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Examination of The Compressive Strength and Buckling Characteristics of Welded H-Beam And Hot-Rolled H-Beam Columns

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ABSTRACT

One of the most common types of steel profiles used in column and beam construction is the H-Beam profile. This profile has the characteristics of high flexural strength and cross-sectional efficiency against axial compressive loads. There are two main types of H-Beams in widespread use: hot-rolled (prefabricated) H-Beams and welded (welded assemblies). The use of H-Beam steel profiles as column elements is increasing due to their good compressive capacity and structural efficiency. H-Beam profiles are generally used in hot-rolled and welded-built-up forms, which have different manufacturing characteristics, structural behavior, and fabrication costs. The analysis was conducted using theoretical calculations based on SNI 1729:2020, nonlinear analysis based on the Finite Element Method (FEM) using SAP2000. The parameters evaluated include nominal compressive capacity (Pn), ultimate numerical capacity (Pu), behavior and stability against global buckling, fabrication costs, and the cost-to-capacity ratio. The results showed that the hot-rolled profile had a nominal control compressive capacity of 162,743 kgf and a numerical control ultimate capacity of 193,015 kgf, higher than the welded built-up profile of 158,710 kgf and 189,465 kgf. The hot-rolled profile also showed better stability with a buckling factor of 1.97455, compared to the welded built-up profile of 1.92217. Based on structural and economic evaluations, the hot-rolled H-Beam profile showed more optimal performance than the welded built-up profile under the conditions of this study

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INTRODUCTION

In the modern construction industry, the use of steel as a primary structural material is increasing in line with the need for strong, efficient, and long-lasting buildings. According to data from the World Steel Association (2024), global steel consumption reached more than 1.88 billion tons per year, with the construction sector contributing approximately 50% of this total consumption. In Indonesia, the demand for structural steel is projected to continue growing by an average of 6–8% per year until 2030, particularly for large infrastructure projects such as high-rise buildings and bridges (Home - worldsteel.org, n.d.).

Along with the increase in construction volume, the industry's focus is also shifting toward faster, more economical, and more flexible construction methods. This has triggered an increase in the adoption of pre-engineered building (PEB) systems and fabricated components (Steel Construction Trends in Indonesia and the World in 2024 | MASTER TUKANG, n.d.).

One of the most common types of steel profiles used in column and beam construction is the H-Beam profile. This profile has characteristics of high flexural strength and cross-sectional efficiency against axial compressive loads. There are two main types of H-beams in widespread use: hot-rolled (prefabricated) and welded (assembled). These two types differ fundamentally in terms of manufacturing process, mechanical quality, and production costs.

The choice between these two profile types is not only a technical design decision but also a crucial economic one. Hot-rolled profiles offer advantages in terms of standardization, assurance of uniform material quality from the factory, and speed of field installation. On the other hand, welded profiles (often referred to as built-up profiles) offer almost unlimited design flexibility, allowing engineers to precisely optimize cross-sections according to the applied loads, potentially resulting in lighter and more material-efficient structures (Hashim et al., 2024).

Globally, the use of welded H-beams is increasing in projects requiring special dimensions or large sizes not available in prefabricated hot-rolled products. However, concerns arise regarding production costs, the reliability of welded joints, and structural stability, especially under dynamic loads such as earthquakes (Shen et al., 2025).

The main problem arises from the welding fabrication process itself. The intense and non-uniform heat input during welding, followed by cooling, induces significant residual stresses within the cross-section (Tu et al., 2025). These residual stresses, particularly compressive stresses at the flange ends, can drastically reduce the axial compressive capacity of the column and trigger premature buckling, long before the material reaches its full yield strength. Furthermore, the welding process produces residual stresses and thermal distortions that affect the strength and stiffness of the structure, particularly at column and beam joints (H. Chang et al., 2021).

From an economic perspective, the comparison is also not simple. While welded profiles may use less steel volume (lighter), the fabrication process, including plate cutting, welding, and inspection, requires more activities, time, and skilled labor (Hashim et al., 2024). Furthermore, fluctuations in the price of steel plate as the raw material for welded profiles compared to the price of finished hot-rolled profiles, are critical variables affecting the total project cost.

