

The Slippage of Bolts on Axial Compression Capacity of Equal Angle Section's Bolt Connection

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Abstract—Steel angles play a vital role in energy infrastructure, especially in latticed transmission towers where they act as axial compression members. Their structural performance is strongly influenced by end restraints, bolt configurations, and slenderness ratios, which affect buckling behavior and compression strength. This study examines how the number and arrangement of bolts, along with slenderness variations, impact the displacement and stress response of steel angles under compression using nonlinear analysis and finite element modeling. The investigation highlights that different bolt patterns and slenderness ratios significantly alter the stability and load-bearing capacity of the members. Single-row bolt configurations tend to experience greater slippage compared to multi-row setups, indicating the importance of connection design in preventing premature failures. Overall, the findings emphasize the need for optimized bolt arrangements and slenderness control to enhance resistance to buckling and improve the overall efficiency and reliability of energy infrastructure systems under axial compression loads.

Keywords: Axial compression, Buckling, Nonlinear analysis, Bolts, and Energy infrastructure.

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1. Introduction

The lattice steel tower is known extensively for its usage as the core element of an electrical power transmission line. These structures are typically constructed using equal-leg angle section members connected via bolted joints [1], [2]. Moreover, an overhead transmission line plays a crucial part in maximizing a dependable electrical power system's operation. When it comes to the loads on the tower, like the weight of the conductors, gravity, and weather, the force pushing down on the main part of the tower's lower levels can be very high, which means larger built-up parts need to be used in that area [3], [4]. Two, three, or four steel angle parts are linked corner-to-corner to make the corresponding parts while considering the degree of axial force acting internally on the parts [5]. The axial force in the main legs decreases at higher locations on the tower, which leads to a reduction in the cross-section of the respective members. Reduction in the cross-section area for most of the legs is obtained by increasing the quantity of steel angle sections within the complete cross-section. Contrarily, at a different location, the complicated geometry around the connection region contributes to eccentricity encompassed by the load path and indirect load transfer across several components, where the number of angle sections within the built-in cross-section changes. As a result of the load path's complicated configurations and ambiguities, the connection is over-designed, resulting in an increased number of bolts and redundant reinforcement connections [6].

A good prediction of tower behavior is essential to ensure an optimal design capable of providing uninterrupted service [1]. Naturally, a tower failure involving multiple adjacent towers along the line occurs when such a failure arises. The need for repair is inevitably costly, disregarding the costs linked to dispute and litigation. Tower collapses are occurring frequently around the world; however, these events are more likely to happen in isolated areas where there are no human casualties. Researchers conducted recent work to better understand the load-flow cycle at specific locations where the cross-

sections of the major leg members differ [7]. Numerical experiments were performed with finite element models, beam elements, and two-dimensional shell and plate elements, or in one dimension [2], [3], [8], [9]. The study also examined the eccentricity of the joints and their flexibility. In most cases, the results of the models were compared to those of a full-scale test load or a small laboratory experiment tower test. These studies showed that we can get a good estimate of load efficiency by using numerical models, which means the method can accurately predict where a member might fail.

1.1 Current Analytical Techniques

Using current analysis methods, a linear elastic truss analysis measures stress in lattice transmission towers by assuming the members only experience tension and compression forces, and are typically connected with pinned joints. In practice, such circumstances are oversimplified as the member's internal actions are assumed to be fixated on tension and compression forces. It was realized that the ideal truss analysis model results could not be relied upon to predict the behavior of transmission towers under complex loading. It was reported in [10] that the study resulted in real-life tests on transmission lattice towers, which showed that the actual bending was up to three times more than what the theory predicted for linear elastic deflections. Safety aspects in member and connection design have compensated for the gap between experimental results and analytical answers. However, to estimate true failure modes and tower capacity under ultimate loads, a more extensive analysis is required.

