Study on Ultimate Capacity of Offshore Jacket Platforms Considering the Effects of Global and Local Buckling of the Elements

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Abstract
The capacity curve of offshore structures including jacket-type offshore platforms is used to achieve structural performance levels and determine their ultimate capacity and ductility. This curve is obtained from the follower analysis and accurate estimation of that is of great importance. Formation of fatigue cracks at the joints, corrosion of members, environmental loads and damages caused by accidental dynamic loads such as impacts of vessels and floating bodies will result in the change of the ultimate capacity of these structures over the course of their life. These cases should be considered in calculating the ultimate capacity of the offshore platforms at any given time of their life. In all of these cases, accurate modeling of the global and local buckling of compression members is important at any time of calculating the ultimate capacity. Buckling modes and deformations due to local buckling will be considered if the compressive braces are modeled by Shell or Solid elements and the imperfections are applied. This paper aims to achieve the correct compressive behavior of compression members. The buckling envelope derived from the Marshall strut theory defines the post-buckling damaged elasticity model and the hysteretic loop response. Finally, by using this modified behavior in Frame elements, the effects of local buckling in compressive braces can be considered. The innovation of this paper is investigating the interaction of local and global buckling in the braces of jacket using ISO equations with 1-Dimentional Frame elements, which results in low computational costs.

Keywords: Jacket type offshore platforms, buckling, Marshall Strat Theory, compressive behavior, ultimate capacity.

1. INTRODUCTION
Critical buckling loads can be calculated for tubular members using empirical equations. These equations are employed by design codes such as API RP 2A-LRFD: (Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms) for achieving load and resistance design factors in order to assure that design loads in normal and extreme operational conditions do not meet buckling loads. Also design codes cover both global elastic buckling and inelastic buckling and include decreasing of loads to prevent local buckling of walls in tubular members as well as imperfections. Hence, structural configuration has been explained by codes in such a way that applied loads are smaller than buckling loads in normal operations.

Some relevant studies have been conducted on the buckling of the tubular members. Yasseri and Skinner (2006) conducted a detailed study in the form of a technical report on the global and local buckling and their interaction for tubular members under concentrated force and moment. They categorized the compressive members based on the ratio of length to gyration radius (L/r) and the diameter to thickness ratio (D/t) and investigated the suggested API curve for elastic and non-elastic buckling based on the slenderness. The effects of taking into account the imperfection of these members and finally, the introduction of the modified properties of steel material which can be used for beam elements to achieve the same response of shell elements, are also presented in that report. Wenjing and Hoogenboom (2011) conducted a buckling check involving manual determination of the buckling lengths of the frame member. They estimated that 5 to 10% of the man-hours in structural analysis of removal projects is spent on checking and correcting buckling lengths. An alternative method is available that does not require determining buckling lengths. In their paper it was shown how this method can be derived from the NORSOK standard for tubular steel frame structures. The method has been demonstrated in the removal analysis of an offshore jacket. They concluded that this method can be successfully applied.

In Extreme load conditions such as DLE the consequences of this event for structures are needed to be considered when some members compress higher than their buckling limit. It is not always necessary to design structures in a way that local failure does not occur in members under critical loads. Subsequently, analysis concentrates on whether the level of structural failure remains within acceptable limits. For this analysis, load factors and capacity factors are equal to 1 and the actual response of the structure is calculated. It is particularly important that local buckling of
members and their post-buckling behavior are properly
diagnosed in a way that load transfer to adjacent
elements is not underestimated. Collapse analysis of
jacket structures is usually done using nonlinear finite
element models with 1-D Beam elements. These
elements do not represent the local buckling response
and local deformation of walls of members. Local
buckling often occurs in members in which diameter to
thickness ratio (D/t) is low. By increasing the diameter
to thickness ratio, local buckling becomes significant
up to the point where interaction between global and
local buckling is established, leading to a further drop
in post-buckling capacity in comparison with the
predicted value using the Beam element. In addition,
models with Beam elements cannot directly consider
radial and circumferential imperfections due to
fabrication tolerances which also play an important
role in reducing load carrying capacity.

Thin-walled tubular members which have been
modeled using Shell elements are capable of
considering global and local buckling at Shell walls
simultaneously.

In this study, for a 2-D frame of a typical jacket
platform, models using Shell or Beam elements are
compared, and then, the Marshall strut theory in the
AB AQUS finite element software suite is used, and the
responses of Beam elements that can also consider the
effects of local buckling are obtained.

