The Old Derrick Steel Truss Structure in Linear Buckling Analysis (EigenValue)

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Abstract. The old derrick steel truss structure had been found in imperfect structure such in some member loses and usual bolt holes. The derrick structures were located in critical path room publics for instance the public transportation path and the electricity transmission networks. This research had a numerical model the single element linear buckling (eigenvalue) that were based on the finite element method. The derrick prototypes had been modeled in single element that had the largest axial compression load that was defined in the perfect model. The artificial circle holes were attached on the member then was defined in the imperfect model. The artificial circle hole as representative the unused bolt holes. The linear buckling analysis had been performed by utilizing the 3D frame and 3D solid idealization with the 6 buckling modes had been required. This research had also compared the single element buckling and the overall derrick structure buckling analysis. The results showed that the buckling capacity reduced due to the number of holes. The greater number had the lowest critical loads (Pcr) then had buckle early. The numerical buckling pattern had similarly with in the elastic buckling theory (Euler buckling) at every mode. The critical buckling stress was lower than the yielding stress for every models. The results also showed that the critical load of the single element was lower than the overall structure. However, considering the environmental load such in the seismic load then the overall derrick structure buckling failure had higher vulnerability than the single element buckling failure.

Keywords: buckling, critical load, derricks, eigenvalue, holes,

1. Introduction
The imperfect steel truss derrick structure had been found in several cases, one of the popular case was the holes presence at the member. The holes can be caused by the unused bolts holes that were distributed on the truss connection system. The hole presence potentially caused the buckling capacity decreased. In another hand, the critical load (Pcr) was indicator for the buckling capacity. The lower critical load (Pcr) had buckle failure early. The buckling failure can be occurred in two condition correspondently: single element buckling and overall structure buckling. Both of them can be occurred on the derrick system structure partially or simultaneous. The linear buckling (elastic buckling) analysis had succeed as numerical buckling solution by conducting the eigenvalue problem.

The classic formula for the linear buckling (elastic buckling) analysis had been derived by Euler (1757) by introducing the critical load (Pcr) as shown as in Equation 1. This formula very satisfied for one dimentional buckling analysis requirements only.

\[ P_{cr} = \frac{\pi^2 EI}{(KL)^2} \] (1)
Where \( P_c \) was the critical load, \( E \) was elasticity modulus, \( I \) was inertia moment, \( K \) was boundary condition coefficient, and \( L \) was the effective length. The equation 2 only satisfied for 1D case buckling analysis requirements. In this case the boundary condition was assumed in simple supported (bolt connection system). That is why the coefficient of the boundary condition (\( K \)) had been determined \( K = 1 \).

Miftahul et.al (2019) had studied the effect of the pitting corrosion, an extremely localized corrosion that leads to the creation of small holes in metal could trigger structural failure in platform structures. The results showed that the overall buckling capacity of the truss were compared to similar truss based on the element buckling of compression element in the truss model with various hole positions, the overall buckling loads were slightly higher than that of element buckling loads. Usami et.al (2008) showed the one of the required performances in buckling-restrained braces as a structural control damper under severe earthquakes is to prevent its overall buckling, i.e., Euler-type buckling of both the brace and restraining members as a whole. This paper deals with an examination into the adequacy of the commonly used buckling prevention condition with a series of well-controlled experiments. Moreover, a design guideline for the buckling prevention condition was discussed.

The compression member design had been provided in AISC-10 for inelastic column (Equation 2). The critical stress design \( (F_{cr}) \) of the compressive was given.

\[
F_{cr} = 0.658E \cdot F_y
\]

where:

\( F_{cr} \quad \text{: Critical stress design} \)
\( F_e \quad \text{: Critical stress (Euler)} \)
\( F_y \quad \text{: Yielding stress} \)

Miftahul (2011) had conducted the experimental model provided a fairly good value and more approaching the critical load value were based on the analytical results of the 1D-Euler method. However, the value is highly dependent on the value of the modulus of elasticity \( (E) \) which is given to the Euler formulation. The results showed that the object's \( P_c \) SHP test was lower than the MHP test object \( (D24 = 15382.48 \text{ N} < 6D4 = 15663.2 \text{ N}) \). Otherwise with laboratory testing but 3-way buckling effect generated by galvanized pipe showed an unique behavior especially at stiffness and buckling orientation. This research also proved that stresses and stress concentrations that was occured at the time of buckling had not reached the yielding stress. Computers and Structures Inc. (2007) had provided a buckling solution by utilizing the eigenvalue problem in finite element method. The linear elastic buckling solution was presented in the Equation 2.

\[
[K - \lambda \cdot G(r)] \psi = 0
\]

where:

\( K \quad \text{: Material stiffness matrix} \)
\( G \quad \text{: Geometric stiffness matrix} \)
\( \lambda \quad \text{: Eigenvalue, Critical load} \)
\( r \quad \text{: Eigenvector} \)
\( \psi \quad \text{: Displacement vector} \)

Iwicki and Krajeweski (2013) was devoted to the study of out-of-plane buckling of a truss with horizontal braces. The truss is a model of real roof truss scaled by factor = 4. A linear buckling and a nonlinear buckling analysis with geometric and material nonlinearity were carried out. The truss buckling and limit load for different stiffnesses and number of braces are found. The numerical analysis
were verified by the experiment test results. Threshold bracing stiffness condition for full bracing of the truss was proposed. Solazzi and Zrnić (2017) have conducted research on designing a very large crane (main boom is 80 m in length and payload is 60 T) by considering the dynamic effect caused by the load transfer process. This research was developed through an analytical calculation model for the initial design of the crane and using different finite element analysis (FEM) to evaluate the dynamic behavior of the crane. The results also show that the buckling phenomenon is the most critical point of view for this type of crane.

Dacovic and Hegedic (2014) conducted research on risk management approaches in oil and gas on shore construction activities on land. The results showed that a detailed quality approach from the risk management process is associated with risk difficulties to the quantity of experiential knowledge with a very limited risk approach in mitigating construction of oil and gas construction. The results also indicate a significant difference in the contingency of activities when two different risk management approaches are established. Jian et.al (2015) had statically designed derrick of deep oil well drilling rig with poor dynamic characteristics, caused earlier structure failure of the drilling rig and harsh working condition. One such designed derrick was found to vibrate severely in operation while the rotation speed of rotary table is about 120 r/min with the working frequency of 2.0 Hz. To solve this problem, an experimental modal test of the derrick was conducted and the modal frequencies and vibration shapes were obtained. Through comparison of modal frequencies with that of exciting devices, it was found that the severe vibration of the drilling rig was caused by the resonance of second modal frequency (1.96 Hz) and the working frequency of rotary table. Based on principles of sensitivity analysis and structural dynamics modification method, the frequency sensitivities of all nodes on the derrick are calculated and compared, and then seven nodes with high-frequency sensitivity were selected on which corresponding mass are added to vary the modal frequency. Result shows that the second modal frequency of the derrick is reduced to 1.42 Hz and is out of the normal working frequency range of rotary table, which demonstrates that the dynamic characteristics of the derrick had been improved and severe vibration can be avoided.

2. Research Method
The numerical models were consisted in 2 models generally: the perfect and the imperfect model. The perfect model had been defined as the single element model without the holes presence (control). The imperfect model had been defined as a model with the holes presence. The circle holes had been determined previously 7/8” for its diameter (3/4” bolts). The holes represented 4-3/4” bolts with number of unused bolts holes correspondently 1, 2, and 3 (Figure 1) then named IM01; IM02; and IM03. Every models had been conducted in eigenvalue problem (linear buckling) analysis by utilizing 3D frame and 3D solid element. This research In addition, the number of numerical applications based on finite element method to describe the failure simulation of the 3D derrick model. The critical load ($P_{cr}$) of the models had been compared by 1D-Euler formula in single element.

This research also compared between the critical load of the single element and the overall derrick steel truss structure. The equal leg steel L4x4x5/16 had been selected for the models. Some of steel property materials such ini modulus elasticity ($E_S$), Poisson ratio ($\nu$), yielding stress ($F_y$) and ultimit stress ($F_u$) had been determined in A36 steel.

![Figure 1 The L4x4x5/16](image-url)
The derrick prototype was a leg member of derrick steel truss structure (Figure 4). The leg member had been selected because it had the largest axial compression load. The element had been determined A36 steel L4x4x5/16 with 664,20 ft for the length. The member had been classified into inelastic column

\[
\frac{KL}{r} < 4.71 \sqrt{\frac{E}{f_y}} \quad \text{(AISC-10)}
\]  

(4)

Where \( KL/r \) was a slenderness ratio and \( E \) was elasticity modulus. The slenderness ration was one the most important thing for the buckling requirements. The equal leg A36 steel L4x4x5/16 was modelled in 3D frame and 3D solid idealization for the Perfect Model as control model (Figure 2). The imperfect one had modelled in 3D solid idealization only for hole present consideration.