Recent literature has concentrated on the topic of failures in transmission line towers. There were three structural collapses at the Tower Testing and Research Station in Chennai, including leg, bracing, redundancy failures, detailing, and connection failures [11]. Buckling compression bracing and leg members lead to numerous failures. Recent research has also looked into the impact of bolt-slip on lattice tower non-linear behavior [3, 4]. According to these research findings, the joint slip may affect tower displacement, but it has no major impact on the tower's failure mode or ultimate strength. The past paper focused on a full-scale transmission tower analysis. In contrast, this paper will focus on the compression capacity of a segmented steel angle, considering the number of bolts.

An approach to propose a non-linear analytical technique was discussed in [13]. Thin-walled structures elastoplastic big deformation analysis was studied, which presented numerical analysis examples that provided information about elements of thin-walled structures considered during further numerical analysis and resulted in different ultimate collapse load factors. Both geometric and material nonlinearity are taken into account by the approach [14]. Additionally, it was discovered that the analysis could incorporate other non-linear effects caused by bolt sliding [15]. However, the design assumption of this software applied the direct stiffness method, which utilizes a shell plate, a form of thin plate, and is therefore subject to buckling. Therefore, this paper will use finite element modeling analysis on a single steel angle, incorporating the non-linear incremental method analysis process, where loads are applied in small increments, similar to that in [14].

1.2 Effect of Bolt Slippage

The next section discusses the effect of bolt slippage. This research considers bolt slippage to accurately predict the final response of a lattice structure. Thus, the structure's non-linear factors must be considered in the study. To begin with, the concern about the effect of bolt slippage escalated when an engineer noticed that the real lateral displacement of a tower during full-scale tests is substantially larger than the deflection estimated by structural analysis. Initially, an older study noted that while bolt slippage could affect deflection, it does not compromise the ultimate strength of the lattice structure [16]. The discussion also revealed that allowing bolt slippage leads to a more flexible tower compared to those that don't allow it. The theory suggests that the tension diagonal slips when the compression diagonal experiences increased load.

As a result, if the truss's strength was still limited by diagonal compression buckling, the truss with slipping tension diagonal bolts would have a lower overall buckling strength. Contrarily, the experiment conducted in [17] on common types of bolted joints in towers argued that past studies' models do not accurately simulate bolted joints' behavior. This is because the models did not consider the joint's yielding, which is dependent on the material and joint types. In addition to that, it was found

that joint stiffness properties, such as modulus of elasticity, yield strength, and fracture strength, are much lower than those of the connected members. Thus, due to the inaccuracy of the prediction, models based on the conventional structural analysis software, built on idealized rigid joint behavior, could not predict the unfavorable response of transmission towers. However, the findings showed that different bolted joints have varying stiffness and slipping behaviors, and it was also understood that the way bolts are arranged and the space between them affect how much the joints slip.

Similarly, in a most recent study [12], it was concluded that a well-defined slippage is more likely in joints with a single row of bolts than in joints with a double row of bolts. Secondly, the greater the gap between the bolt-hole diameter and the bolt diameter, the more considerable the apparent slippage. Moreover, the experiment revealed the absence of perfect plastic and the need for a lengthy hardening process before fracture. In short, recent papers indicated that bolted joints greatly influence the axial rigidity of leg members. This raised a question: whether our current study will overcome the problem that loads seemingly exceed the ultimate design load due to the conventional method's analysis, using a different approach to conducting a non-linear finite element method? Additionally, we aim to monitor the impact of the quantity and positioning of bolts on the slippage behavior.

The way bolt slippage is included in a non-linear joint finite element model, which is used to study how transmission towers perform under working loads with the finite element method. The analysis revealed that slippage at the tower-leg joints significantly affected the behavior of the tower [18]. Interestingly, one of the findings, as opposed to the previous study, depicts that the impact has reduced the load-carrying capacity of the tower. Either that or significantly increase the deflections under working load. Therefore, this paper aims to analyze current design practices by running a finite element model portraying non-perfect joint behavior by incorporating non-linear factors concerning the structure.