Another issue is the imbalance between cost efficiency and structural performance. Although welded H-beams are generally less expensive initially, long-term maintenance costs due to corrosion or weld failure can be higher than hot-rolled H-beams, which have better material homogeneity. This situation necessitates further research into fabrication cost analysis during the early planning stages.

In modern structural analysis, numerical modeling based on the Finite Element Method (FEM) has become the primary approach for evaluating the performance of steel structural elements. One widely used software globally is SAP2000, developed by Computers and Structures, Inc. (CSI). This program is highly capable of analyzing nonlinear responses, buckling behavior, and stress distribution in steel elements with various connection types, including welded joints and hot-rolled structures.

Several recent studies have emphasized the importance of numerical simulation in evaluating the strength and stability of steel columns. (W. Qiao et al., 2025) demonstrated that finite element analysis can predict the plastic and nonlinear behavior of composite steel columns with greater than 90% accuracy compared to experimental tests. This makes SAP2000 a very effective tool for analyzing column response to compressive loads and bending moments, especially for welded profiles that have geometric imperfections.

In the context of this research, the use of SAP2000 will enable an objective quantitative and comparative analysis between two types of steel columns. The results of this analysis are used to evaluate the differences in strength capacity and buckling behavior between welded and hot-rolled columns as a basis for assessing the structural performance of each profile. The numerical model can predict differences in deformation, stiffness, and compressive strength between welded and hot-rolled columns with varying column height and slenderness ratio. This is consistent with the results of (X. Zhao et al., 2025), which found that FEM simulations were able to replicate experimental test results with a deviation of less than 8%.

Many field engineers may use SAP2000 by modeling welded profiles as ordinary custom sections, but forget or are unaware of how to adjust the buckling parameters (buckling curves) to reflect the reduced capacity resulting from the fabrication process. They may mistakenly apply the more optimistic hot-rolled buckling curve to welded profiles, potentially resulting in an unsafe design. This research is crucial for precisely quantifying the magnitude of the strength differences when modeled correctly and incorrectly in SAP2000.

In addition to analyzing structural strength, this study also integrates SAP2000 simulation results with material and fabrication cost analysis to compare the efficiency of the two types of steel column profiles from technical and economic perspectives. This integrative approach aligns with current research trends that assess steel material efficiency not only from a mechanical perspective but also from an economic and construction sustainability perspective (T. D. Gemechu & Lu, 2024).

Through the application of SAP2000, this research not only contributes to the understanding of the mechanical behavior of welded steel columns but also provides an FEM-based analysis model that can be replicated by practitioners and academics in the local Indonesian context. The results are expected to strengthen the scientific basis for economical and high-durability steel column design recommendations, supporting the development of national standards (SNI) that are more adaptable to modern fabrication technologies.

THEORETICAL REVIEW

Studies on the behavior of steel columns indicate that the compressive capacity and stability are significantly influenced by residual stresses resulting from the fabrication process. (Yuan et al., 2014) showed that residual stresses in welded profiles can reach up to 80% of the material's yield stress, significantly reducing the column's compressive capacity.

Research by (Tuezney et al., 2021) showed that welded-fabricated steel columns experienced a 15% reduction in buckling capacity compared to standard predictions due to the non-uniform distribution of residual stresses. Furthermore, (X. Chang & He, 2021) demonstrated that thermal distortion due to welding can affect column geometric stability and accelerate buckling.

Thus, the behavioral characteristics of welded columns differ significantly from those of hot-rolled profiles, necessitating a more realistic analytical approach.

The finite element method (FEM) has been widely used to numerically analyze the behavior of steel columns. (H. Zhao & Chen, 2025) demonstrated that nonlinear analysis using FEM can predict column capacity with greater than 90% accuracy compared to experimental results.

Research by (W. Qiao et al., 2025) also demonstrated that FEM can capture the plastic and nonlinear behavior of steel columns with deviations of less than 10%. Furthermore, Lin & Far (n.d.) demonstrated that post-buckling analysis using FEM yielded results highly consistent with experimental data. The development of advanced methods such as the Continuous Strength Method (CSM) has also been shown to improve the accuracy of steel column capacity predictions, with error rates below 5% compared to conventional methods (Afshan & Rossi, 2021).

In Finite Element Method (FEM)-based modeling, structural conditions are generally assumed to be ideal, including material uniformity, precise element dimensions, and perfect loading conditions. Meanwhile, actual field conditions of structures generally involve geometric imperfections, material variations, and fabrication influences that can cause differences in response between numerical analysis results and the structure's actual behavior.