2. ANALYTICAL MODELS

Models were generated in two ways with second-
order Shell and Beam elements. Frame elements were
used to investigate local buckling that it can be formed
well in the Shell elements because of local
deformations in walls of tubular parts. The most
important difference between Beam and Frame elements is that Frame elements in addition to the Beam
elements equations, use ISO 19902 buckling equations.
Also, in terms of numerical modeling, Frame elements
should have one mesh element in length. After buckling
occurs, the standard frame element response is switched
to the buckling strut response and is never switched
back again. If the buckling strut response is requested
for the element from the first step of the analysis, the
member will be changed to a simply-supported member
and the bending moments cannot be supported by the
member. An appropriate mesh in a model provides
optimum precision. The results of a fine-meshed model
show that in Shell models, local buckling in the load-
displacement graph occurs earlier than Beam models
and also does not reach the same post-buckling
strength. But Frame elements compensate this
deficiency of Beam elements and even possess less
computational costs as well.

To consider buckling phenomena under compressive
loads in Shell and Beam models, a primary
imperfection equal to 0.1% of member length was
applied to the model. The locations of imperfections
were determined and applied based on possible
buckling modes. This primary imperfection was
obtained by calculating the eigenvalues of the model.
Applying the imperfection is not required in models
with Frame elements; the Frame elements consider both
elastic and inelastic buckling.

For better comparison of results between Shell
elements and Beam-Frame elements, mesh size is
almost the same at both models. It should be noted that
Frame elements only need a single mesh along its
length.

3. ANALYSIS METHOD

Among the commonly used methods, the Static Riks
analysis was conducted using AB AQUS because the
other analysis methods become unstable under a sudden
reduction in stiffness caused by buckling. The Riks
procedure makes it possible to consider deformations
and loads simultaneously. This means that the
magnitude of the load is taken as a variable and the
solution is obtained using the Arc length method in a
static equilibrium in the load-displacement space. In
this way, AB AQUS is able to continue solving the
load-displacement equation by reducing the applied load
to achieve the post-buckling response of the model.

This procedure was used in Beam and Shell models
with and without imperfection to compare the buckling
and post-buckling responses. Finally, the materials of
Beam elements were calibrated based on the responses
of the two models, to achieve the same post-buckling
stiffness as the Shell model.

4. LOADING

The purpose of this article is enforcing displacements
to nodes at different levels of the platform and
calculating the reaction forces at the level of the fixed
supports at the bottom of piles, which is the base shear
and equal to the total load applied to the platform. Thus,
the load-displacement curve of the structure is obtained
that also includes total ductility path. Therefore, lateral
displacement was applied by pushing the structure
instead of applying concentrated lateral load.

Distribution of displacement was imposed at the
highest level by 100 percent and 60 percent of the same
displacement at the lower level.

5. MATERIAL MODEL AND SOLVING
EQUATIONS

Yield stress of 355 MPa and ultimate strength of 535
MPa at a plastic strain of 0.144 for almost all elements
were defined. True stresses are obtained from the
narrowing test of the coupon bar under axial load. The
following equations are used to achieve true stresses
and logarithmic strains from the nominal experimental
values.

\[ \sigma_{\text{true}} = \sigma_{\text{nom}}(1 + \varepsilon_{\text{nom}}) \]  
\[ \varepsilon_{\text{in}}^{pl} = \ln(1 + \varepsilon_{\text{nom}}) - \frac{\sigma_{\text{nom}}}{E} \]

6. BUCKLING PREDICTION AND ISO
EQUATION

The ISO equation is used to predict the onset of
buckling in slender members with pipe-like cross-
sections. All quantities with dimensions have dimensions of stress. The quantity of $I$, which is a function of the axial compressive stress, $f_c$, and the maximum bending stresses about the local 1 and 2 axes, $f_{b1}$ and $f_{b2}$, are defined by the following expression:

$$
\frac{\gamma_k \varepsilon_k}{f_c} + \frac{\gamma_k s_k}{f_c^2} \left( \frac{c_m s_m}{f_c} \right)^2 + \left( \frac{c_m s_m}{f_c} \right)^2 \leq 1.0
$$

(3)

Here, $f_c$ is the characteristic axial compressive stress, $f_b$ is the characteristic bending stress, $c_m1$ and $c_m2$ are reduction factors corresponding to the cross-section directions 1 and 2, and $F_{c1}$ and $F_{c2}$ are the Euler buckling stresses corresponding to the 1 and 2 directions.

