with obtained from the Buckling analysis (this represents the location which will first reach yield).

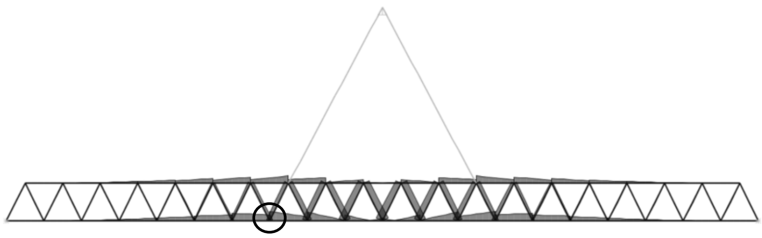

Figure 8. Buckling Modes of a Structure. Circle marks the location with maximum stress.

- The representative VM stress is obtained at the same location above for the Self Weight load case as 30.3MPa.
- The Critical Buckling stress is determined as 85.45MPa by multiplying the 3rd mode eigenvalue and the buckling representative VM stress.
- The reduced slenderness is calculated as 2.00 (See [1]). Assuming a moderate condition for tolerances and residual stresses, a reduction factor on the yield capacity of 0.1938 is obtained using a buckling curve for a column type structure.

With the parameters above the total resistance of the problem against buckling is calculated and expressed in terms of maximum possible weight as 592.48kN. For a real problem, load factors and material factors should be applied on this value. Both global and local stability are understood as checked by this methodology.

3.3 Results from AISC Direct Method

3.3.1 Global Stability

By making use of the 3rd eigenshape above, an imperfect shape is considered as a starting geometry for a non-linear analysis. The shape is scaled to have the peak lateral displacement matching L/1000 * 1.5 (due to the complete wave shape of the third mode, L is the half length of the truss), and leads to a lateral deviation on the point marked in Figure 7 of 45mm. The L/1000 is a fabrication imperfection criterion take from AISC COSP [3] and the 1.5 factor allows for a simplified procedure for the required softening of the structure as just having the Elasticity Modulus set to 0.8*E (this is equivalent to considering a Notional Load of 0.003*(Vertical Actions). See AISC Design Guide 28 [2]).

The load is amplified so that it is ensured that the collapse load will be reached. The plot on Figure 9 demonstrates the evolution of the out-of-plane displacement of the node marked on Figure 7 until the last step converged.

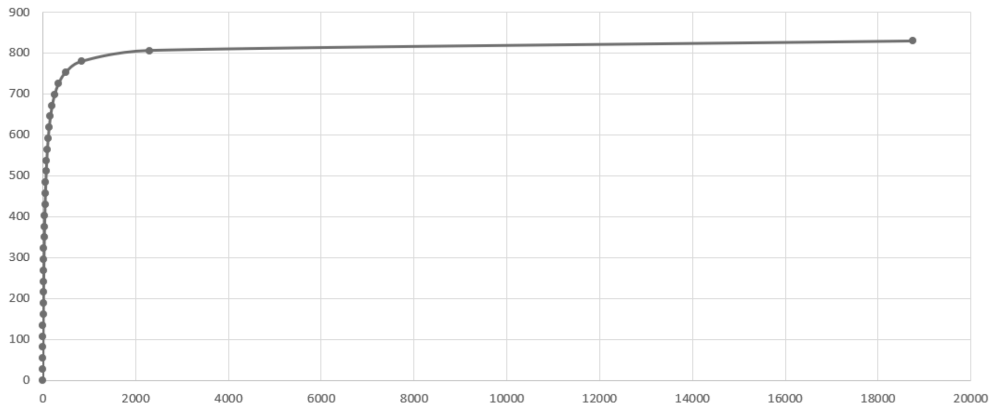

Figure 9. Buckling Modes of a Structure. Circle marks the location with maximum stress

The capacity for global instability is obtained by capturing the reaction at the support at the last stage, which is 828.57kN. Again, in a real-life analysis the required load and material factors would need to be applied.

3.3.2 Local Stability

The local stability is checked using the traditional analytical formulae from AISC [6] for a column using the internal forces from the non-linear analysis described above. The chord element capacity is exceeded (i.e. no load factors applied) when the weight reaches 771.67kN at the same location marked in Figure 8. This capacity governs the design.

4 CONCLUSIONS

This paper demonstrates the challenges that arise when checking the stability of a structure for its lifting, as well as presents two suitable design approaches to the problem. In the Case Study, the approach using the Direct Method from AISC [2] gives a capacity 30.2% larger than the procedure as described by DNV-GL [1] to check for buckling. This value seems reasonable since the Linearized Buckling approach is a less refined method that depends on many assumptions and checks both the local and global stability in one single analysis.

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