(Indriyantho et al., 2024) state that numerical models are essentially built based on the assumption of ideal conditions, while actual structures have imperfections and material non-uniformities that can affect the structure's load-displacement response. Therefore, a numerical model validation approach is necessary so that the analysis results can more realistically represent the structure's behavioral tendencies.

(Indriyantho et al., 2024) also explain that the structure's deformation response through the load-displacement curve is a critical parameter in evaluating the suitability of the structure's behavior from numerical analysis to actual conditions. This approach is used to review the initial stiffness, ultimate capacity, and deformation tendencies of the structure during the loading process.

Based on the study, FEM-based nonlinear analysis in this study was used to evaluate the compressive behavior of welded built-up and hot-rolled steel columns by considering the influence of initial imperfections, residual stress, and geometric nonlinearity on the buckling response and ultimate capacity of the structure.

METHODOLOGY

This research is a quantitative study using a numerical analysis approach based on the Finite Element Method (FEM) combined with a case study. The quantitative approach was used to obtain a measurable relationship between structural capacity and steel column fabrication costs, allowing for an objective comparative analysis between welded-built-up and hot-rolled profiles.

The FEM approach was chosen because it can more realistically represent the behavior of steel structures, particularly in capturing nonlinear phenomena such as buckling, slenderness effects, and stress distribution in structural elements. The analysis was conducted using SAP2000 software to obtain the compressive capacity and behavior of the steel columns, with boundary conditions adjusted to the actual conditions of the structure in the field.

This study employed a case study method on the J&T Solo Gateway project, focusing on the main warehouse structure that uses welded-built-up H-beam steel columns. This case study was chosen to represent real-world conditions, ensuring that analysis parameters such as cross-sectional dimensions, boundary conditions, and material properties reflect the actual project conditions.

The independent variables in this study were the steel column profiles: welded-built-up and hot-rolled. The dependent variables include column pressure capacity (Pn) and fabrication costs (C).

The subject of this study is a welded H-Beam steel column element (welded built-up) used in the main structure of the J&T Solo Gateway project warehouse building. The column under review has a total height of 13 meters, consisting of a 1-meter-high pedestal column and a 12-meter-high main steel column. The analysis focuses on the main steel column with an effective length of 12 meters.

The unit of analysis is a single steel column subjected to axial compressive loads. Parameters used in the analysis include the column's cross-sectional dimensions (HB 300×300×10×15), column length, material properties, and boundary conditions assumed to reflect actual field conditions. Modeling was performed by considering the actual boundary conditions of the structure, namely the fixed support at the base of the column and the hinge support at the top of the column. This approach was used because the base of the column is connected to the concrete pedestal via a base plate and anchor bolts, while the top of the column is connected to the roof truss element, which still allows for rotation.

The steel used in the welded columns was obtained from steel plate fabrication. Therefore, the material characteristics in this study were based on tensile test results of the steel plate used as the profile component. The tensile test data were used as input material parameters in the numerical modeling. The analysis focused on the compressive capacity, lateral deformation, and buckling behavior of the steel columns in two main directions. The X-direction represents the longitudinal direction of the building, while the Y-direction represents the span direction of the portal. Both directions were analyzed to determine the buckling conditions most critical to column stability.

As a comparison, hot-rolled H-beam steel columns were used. They were not directly observed in the field but were numerically modeled using SAP2000 software with cross-sectional dimensions and boundary conditions comparable to those of welded columns. This approach aimed to obtain an objective comparison of structural performance by controlling key variables. The study focused on the axial compressive and buckling behavior of the steel columns, without considering the contribution of the pedestal.

Therefore, this study used a Finite Element Method (FEM)-based numerical analysis approach to evaluate the differences in compressive capacity and cost efficiency between welded and hot-rolled steel columns, with consistently controlled parameters.

Data collection in this study was conducted through several stages tailored to the needs of the structural analysis and cost analysis. The data collected included primary and secondary data obtained from material testing, field observations, and cost surveys. The data collection procedure was carried out through the following stages:

Material Data Collection

Material data was obtained through tensile tests on welded steel plates conducted at the Materials Laboratory of Sebelas Maret University (UNS) in Surakarta. The resulting data included yield strength (Fy), ultimate tensile strength (fu), and material strain. This data was used as material input parameters in numerical modeling based on the Finite Element Method (FEM).