Considering the following notation

<table>
<thead>
<tr>
<th>D: outer diameter</th>
<th>t: wall thickness of pipe</th>
<th>P: axial force</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma^0$: yield stress</td>
<td>E: Young’s modulus of elasticity</td>
<td>A: cross-sectional area</td>
</tr>
<tr>
<td>$Z_e$: elastic section modulus</td>
<td>$k_1$, $k_2$: effective length factors in the 1 and 2 directions</td>
<td></td>
</tr>
<tr>
<td>$Z_p$: plastic section modulus</td>
<td>$L_1$, $L_2$: unbraced lengths for the 1 and 2 directions</td>
<td></td>
</tr>
<tr>
<td>$r$: radius of gyration</td>
<td>$I_{11}$, $I_{22}$: bending moment of inertia in 1 and 2 directions</td>
<td></td>
</tr>
</tbody>
</table>

where,

$$
I_{11} = I_{22} = \frac{\pi}{64} (D^4 - (D - 2t)^4)
$$

(5)

$$
Z_e = \frac{\pi}{64} (D^4 - (D - 2t)^4) / (D/2)^4
$$

(6)

$$
Z_p = (D^3 - (D - 2t)^3) / 6
$$

(7)

$$
r = \sqrt{I_{22}/A} = \frac{1}{4} \sqrt{(D^2 - (D - 2t)^2)}.
$$

(8)

Therefore, the terms of the ISO equation are calculated as follows

$$
f_c = P/A
$$

(9)

$$
f_{b1} = M_t/Z_e
$$

(10)

$$
F_{yc} = f_y
$$

for $f_y / f_c \leq 0.17$

$$
F_{yc} = (c_2 - c_3 f_y / f_c) f_y
$$

for $f_y / f_c \leq 1.911$

$$
F_{yc} = f_c
$$

for $f_y / f_c > 1.911$

$$
c_2 = 1.04654873 \quad \& \quad c_3 = 0.27381606
$$

(12)

$$
F_e = 2CE \left( \frac{1}{D} \right) \quad \& \quad C = 0.3
$$

(13)

where, $F_{yc}$ is the local buckling strength;

$$
F_c = \left[ 1 - 0.278\lambda \right] F_{yc} \quad \text{for} \quad \lambda \leq 1.34
$$

$$
F_c = \frac{c_1}{\lambda^2} F_{yc} \quad \text{for} \quad \lambda > 1.34
$$

(14)

$$
\lambda = \text{max}(\lambda_1, \lambda_2) \quad \& \quad c_1 = 0.89282978
$$

(15)

$$
\lambda_i = \frac{k_i L_i}{A} \sqrt{\frac{F_{yc}}{E}}
$$

(16)

where, $F_c$ is the representative axial compressive strength, in stress units; $\lambda$ is the column slenderness parameter,

$$
F_b = \left( Z_y / Z_e \right) f_y
$$

for $f_y / f_c \leq 0.0517$

$$
F_b = \left( c_4 - 2.58 \left( \frac{f_y}{f_e} \right) \right) \left( Z_y / Z_e \right) f_y
$$

for $0.0517 < f_y / f_e \leq 0.1034$

$$
F_b = \left( c_5 - 0.76 \left( \frac{f_y}{f_e} \right) \right) \left( Z_y / Z_e \right) f_y
$$

for $0.1034 < f_y / f_e \leq 120 f_y / E$

$$
c_4 = 1.133386
$$

(17)

$$
c_5 = 0.945198
$$

(18)

where, $F_b$ is the bending strength;

$$
F_{el} = \frac{F_{yc}}{A}
$$

(19)

$c_{m1}$ and $c_{m2}$ are reduction factors corresponding to the cross-section directions 1 and 2. These factors are functions of the end moments, compressive stress and Euler buckling stresses with the default values of 0.85. By satisfying the following condition

$$
I(f_{c1} f_{b1}, f_{b2}) = 1.0
$$

(20)

the standard frame element response is switched to the buckling strut response and is never switched back again. The ISO equation yields the critical load, $P_{cr}$, which is defined as $f_y A$. When the axial forces are negligible and the bending moments have large values, another inequality is also used. This additional control is called the strength equation and is as follows

$$
S = \frac{f_c}{F_{yc}} + \frac{1}{F_{yc}} \sqrt{f_{b1}^2 + f_{b2}^2}
$$

(21)

Both the following equations must be satisfied ($I=1.0$ and $S\leq1.0$) to switch to the buckling strut behavior for a frame element.