The 3D frame model had been also developed in this study that was concerned in the perfect model. The geometric had refered into AISC Pro for L4x4x5/16 that was available in SAP2000. The frame element had been divided into 6 element for meshing discretizing. The perfect model had been analyzed by performing the buckling analysis case with 6 mode buckling pattern had been required. The compression member had been idealized in simple frame with the simply supported (pinned-roll) as shown as Figure 3. The concentrated load had been carried at one of the edge of the member. The eigenvalue problem had been runned out for the linear (elastic) buckling analysis than compared by the critical load (Pcr) of the 1D-Euler’s formula (Equation 1).
The unused bolt holes could not be modelled in 3D frame element, this was need more powerful element such in 3D shell or 3D solid element. In this case, 3D solid element (Hexagon) had been developed to create the artificial circle hole as represented the unused bolt holes (Figure 4). The maximum bolt holes number in this research had limited in 3 bolt holes number.

Figure 4 showed the imperfect single element model in 3D solid idealization. The bolt size had been determined previously ¾ inch A325. The location and space of the holes had accorded into AISC-10 chapter J.2. The imperfect model had been analysis by linear perturbation to perform linear buckling analysis with 6 buckling modes requested. The meshing had been organized hex element shape with tetrahedron element surrounding the holes.

Figure 5 The derrick prototype and model
Figure 5 showed the derrick prototype and the 3D frame structure model. The model had been built up in full scale with by emphasizing on the four leg deformation caused by buckling failure. The 3D derrick structure analysis was also performed in linear buckling analysis. The overall buckling on the derrick structure had been evaluated at the specified mode. The results had been compared by the single element buckling analysis. The connection system was assumed in simple supported idealization re-presented the centre bolt connection system.

3. Results & Discussion
The critical load ($P_{cr}$) every models had been evaluated and compared. The one dimentional Euler buckling analysis had the highest critical load (166,90 lbs) for the perfect model. The lowest critical load ($P_{cr}$) had been achieved by the perfect model in 3D solid idealization (103,69 lbs). The critical load ($P_{cr}$) comparison had been shown in Tabel 1.

The 3D frame single element had the moderate critical load closed to the Euler formula (Equation 1). The critical load difference between the 1D-Euler buckling analysis and numerical eigenvalue problem analysis was about 1,19%. This was showed that the frame idealization more closed to elastic buckling theory than the 3D idealization. There were some factors influenced the situation, one of them was meshing control and boundary condition

|                | Euler | 3D Frame | 3D Solid |
|----------------|-------|----------|----------|
| Perfect        | 166,90| 164,91   | 103,69   |
| IM01           | -     | -        | 100,42   |
| IM02           | -     | -        | 62,12    |
| IM03           | -     | -        | 50,09    |

Table 1 also showed the critical load ($P_{cr}$) of the imperfect model correspondently IM01, IM02; IM03. The lowest critical load ($P_{cr}$) was occurred when the member had 3 unused bolt holes. This mean due the holes number increased then the buckling failure potentially occurred early. The buckling capacity decreasing percentage correspondently IM01(49,65%); IM02(62,03%); and IM03(69,57%). This value was very significant for the buckling capacity reducing that was caused by the presence of 7/8 inch for the holes diameter. The percentage of the buckling capacity reduction can be shown on Table 2.

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Table 1 The critical load (lbs)

Table 2 The buckling pattern
Figure 6 showed the buckling pattern that was occurred in overall and single element derricks structure. Figure 6(i) showed the overall buckling pattern of the derrick structure. The overall buckling was occurred at mode 5. The critical load ($P_{cr}$) had been recorded 159,31 lbs was lower than Euler buckling theory and linear buckling eigenvalue problem analysis. Figure 6(ii) also showed that the buckling was also occurred on the 4-legs of the derrick structure. This satisfied the first assumption that the buckling failure was occurred on the legs of the derrick structure.

Figure 6(ii) and Figure 6(ii) showed the single element buckling pattern numerically, correspondently 3D frame and 3dsolid idealization. The buckling failure was occurred at the first mode. This was satisfied the theorithic buckling pattern at the first mode. Both of them showed the single curvature buckling pattern at the first mode.

| Table 2 Error (%) |
|-------------------|
| The model | $\alpha$ |
| Perfect | - |
| IM01 | 49,65 |
| IM02 | 62,63 |
| IM03 | 69,57 |

Table 2 showed the percentage the decreasing of the buckling capacity due to the unused bolt holes number increased. Table 2 also showed that hole where located closed to the edge of the member had insignificant effect on the buckling capacity decreasing. The additional holes number had significant effect on the buckling capacity reduction. Furthermore, it was also had high vulnerability when the lateral load such in seismic load was considered.

The highest buckling capacity decreasing percentage showed that the member buckling earliest. The additional holes number in this case had not reduced the effective area ($A_{eff}$). The hole position in align due to member neutral axis had not effect on the effective cross section area. The effective area ($A_{eff}$) in this case has been evaluated 1809858.77 in$^2$. Based on the compression the design member AISC-10 the critical stress design ($F_{cr}$) had been calculated 28,73 Ksi. This value had been Euler critical stress consideration.