Welded Joint Quality Data Collection

Welded joint quality data was obtained through Non-Destructive Testing (NDT) results on steel column elements. This testing aims to identify weld defects such as cracks, porosity, or incomplete fusion that could affect structural performance. NDT data was used to validate the assumption that the welded joints were in good condition and free of significant defects in the FEM modeling.

Fabrication and Erection Data Collection

Fabrication and erection data were obtained through field documentation, covering the plate cutting, assembly, welding, and column installation processes at the project site. This data was used to verify the conformity of actual field conditions with the assumptions used in the structural modeling, particularly regarding fabrication methods and boundary conditions.

Column Geometric Data Collection (Verticality)

Column geometric data was obtained by measuring the column's verticality after the erection process using a total station. This measurement aimed to evaluate the column's inclination relative to the ideal vertical condition and ensure that any deviations remained within the tolerance limits set by the Indonesian National Standard (SNI). Measurements were also conducted to determine the column's actual deviation from the ideal vertical position in two main directions: the X and Y directions. This data was used to evaluate the column's installation conformity to the tolerance limits and as a basis for determining the initial imperfection value in the SAP2000 nonlinear model. This data was used to verify the column's actual geometric conditions in the field, which can affect structural behavior, particularly in column stability analysis. Based on the measurement results, the maximum deviation values in the X and Y directions are used to represent the initial geometric imperfections of the column. Thus, the initial imperfection values used in the modeling are not only based on a theoretical approach but also refer to the actual conditions of the field measurements.

Data analysis in this study was conducted through a numerical approach using SAP2000 software based on the Finite Element Method (FEM) as the primary approach, combined with theoretical calculations based on SNI 1729:2020 for validation purposes. This approach was chosen because the behavior of steel columns, particularly under axial compression, is significantly influenced by instability phenomena that cannot be accurately represented through linear analysis.

The nonlinear analysis applied in this study encompasses geometric nonlinearities (P-Delta effects and large deformations), material nonlinearities (elastic-plastic behavior), initial imperfections, and residual stress. These four aspects are modeled simultaneously to capture the column's actual behavior until it reaches failure.

The analysis was conducted in stages using the incremental loading method, where the axial load is gradually increased until instability (buckling) is reached.

This stage aims to obtain more realistic results than linear analysis, particularly in evaluating compressive capacity, buckling behavior, and the effect of profile type on structural efficiency.

Steel Column Strength Analysis

Steel column strength analysis was performed using SAP2000 software using the Finite Element Method (FEM) approach. Column modeling was performed as a frame element, taking into account material properties, cross-sectional geometry, and boundary conditions consistent with actual field conditions.

Material parameters for welded H-beam columns were obtained from steel plate tensile tests, while for hot-rolled H-beam columns, standard steel grade data was used. Modeling was performed under the assumption of elastic-plastic material behavior. Boundary conditions were modeled according to the actual conditions of the column, namely the placement of the clamp at the base of the column, and the top of the column was modeled as a hinged support because it is connected to the roof frame element, which still allows rotation. The applied load was axial compression, applied incrementally.

The analysis was conducted using a nonlinear analysis approach, which includes:

Geometric Nonlinearity

Geometric nonlinearity is considered to capture changes in structural configuration due to significant deformation during loading. The effects considered include:

P-Delta effect (second-order effect)

This effect occurs due to the interaction between axial compressive forces and lateral displacements, resulting in additional moments that can accelerate structural instability.

Large displacement (geometric nonlinearity)

Considers structural shape changes large enough to invalidate the initial geometric assumptions. This is important in buckling analysis because large deformations will affect the distribution of internal forces.

In the SAP2000 implementation, geometric nonlinearity is enabled through the P-Delta option and the large displacement formulation. This allows the analysis to accurately capture the phenomenon of global column instability.

Material Nonlinearity

Material nonlinearity is used to describe the behavior of steel materials, which are not only elastic but also undergo plasticity after reaching yield stress. The material is modeled using an elastic-plastic approach (bilinear stress-strain model).

Elastic stage: follows Hooke's law

Plastic stage: after reaching F_y, the material undergoes plastic deformation with reduced stiffness.