$I=1.0$

(22)
S ≤ 1.0

If the buckling strut response is requested for the element from the first step of the analysis, the member will be changed to a simply-supported member and the bending moments cannot be supported by the member. In this state, the ISO equation turns into the following simple equation:

\[ P_{cr} = F_c A \quad \text{and} \quad f_c < F_c \]  

(24)

7. MARSHALL STRUT ENVELOPE

The Marshall strut envelope defines the post-buckling damaged elasticity model and the hysteretic loop response. To define the Marshall strut envelope, the value of \( P_{cr} \) and the following seven constants are needed:

- \( \xi \): is the coefficient defining \( P_y = \xi \sigma_0 A \) (\( \xi = 0.95 \))
- \( \gamma \): is the isotropic hardening slope coefficient (0.02), \( \alpha_0 \): is the coefficient defining \( A_0 + \alpha_1 \frac{D}{L} \) (\( \alpha_0 = 0.03 \))
- \( \alpha_1 \): is the coefficient defining \( A_0 + \alpha_1 \frac{D}{L} \) (\( \alpha_1 = 0.004 \))
- \( \kappa \): is the force coefficient (0.28), \( \beta \): is the slope coefficient (0.02), and \( \zeta \): is the force coefficient \( \left( \min \left( \frac{1}{10} , \frac{5.8}{\kappa} \right) \right) \).

The values in parentheses are the default values supplied by ABAQUS, and the value of \( P_{cr} \) is found from the ISO equation as explained above. The Marshall envelope governs the compressive and tensile response of the strut as shown in Figure 1. The dotted lines in the interior of the envelope indicate the damaged-elastic modulus defining the loading-unloading force versus strain path [1,2].

8. MODELING

A two-dimensional frame of a four-legged jacket platform of the South Pars phase 10 was modeled. The dimensions and geometry of the platform are shown in Figure 2.

Table 1 consists of aspect ratios of sections used in the model.

The jacket was modeled using Shell elements and its envelope was obtained. Then, using two-node linear Beam elements, modeling and pushover analysis was executed and in the next step, pushover analysis was performed using the Frame elements and Marshall theory.

Jackets and piles were modeled separately and the cylindrical connection between them was defined so that free rotational and translational driving of piles into the legs can be done. In all the models, the Pile Jacket technique was used in a way that the fixed end point of the piles was set at a depth equal to 10 times its diameter. The main deck was located on the upper end of the piles in the form of a rigid plate and was welded to them. Two concentrated masses were applied on both sides of the main deck and each of them was intended to be equal to 25% of the total mass of the platform deck.

Figures 3 and 4 show geometries modeled with Shell and Beam elements. The geometry of Frame elements is the same as Beam elements.

One of the cross-section geometries of the Frame elements has been shown in Figure 5.

<table>
<thead>
<tr>
<th>Outer Diameter (cm)</th>
<th>Thickness (cm)</th>
<th>D/t Ratio</th>
<th>Moment of Inertia (cm4)</th>
<th>Area (cm2)</th>
<th>Radius of gyration (r) (cm)</th>
<th>Minimum Length (m)</th>
<th>Maximum Length (m)</th>
<th>Min. Length to r Ratio (Lmin/r)</th>
<th>Max. Length to r Ratio (Lmax/r)</th>
</tr>
</thead>
<tbody>
<tr>
<td>45.7</td>
<td>1.27</td>
<td>35.98</td>
<td>43777</td>
<td>173.27</td>
<td>15.71</td>
<td>14</td>
<td>14</td>
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<tr>
<td>50.8</td>
<td>1.588</td>
<td>33.99</td>
<td>74400</td>
<td>243.51</td>
<td>17.41</td>
<td>15</td>
<td>15</td>
<td>86.17</td>
<td>86.17</td>
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<tr>
<td>50.8</td>
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<td>135280</td>
<td>473.04</td>
<td>16.88</td>
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<td>11</td>
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<td>48.03</td>
<td>106326</td>
<td>238.31</td>
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<td>13.75</td>
<td>18</td>
<td>65.10</td>
<td>85.22</td>
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<tr>
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<td>51.97</td>
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<td>22.89</td>
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<tr>
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<td>60.00</td>
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<td>64.16</td>
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<tr>
<td>76.2</td>
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<td>40.00</td>
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<td>444.64</td>
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<td>22</td>
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<tr>
<td>165.5</td>
<td>1.5</td>
<td>110.33</td>
<td>2998478</td>
<td>772.83</td>
<td>57.99</td>
<td>22</td>
<td>22</td>
<td>37.94</td>
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<tr>
<td>166.6</td>
<td>2</td>
<td>83.25</td>
<td>3496647</td>
<td>1033.58</td>
<td>58.16</td>
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<td>26</td>
<td>32.67</td>
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<tr>
<td>152.4</td>
<td>7.62</td>
<td>20.00</td>
<td>9106300</td>
<td>3465.88</td>
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<td>17.14</td>
<td>10357895</td>
<td>4008.06</td>
<td>50.84</td>
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<td>152.4</td>
<td>4.445</td>
<td>34.29</td>
<td>5658644</td>
<td>2066.10</td>
<td>52.33</td>
<td>19</td>
<td>22</td>
<td>36.31</td>
<td>42.04</td>
</tr>
</tbody>
</table>
Fig. 2. Geometry of 2-D modeled frame belongs to the platform of the South Pars phase 10.
9. RESULTS