1.3 Compression Capacity

The third section discusses the compression capacity of a short and a long member. This research utilizes angles of various sizes ranging from 50 x 50 x 5 mm, 65 x 65 x 7 mm, and 75 x 75 x 6 mm to 90 x 90 x 7 mm with different slenderness ratios (L/r) influenced by the member's length starting from 1200 mm up to 2000 mm with the purpose of catering to uncertainties due to the slenderness [5]. As in the last section, the number of bolts and their arrangements will also be a concern. This brings us to the objectives, where our single member aims to support the decision for member selection. Although numerous studies on the strength analysis of a steel angle have identified the benefits of playing around with the slenderness of angles and, most recently, of bolt arrangements, little analytic attention has been paid to utilizing the effect of offset with the number of bolts on the compression capacity of equal-angle steel sections. We will address this statement by demonstrating the methods and approaches used in recent years. Researchers in the past have figured out how much long members ($140 \leq L/r \leq 180$) and short members ($70 \leq L/r \leq 100$) can handle when they are quickly loaded. The result showed that compression capacity increased for long members instead of short members. The justification for the result is that the buckling failure load of long members does not depend on their yield strength. Euler buckling is used to design members with a high slenderness ratio.

Conversely, the yield strength of the member determines its low slenderness ratio. Furthermore, it was opined that as the member sizes increase, the load factor reduces, as the larger the cross-section angle, the greater the inertia. Because of lateral inertial effects, an increase in lateral deflection could result in yielding at a lower test load. The method that has been used to test ranges of a single member of different slenderness ratios will be used in this study, with one of the theories being that the angle would slip into place and bear directly on the offset plate once loaded. Ensure that the applied load travels through the gusset plate and into the angle, thereby harnessing the member's internal capacity and preventing load transfer to the bolts. This would depend on the type of end support at the angle, as discussed in [19]. This study will take a similar approach in depicting a single leg attached at one end of the member with a partial end restraint on the other. While it is true that steel angle buckling or compressive strength is a consequence of several factors, such as the level of restraint supplied at the ends, slenderness ratios, and other parameters that this paper will not be diving deep into, it could be claimed that the nature of end restraint effects in the angle is too multifaceted to evaluate [20]. It was

then agreed that end connection effects are visible for a minimum of single-bolted joints, and end restraints play a critical role for long members. As discussed earlier in a past paper, the buckling capacity of a long member is independent of the yield strength, thereby utilizing the end restraint more to resist buckling. This is also true as [20] concluded that an increase in bolts' quantity decreases the length of effective member buckling.

This paper refers to the ASCE Standard 10 – 15 for the existing tower angle design technique [21]. The process splits the effective slenderness into two groups: short columns ($L/r \leq 120$), which are affected by load eccentricity, and long columns ($120 \leq L/r \leq 250$), which are affected by end restraint. The buckling capacity of a pin-ended column, as indicated in equation (1), and a fixed-end column, where both ends are entirely restrained against translation and rotation, as shown in equation (2), using the Euler formula given below:

$$P_E = \pi^2 EI / (kL)^2 \quad (1)$$

$$P_E = 4\pi^2 EI / L^2 \quad (2)$$

According to several studies, as the number of bolts in the end connections increases, member behavior resembles a fixed-ended column [5], [6]. With the current understanding of slenderness ratios and the number of bolts discussed by past researchers, the study will conduct an analysis using finite element analysis to determine how a single steel angle behaves with offset, different arrangements, and numbers of bolts.

2. Method

In this paper, non-linear buckling analysis was conducted on an equal angle section of a lattice steel member, considering the different numbers of bolts used and offset plates ranging from 0 mm, 10 mm, 15 mm, 20 mm, and 25 mm. The procedure for calculating the compression capacity of the equal angle section is provided in the ASCE Standard 10 – 15 [4], [21]. Figure 1 represents the summary of the compression design procedure [21]. The effective buckling length coefficient, K , and the column slenderness ratio, C_c , distinguish the elastic and inelastic buckling and govern in determining the reduced material strength or the compressive design stress, F_a , in the equal angle section. The end support condition determines the rigidity of the member. The solution method includes numerical methodologies for predicting various types of buckling or instability and determining the non-linear load-deflection path.