This approach allows for the simulation of:

Stress redistribution

Structural stiffness reduction

Post-yield behavior

Therefore, the analysis results are more representative of real-world conditions than the linear elastic assumption.

Initial Imperfections

Initial geometric imperfections are incorporated into the model to represent the actual condition of columns that are not geometrically straight. The presence of initial imperfections affects the nonlinear response and buckling behavior of steel columns, thus making the structural modeling more representative of field conditions. Without considering initial imperfections, Finite Element Method (FEM) models tend to overestimate structural capacity.

In this study, the initial imperfection values are not only assumed theoretically but also take into account the results of field measurements of the verticality of the fabricated columns. The verticality data is used to represent the actual initial geometric deviations that occur in the column elements before they receive loading. This approach is taken to ensure that the initial conditions of the numerical model more closely align with the actual conditions resulting from the fabrication and erection of the steel structure.

Theoretically, the initial imperfection limit refers to the approach:

$$\delta_0 = L/1000$$

Where:
δ_0 = initial imperfection amplitude
L = column length

Imperfections are modeled based on the first buckling mode (eigenbuckling mode) so that the initial deformation follows the most critical buckling pattern in the column element.

The initial imperfection value in the numerical model is determined by considering the verticality measurements of the fabricated columns as a representation of the initial geometric imperfections of the structural element. To evaluate the influence of the direction of geometric imperfections on buckling behavior, the analysis was conducted in two scenarios: initial imperfections in the X direction and initial imperfections in the Y direction. Both scenarios were used to obtain the buckling response of the columns in the longitudinal direction of the building and the portal span direction. This approach aims to improve the agreement between actual field conditions and the geometric assumptions in nonlinear FEM modeling.

Residual Stress

Residual stress is the internal stress remaining in a structural element due to the fabrication process, particularly in built-up welded profiles due to the welding process. In this study, the effect of residual stress in SAP2000 modeling was not directly modeled as an actual stress distribution due to the limitations of nonlinear material modeling in the software.

Therefore, the effect of residual stress is represented using an approach assuming a reduction in yield stress in certain cross-sectional zones based on the literature, namely:
At the flange: Tensile and compressive stresses up to approximately $\pm 0.3\ F_y$
At the web: Stress approximately $\pm 0.1\ F_y$

In hot-rolled profiles, residual stress is assumed to be smaller and more evenly distributed. The effect of residual stress on structural behavior can reduce the effective compressive capacity of the column, accelerate local plasticity, and influence the buckling mode and critical load. By incorporating residual stress, the analysis can more realistically differentiate between the performance of welded and hot-rolled columns.

Nonlinear analysis is performed in stages until the structural failure condition is reached, characterized by:

1. Significant lateral deformation
2. Decrease in structural stiffness
3. Failure to achieve convergence of the numerical solution
4. This condition is interpreted as buckling or structural instability.

The nominal compressive capacity of the column is calculated based on the provisions of SNI 1729:2020, which considers the influence of the column slenderness ratio, material properties, and instability (buckling) phenomena.

This value represents the theoretical capacity of the column, which is used as a reference in structural design, as follows:

$$P_n=F_{cr}\cdot A_g$$

Where:
P_n = Nominal compressive capacity
F_{cr} = Critical column stress
A_g = Column cross-sectional area.

This value is then compared with the results of the numerical analysis to evaluate the consistency and conservatism of the method used.

RESULTS AND DISCUSSION

The highest numerical ultimate capacity was obtained in the H-Y model at 194,430 kgf, while the lowest ultimate capacity was obtained in the W-Y model at 189,465 kgf. The difference in ultimate capacity between models indicates that both the profile type and the direction of the initial geometric imperfection influence the column's ability to withstand axial compressive loads before reaching instability.

In terms of maximum displacement values, the H-X model showed the largest lateral deformation of 27.25 cm, while the H-Y model showed the smallest lateral deformation of 23.60 cm. These results indicate that the maximum displacement value is not always directly proportional to the structure's ultimate capacity. Under some conditions, the structure is still able to withstand increased loads despite experiencing relatively large lateral deformation due to geometric nonlinearities.