The numerical results of models with Frame elements were validated with the ISO equation in such a way that the critical buckling load obtained from the Frame elements was compared with the results of the ISO equation, and the response error was less than 2% in all members.

Buckling force is independent of the cross-sectional diameter to thickness ratio (D/t) and by increasing the length to radius of gyration ratio (L/r) in long members with L/r > 100, failure via elastic buckling of the member occurs prior to yielding. The critical buckling force can be predicted by the Euler formula [3]:

\[ P_{cr} = \frac{n \pi^2 E I}{L a^2} \]  

(25)
n=1 is equivalent to the first mode and $L_e = L$ in the simply supported members. For hollow members with intermediate length ($40 \leq L/r \leq 100$), the Euler formula yields a higher amount of buckling force as the material reaches its limit of proportionality at the outer fiber of the member leading to a reduction in stiffness and kneeling of the section. In these members, inelastic buckling occurs at the inelastic stability limit. Figure 6 gives a comparison between the typical elastic and inelastic buckling shapes.

In short elements ($L/r < 40$), meeting the ultimate limit of resistance and yielding in the section is possible before instability occurs. In the event of buckling in these elements, the buckling shape will be similar to the elements with moderate length; but the difference is that a larger area around the buckled section meets the yield level. Figure 7 shows the relationship between the critical buckling load of columns and column slenderness parameter, $\lambda$, which was introduced in API RP 2A-LRFD as

$$\lambda = \left(\frac{kL}{\pi^2}\right) \times \left(\frac{F_y}{E}\right)^{0.5}$$

Figure 7 exhibits the transition from elastic to plastic buckling by reducing $L/r$ and $\lambda$. The API standard proposes equations for calculation of local elastic and plastic buckling, but global yielding can occur prior to the local yielding of the member by controlling the D/t ratio.

Local buckling increases in shell walls as the D/t ratio increases. At the time of buckling, the shell walls start to swing, and ultimately, the member bends. As the D/t ratio increases, the flexural stiffness of shell wall
decreases with a power of 3 [4]. Local flexural moments occurring due to the deformation of the shell wall quickly create a local plastic zone. As a result, the local buckling reduces the load capacity, ultimately resulting in global buckling.

As the L/r ratio increases, the critical buckling load decreases and the plastic zone gets larger. In this state, if the local buckling occurs, the enlarged plastic area causes a sudden collapse [4].

According to Table 1, for the frame discussed in this article, D/t values range from 16 to 110 and slenderness ratio (L/r) ranges from about 33 to about 89.

The results of the pushover analysis on the three models are as follows in Figure 8. Three models were created using a) Shell elements, b) Beam elements and c) Frame elements (using the Marshall theory).

As can be seen, Using Frame elements with the Marshall’s buckling theory can perfectly illustrate the buckling and post-buckling behavior of the system, such as the shell element model. Frame elements can consider the effects of local buckling. The time and cost of modeling and analysis by using Frame elements are much less than those using shell elements. The images in Figure 9 show the deformation of the models after the pushover analysis; the figures also display tension and displacement contours at members.

![Fig.8. Results of the pushover analysis on the three models.](image-url)
10. CONCLUSIONS

The Frame elements use Marshall’s buckling theory, which is described in detail in the article. As the internal loads of a Frame element reach a certain value that results in equalization of \( I(f_1, f_2, f_3) = 1 \), the response of the standard Frame element switches to the strut buckling response. Based on the results obtained from the Frame elements in the buckling analysis, it is proved that these elements can consider the effects of global and local buckling. Frame elements can consider yielding of a member due to local buckling. The Kneeling phenomenon is considered in these elements. Therefore, the time and cost of modeling and analysis by using these elements are much less than those using shell elements. These elements can be used for assessment of buckling and post-buckling behavior in offshore platforms. Many other problems involved in modeling with Shell elements, such as changing the response precision and error due to mesh size, selecting the correct imperfection value, and other similar issues are eliminated with Frame elements.

REFERENCES