In this study, steel angles of grade S355 with dimensions of 50 mm x 50 mm x 5 mm, 65 mm x 65 mm x 7 mm, 75 mm x 75 mm x 6 mm, and 90 mm x 90 mm x 7 mm were chosen to be modeled. Steel angles were attached to an offset plate of 10 mm for every member and an offset plate of 20 mm for 75 mm x 75 mm x 6 mm steel angles. A 16 mm diameter bolt was used, as it was the standard size of bolt used in lattice steel equal angle in bolt connection. The steel angles were analyzed for the different numbers of bolts used. We used non-linear buckling analysis to determine the buckling capacity of several offset plates and the number of bolts utilized. Figure 2 shows the model of steel angle S355 from left to right representing: (a) 50x50x5 1200mm, 2 holes with 10mm offset plate; (b) 50x50x5 1200mm, 3 holes with 10mm offset plate; (c) 65x65x7 1200mm, 2 holes with 10mm offset plate; (d) 65x65x7 1200mm, 3 holes with 10mm offset plate; (e) 75x75x6 1200mm, 2 holes with 20mm offset plate; (f) 75x75x6 1200mm, 3 holes with 20mm offset plate; (g) 90x90x7 1600mm, 2 holes with 10mm offset plate; (h) 90x90x7 1600mm, 3 holes with 10mm offset; (i) 90x90x7 2000mm, 4 holes with 10mm offset plate; and (j) 90x90x7 2000mm, 5 holes with 10mm offset plate. The Autodesk Inventor Nastran software performed the numerical simulation of the modelled steel angles under non-linear buckling analysis.

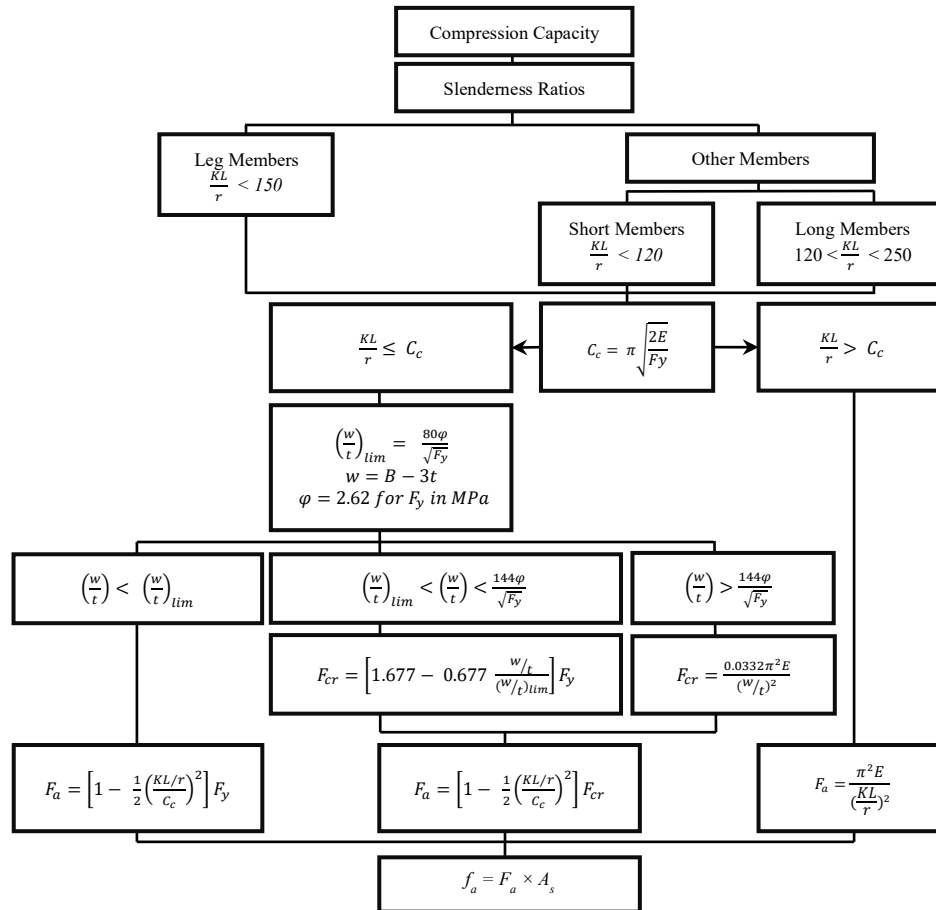


Figure 1. Summary of the compression design procedure.