The influence of the direction of the initial geometric imperfection is evident in both profile types. In the built-up welded profile, changing the direction of the imperfection from the X-axis to the Y-axis causes the ultimate capacity to decrease from 190,555 kgf to 189,465 kgf and increases the maximum displacement from 23.65 cm to 25.31 cm. Meanwhile, in the hot-rolled profile, the Y-direction imperfection produces a higher ultimate capacity than the X-direction, but with a smaller maximum displacement.

This difference indicates that the orientation of the initial geometric imperfection influences the development of lateral deformation and the structure's sensitivity to the P-Delta effect during the loading process. Thus, the direction of the initial geometric imperfection not only determines the dominant direction of deformation but also influences the numerical ultimate capacity obtained from the nonlinear analysis.

Overall, the load-displacement relationship curves indicate that the nonlinear response of the column is influenced by a combination of cross-sectional characteristics, geometric nonlinearities, and the direction of the initial geometric imperfection. The ultimate capacity and maximum displacement data obtained at this stage were then used as the basis for evaluating the nonlinear deformation patterns and global buckling mechanisms in each model.

Based on the results of data extraction and processing from SAP2000, the main parameters used in the evaluation of column behavior include the number of load steps successfully achieved, maximum lateral displacement, ultimate numerical capacity (ultimate load capacity), and the buckling pattern formed in conditions approaching failure.

All models successfully converged during the nonlinear analysis process with 69 to 71 load steps. These results indicate that the analysis parameters used were capable of producing stable numerical solutions, allowing the obtained structural responses to be used for further evaluation.

In terms of numerical ultimate capacity, the H-Y model produced the highest capacity of 194,430 kgf, while the W-Y model produced the lowest capacity of 189,465 kgf. The difference in ultimate capacity between the two models was approximately 2.62%, indicating the influence of the profile type and the direction of the initial geometric imperfection on the column's ability to withstand axial compressive loads.

In the Welded Built-Up profile, changing the direction of the initial geometric imperfection from the X-axis to the Y-axis caused the ultimate capacity to decrease from 190,555 kgf to 189,465 kgf. Conversely, in the hot-rolled profile, changing the direction of the imperfection from the X-axis to the Y-axis increased the ultimate capacity from 193,015 kgf to 194,430 kgf. These results indicate that the orientation of the initial geometric imperfection influences the development of the nonlinear response and the structure's sensitivity to global instability.

In terms of maximum lateral displacement, the H-X model produced the largest deformation of 27.25 cm, while the H-Y model produced the smallest deformation of 23.60 cm. This difference indicates that a higher ultimate capacity is not always accompanied by a larger or smaller maximum displacement. The deformation response is the result of the interaction between the structure's global stiffness, the influence of geometric nonlinearities, and the direction of the initial geometric imperfection applied to the model.

All models exhibited a failure pattern of global flexural buckling, characterized by dominant lateral deformation along the column height. These results are consistent with the first buckling mode obtained in the eigenvalue buckling analysis in Section 4.4.7. Therefore, it can be concluded that the instability mechanism developed in the nonlinear analysis remains within the scope of the column's global buckling behavior.

In general, the hot-rolled profile exhibited a slightly higher ultimate capacity than the welded-built profile under the same loading configuration and boundary conditions. However, the direction of the initial geometric imperfection was shown to significantly vary the response to both the ultimate capacity and the maximum lateral deformation. Therefore, both the profile type and the orientation of the initial geometric imperfection need to be considered in evaluating the buckling behavior of slender compression steel columns.

Based on the results of the nonlinear static analysis performed on all column models, the next step is to determine the ultimate numerical capacity (ultimate load capacity, Pu). The ultimate numerical capacity is defined as the maximum axial load that a structure can withstand before a significant increase in lateral deformation occurs due to the development of global column instability.

The Pu value is determined based on the load-displacement relationship curve from the SAP2000 nonlinear analysis. In this study, the Pu value is not determined based on the final load that can be calculated by the program, but rather at the point of maximum loading before a sharp change in the curve's slope occurs, characterized by a large increase in lateral displacement due to a relatively small load increase.

This approach is used because in the post-critical condition (post-buckling region), the numerical model can still produce convergent solutions, but mechanically, the structure has experienced a significant decrease in stiffness and no longer represents the effective compressive capacity of the column. Therefore, the Pu value used in this study is considered the limit of the structure's ability to maintain global stability before the development of a dominant buckling mechanism.