The nominal load ($P_n$) can be evaluated by multiplying the critical stress design ($F_{cr}$) and the effective area ($A_{eff}$) then we have 48372594,54 lbs. For the limit design $P_n$ should be multiply by reduction factor ($\phi = 0,9$) then we have 43535335,09 lbs. However, this value still had satisfied the limit load ($1,4D$) design requirements.

![Figure 7 The critical load vs load combination](image-url)
Figure 7 showed the critical load in the different conditions correspondently single element buckling (Euler and eigenvalue) and overall structure buckling (eigenvalue). Figure 7 also showed that the critical load ($P_{cr}$) of the overall derrick structure buckling was the lowest. However, the buckling failure was occurred on the legs member not at the first mode. The first mode of the overall buckling failure was occurred at the bracing of the derrick steel truss structure system (Figure 8).

![Figure 8](image)

**Figure 8** The first mode of overall buckling structure

The first mode of the overall structure buckling had critical load ($P_{cr}$) 121.17 lbs. The lowest critical load ($P_{cr}$) that was produced by eigenvalue problem iteration was the real critical load ($P_{cr}$). The bracing member had been identified as elastic column accorded into Equation 4. That was why the bracing member buckled early.

![Figure 9](image)

**Figure 9** The first mode of single imperfect model
Figure 9 showed the first buckling mode of the imperfect single models. All buckling pattern showed the single curvature that satisfied the first Euler buckling mode (theorithic) with the holes presence. In another hand, it also showed that even holes presence the buckling failure was occurred in linear elastic zone. In this case the buckling failure never occurred because the critical stress was higher than the yields stress. That was mind the member failure by materal not geometric.

Figure 9 also showed the displacement contour of the imperfect models with the holes presence. The maximum displacement was occurred at the mid span in the -y direction of the member correspondently IM01 (0,039 in); IM02 (0,039 in); and IM03 (0,038 in). There was no significant effect between the displacement to the increasing of the holes number. The displacement was monitored in this case was the linear case.

![Graph showing P_cr trendline due to holes number](image)

**Figure 10** $P_{cr}$ trendline due to holes number

Figure 10 showed the critical load trendline due to unused bolt holes increased. The increasing of unused bolt holes had very significant effect on the buckling capacity reducing numerically. The critical load ($P_{cr}$) had been recorded were based linear buckling analysis (eigenvalue) with in 3D solid idealization. Figure 10 also showed that the 3D solid idealization had good results for describing buckling capacity decreased due to the unused bolt holes number increased. However, these results should be compared and verified by nonlinear buckling analysis (load-displacement).

Based on Figure 10 the IM03 (model with 3 unused bolt holes) had buckled earlist among the another models. The trendline was looked linear because of the eigenvalue had been conducted was linear. However, this solution was good enough for the critical load pre-eliminary prediction. For the accuracy the nonlinear P-delta should be proposed for the solution.

| Table 3 The critical loads |
|-----------------------------|
| Model              | The critical load (lbs) | Euler | Eigenvalue |
| Single element       | 166.82                | 164.91|
| Overall structure    | -                     | -     | 159.31     |

Table 3 showed the critical load of the single element and overall of derrick structure when the buckle was occured. The Euler formula in Equation 1 satisfied for one dimentional case only in the single element buckling requirements. Furthermore, for overall structural buckling the 3D finite element was
one good solution for conducting eigenvalue problem to find out the critical load \( (P_{cr}) \). The percentage critical load relative to 1D-Euler formula correspondently were 1.15% for the 3D frame element and 4.50% for overall buckling analysis.

4. Conclusions
Based on the results of numerical linear buckling (eigenvalue) analysis of the derrick structure modeling can be concluded as below: First, generally the buckling failure never occurred in the derrick structure system. That was caused the critical stress was lower than the yield stress, except in the case of bracing member that was satisfied slender member (elastic column) requirements. Second, the overall buckling failure can be occurred earlier than the single element buckling failure. That was depend on the lowest critical load \( (P_{cr}) \) were give from the eigenvalue problem analysis. In this case, the buckling was occurred on the bracing element at the first mode. Third, the holes presence had the very significant effect on the buckling capacity reduced (69.57% for 3 unused bolt holes). However, they had not significant effect on the compressive member design because the presence of holes was located in align neutral axis of the member then reduced effective area insignificant. Conversely, where the holes were located parallel on the neutral axis then the effectif area reduced significantly.

5. Acknowledgments
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