3. Result and Discussion

The discussion has been divided into three sections: the non-linear buckling analysis, analysis of bolt slippage, and compression capacity. The results and findings were confirmed and supported by earlier research on how steel angles behave when they experience bolt slippage and buckling, which can be found in past journals and articles.

3.1 Non-linear Analysis

Non-linear static analyses were performed on a total of ten models. As a result, the steel angles subjected to the load are similar to each other, as shown in Figure 6. From the analysis, when loadings are applied directly to the top plate, the load is distributed throughout the whole model and produces maximum stress at the middle section of the steel angle. However, Figure 5 shows that the bolt connection area also produces maximum stress. The slenderness ratios (L/r) influence the buckling-induced displacement of the steel angle from its original position. From Table 1 and Figure 3, for $L/r \geq 120$, the displacement was reasonably small when the members reached their maximum stress. This demonstrates that the end restraint-controlled steel angles of size 50x50x5 mm are effective. It can also be observed that for the same member, the member with three bolts has a higher load capacity, as the member only failed at load factor 1. This is because as the buckling restraint increases, it will promote a reduction in buckling behavior and, therefore, the buckling displacement from the steel angle's original position. This was supported by previous studies [20], stating that as the number of bolts in the end connections increases, member behavior approaches that of a fixed-end column. The trend was similar for all the other members, as observed from Table 1. Besides, the result can be related to the slenderness ratio for when $L/r \geq 120$ and when $L/r \leq 120$. The phenomena have been discussed in a review of steel

and end restraints [10-11]. For $L/r \leq 120$ at a load factor < 0.8 , the maximum stress of the member was reached with higher displacement. The displacement values were similar, but Figure 3 shows that the stress–displacement graph of the steel angle's behavior suggests the bolt may have slipped, which we will discuss next. Table 1 shows the effect of the number of bolts used and the comparison between the behavior of different steel angles used when they are subjected to loading.