To maintain consistency with the determination of the nominal control compressive capacity and the ultimate numerical control capacity, further evaluation was conducted using the control capacity values for each profile.

The control capacity was defined as the smallest capacity value obtained from all variations in the initial geometric imperfection direction within the same profile. This approach was used to represent the most critical conditions for the development of global column instability, resulting in a more conservative evaluation.

For the welded built-up profile, the control capacity was obtained from the W-Y model with Pn values of 158,710 kgf and Pu of 189,465 kgf. Meanwhile, for the hot-rolled profile, the control capacity was obtained from the H-X model with Pn values of 162,743 kgf and Pu of 193,015 kgf.

Both profiles produced higher numerical ultimate capacity values than the nominal compressive capacity calculated based on SNI 1729:2020. This indicates that the design approach used in the standard still maintains a degree of conservatism regarding the actual structural behavior obtained through geometric nonlinear analysis.

Based on profile type, the hot-rolled profile produced a numerical ultimate control capacity of 193,015 kgf, higher than the welded built-up profile, which was 189,465 kgf. These results are consistent with the previous discussion regarding the effect of residual stress distribution on the reduction of column stiffness and global stability.

It can be seen that both profiles experienced an increase in capacity from theoretical conditions (Pn) to the numerical ultimate capacity (Pu). Although both profiles showed a similar trend, the Hot-rolled profile still produced a higher ultimate capacity than the Welded built-up profile. These results indicate that cross-sectional characteristics and internal stress distribution influence the structure's ability to maintain global stability under loading conditions approaching the ultimate capacity.

Both profiles produced relatively similar Pu/Pn ratios, namely 1.194 for the welded built-up profile and 1.186 for the hot-rolled profile. The difference in these ratios is relatively small, less than 1%, indicating that both profiles have comparable levels of compressive capacity efficiency when evaluated using their respective control conditions.

A ratio value greater than one indicates that the numerical ultimate capacity obtained through nonlinear analysis is still above the nominal capacity calculated according to SNI 1729:2020. This condition indicates that the design approach in the standard provides an adequate level of conservatism regarding the actual behavior of steel column structures.

Overall, the comparison of Pn and Pu results indicates that the hot-rolled profile has a slightly higher ultimate capacity than the welded built-up profile. However, the effect of profile type on the Pu/Pn ratio is relatively small compared to the influence of initial geometric imperfections on the development of lateral deformation and structural stability behavior, discussed in the previous subsection.

The evaluation results in this sub-chapter are then used as a basis for the analysis of structural efficiency and fabrication costs in the next chapter to determine the most effective alternative steel column profiles in terms of compressive capacity, structural stability, and cost efficiency.

CONCLUSION AND RECOMMENDATION

Based on the theoretical analysis results based on SNI 1729:2020, nonlinear analysis using SAP2000, and evaluation of fabrication costs and economic efficiency of built-up and hot-rolled welded steel column profiles, the following conclusions were obtained:

Based on the nominal compressive capacity calculation according to SNI 1729:2020, the hot-rolled profile produced a nominal control compressive capacity of 162,743 kgf, while the built-up welded profile produced 158,710 kgf. These results indicate that the hot-rolled profile has a slightly higher theoretical compressive capacity under the analyzed cross-sectional conditions and structural configuration.

The nonlinear analysis results using SAP2000 indicate that the hot-rolled profile has a numerically controlled ultimate capacity of 193,015 kgf, higher than the 189,465 kgf for the built-up welded profile. These findings indicate that the hot-rolled profile is capable of maintaining a greater structural capacity even near global buckling conditions.

The buckling eigenvalue analysis results show that the hot-rolled profile has a first-mode buckling factor of 1.97455, while the welded-built profile has a buckling factor of 1.92217. These values indicate that the hot-rolled profile has a better level of elastic stability against potential global buckling than the welded-built profile.

A comparison of the theoretical nominal compressive capacity (Pn) and the numerical ultimate capacity (Pu) shows that both profiles produce Pu/Pn ratios greater than one. The welded-built profile produces a ratio of 1.194, while the hot-rolled profile produces a ratio of 1.186. These results indicate that the actual capacity of the structure obtained through nonlinear analysis is still above the nominal capacity calculated according to SNI 1729:2020. Therefore, the design approach in the standard maintains a certain degree of conservatism regarding the structure's actual behavior.

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