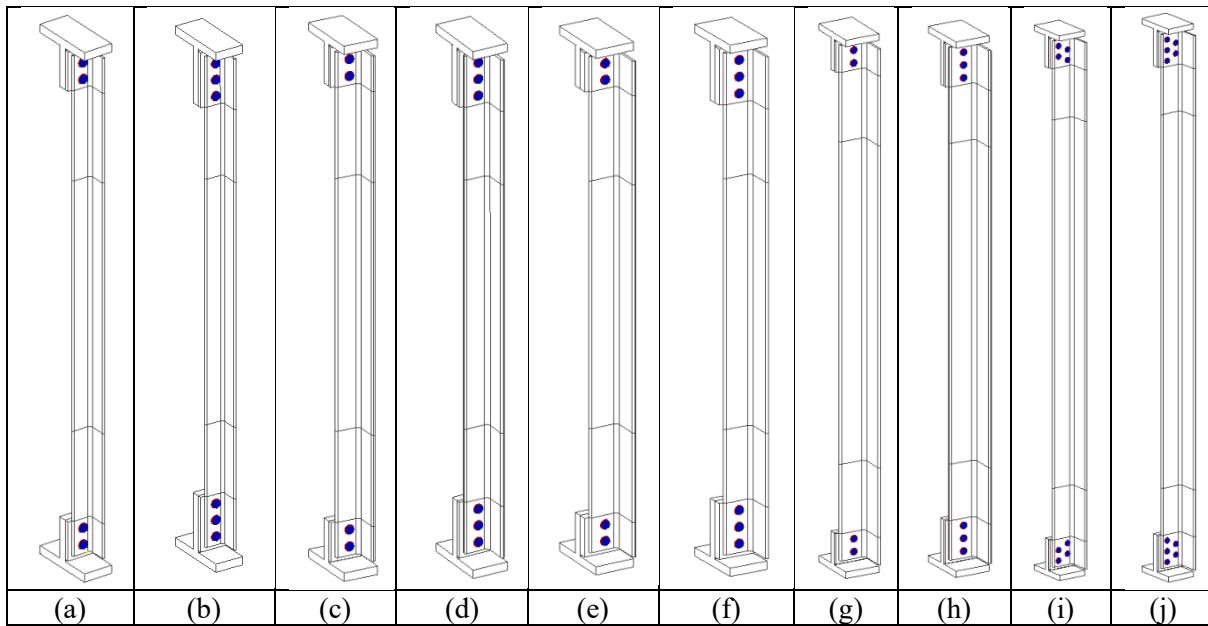


Figure 2. Equal angle steel section of S355 steel grade model used in the study

Table 1. Summary of nonlinear analysis.

Steel angles	Length (mm)	L/r	No. of bolts	Offset (gusset plate, mm)	Design compressive stress, Fa (kN)	Non-linear analysis		
						Load factor	Displacement (mm)	Stress (MPa)
50x50x5	1200	123.33	2	10	63.85	0.8	6.093	355.65
50x50x5	1200	123.33	3	10	63.85	1	7.673	360.419
65x65x7	1200	95.24	2	10	185.27	0.8	8.439	359.316
65x65x7	1200	95.24	3	10	185.27	0.8	7.460	355.524
75x75x6	1200	81.63	2	20	219.32	0.6	8.856	356.915
75x75x6	1200	81.63	3	20	219.32			
90x90x7	1600	90.40	2	10	277.86	0.7	10.457	358.551
90x90x7	1600	90.40	3	10	277.86			
90x90x7	2000	112.99	4	10	193.33	0.8	9.012	357.734
90x90x7	2000	112.99	5	10	193.33	0.8	8.176	359.626

Figure 3 represents displacement–stress behavior of the steel angle member subjected to compression, representing (A) 50x50x5 1200mm, 2 holes with 10mm offset plate; (B) 50x50x5 1200mm, 3 holes with 10mm offset plate; (C) 65x65x7 1200mm, 2 holes with 10mm offset plate; (D) 65x65x7 1200mm, 3 holes with 10mm offset plate; (E) 75x75x6 1200mm, 2 holes with 20mm offset plate; (F) 75x75x6 1200mm, 3 holes with 20mm offset plate; (G) 90x90x7 1600mm, 2 holes with 10mm offset plate; (H) 90x90x7 1600mm, 3 holes with 10mm offset plate; (I) 90x90x7 2000mm, 4 holes with 10mm offset plate; and (J) 90x90x7 2000mm, 5 holes with 10mm offset plate. The non-linear analysis used a step-by-step method with several rounds for each loading cycle to consider how loads and strains change during the cycle, like [6]. The figure shows that for member sizes 50x50x5 and 65x65x7, marked as (A) and (B), the slope in each area was steeper, while for member sizes 75x75x6 and 90x90x7 with

two bolts, marked as (E) and (G), they reached local yield at a lower stress level. This might mean that as the member grows longer and wider, its ability to handle compression depends on how strong the material is; also, having two bolts doesn't act the same way as if the ends were fixed. In relation to members with a low slenderness ratio, lateral inertial effects cause an increase in lateral deflection; as the size of the members grows, so does the inertia of the greater cross-section angle, which, therefore, would result in yielding at a lower test load. However, higher stress was reached for the same member size with three, four, and five bolts before it yielded [19]. This paper analyzes the load-carrying capacity of each member of factors such as the number of bolts and offset plate is not considered; if a big difference is encountered, the design approach may need to be modified.

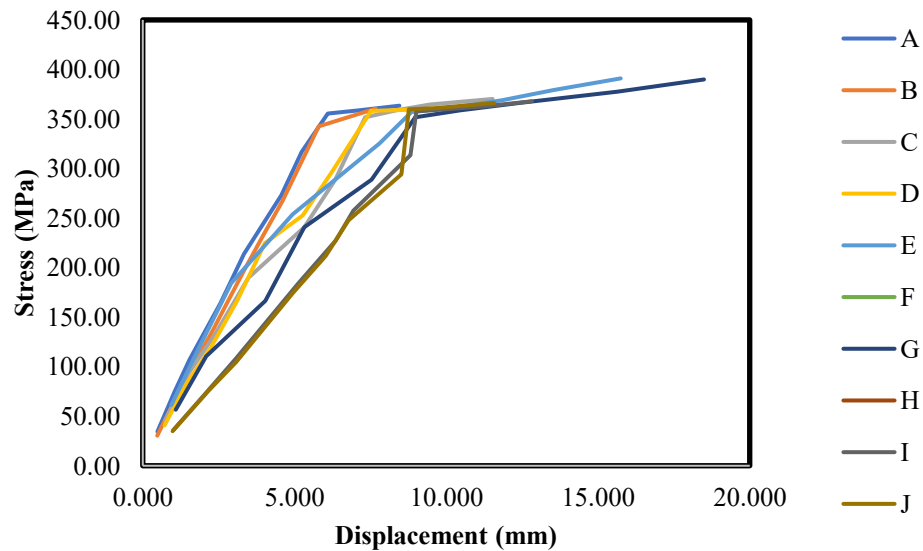


Figure 3. Summary of variations of equal angle single members in compression

The effect of bolt slip, where bolt slip appears to cause additional deformation [22]. Although it was mentioned that it is tough to foresee, and it is impossible to account for in the analysis, from Figure 3, the result analysis agreed with [12], as members with four and five numbers of bolts represented by (I) and (J) are least susceptible to a well-defined slip compared to the single-row arrangement of bolts if we were to analyze the graph carefully on slippage. For instance, take the steel angle of size 90x90x7 for bolt arrangements of (2) two, (3) three, and (4) as illustrated in Figure 4 to visualize the slippage more clearly. The observed slippage in the bolt-connected steel member, as shown in the stress-displacement graph, occurs when the connected parts begin to move relative to each other, typically around 5-10 mm of displacement. This transition from a stable "no slippage" state to a "slippage" phase results in a temporary reduction in stress, reflecting a loss of initial stiffness, followed by an increase as the system adjusts to new frictional conditions. Subsequently, the material enters a prolonged hardening process, where stress continues to rise with increasing displacement, indicating enhanced strength due to deformation, without reaching a uniform stress level prior to fracture. The number of bolts, from 2 to 5, plays a significant role, with a higher number delaying the onset of slippage and supporting greater stress levels thereafter, likely due to increased resistance from additional clamping forces. Likewise, the analyses resulted in no noticeable perfectly plastic, and the angle underwent a long hardening process before fracture; similar results were obtained in [15]. For observation on bolt slippage, refer to Figure 5 where the holes of the bolt begin to move relative to the other parts of the connection system.

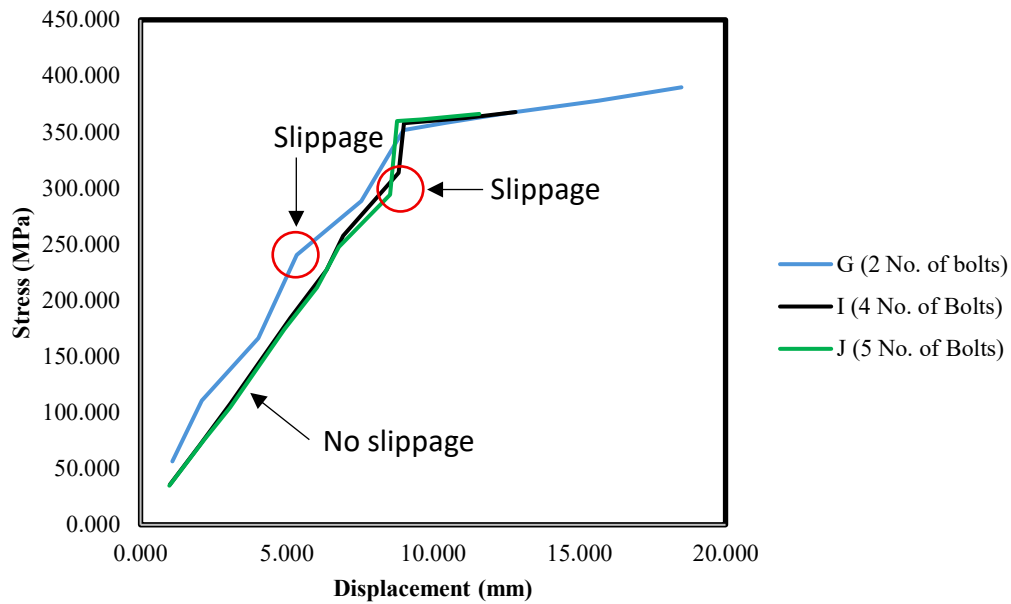


Figure 4. The models marked as G, H and I with the same offset of 10 mm and different numbers of bolts.

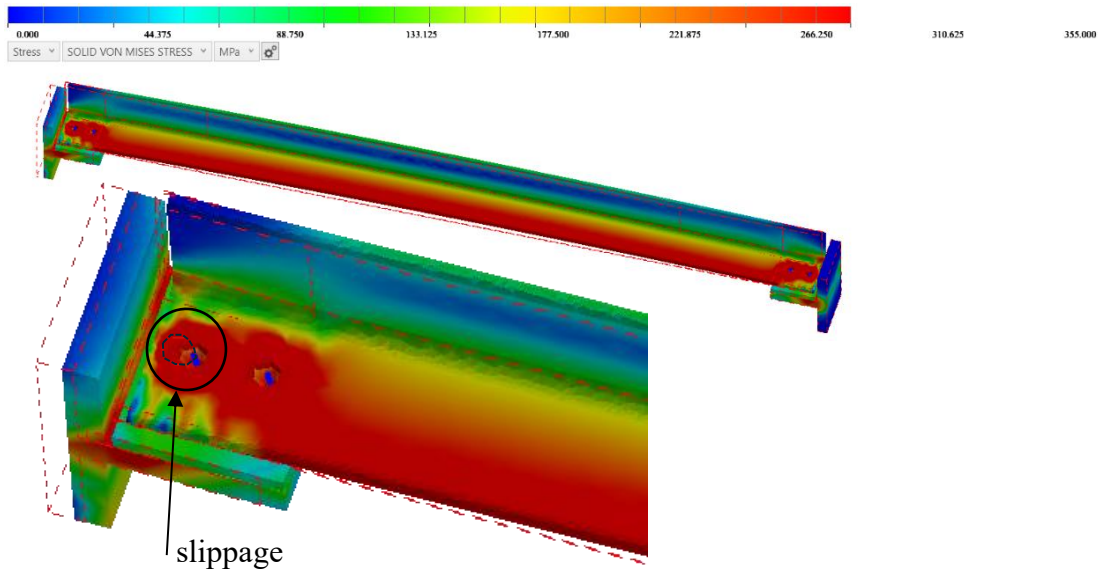


Figure 5. The yield stress in angle section (G) 90x90x7 1600mm, 2 holes with 10mm offset plate showing the slippage of the bolt.

4. Conclusion

The findings of this study reveal that using more bolts increased the rigidity of the steel angle, resulting in less displacement. In addition, the slenderness ratio influences the compression capacity of the member; at high load factor increments, members with high slenderness ratios result in lower displacement and achieve stress failure at the final load increment. Based on the analysis results, a minimum of three bolts may be a reasonable amount to represent an optimized member design. The

compression capacity was not as great for longer members with an increase in members cross-section unless compensated with the end restraint of a fixity. This is reassuring because long members fail due to compression buckling, and increasing the number of bolts utilized reduces the effective buckling length of the member. To better understand the implications of these results, future studies could address the use of shorter members with a minimum of three bolts of 16 mm in diameter arranged with different sizes of offset plates to avoid over-designing the connection, resulting in an unsustainable design due to the need to use a greater number of bolts.

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