Research Article

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Finite element analysis for built-up steel beam with extended plate connected by bolts

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Abstract: Extended end-plate type connections are widely utilized in steel assemblies for easy production and assembly. The proper end-plate and bolt selection is paramount to confirming safety and economy in all connections and, consequently, steel buildings. This study presented an approach for refined parametric three-dimensional finite element analysis of bolted steel beam-to-beam extended end-plate connections. The model considers geometrical and material non-linearity to determine the impact of various factors on the connection's performance. The proposed model was used to construct a parametric study. The study variables were bolt diameter and end-plate thickness, focusing on how thick and thin end plates affect connection conductivity. Numerical outcomes demonstrate that the connection's flexural strength and stiffness capability improved by increasing the end-plate thickness and bolt diameter. Finally, the analysis results were assessed, and the main conclusions were presented.

Keywords: extended end plate, high-strength bolts, connection, weak plate, strong plate, nonlinear finite element

1 Introduction

Recent steel construction practices frequently include moment-resisting frames for steel structures due to the high ductility of these types of structures. The behavior of the steel frame is largely determined by the layout and type of connections. End-plate connections were proposed to overcome the burdens of the traditional way of jointing by welding beam to beam/column. End-plate connections misplace the welding in the field, so the connection can be more ductile and reliable [1].

End-plate connections were used to join two beams (a "spliced joint") or a beam and a column (Figure 1). These connections are made up of steel plates welded to the beam's end section and then joined to the other part of the structure using high-strength bolts. If the connection's steel plate does not extend beyond the beam section, it is known in the literature as a "flush connection," and accordingly, the bolts will be located between flanges. If the details of the connection are such that the end-plate extends beyond the beam section size so that another row of bolts can be located, then it is stated as an "extended connection." End-plate connections can also be with or without stiffeners, as well as with varying bolt numbers and positions.

These connections are becoming more popular for a variety of economic reasons, including the elimination of field welding, the fact that they are less susceptible to cracking in freezing conditions, are more ductile and reliable, and reduce construction time. They enable a wide range of structural arrangements by legitimately changing the connection structural details. The moment and flexural resistance can be validly adjusted by picking a proper size and number of bolts and, to end with, a stiffening detail [2].

Many previous studies [3–7] have expressed concern about the connection's performance, particularly with common-sized beam members in monotonic force circumstances, for which design approaches have been implemented.

The vast general-purpose finite element software has expanded rapidly, and its capabilities are becoming increasingly flawless. This makes it conceivable to apply for the investigation of various bolted joints with extended end-plates. Accordingly, finite element simulation for extended end-plate connections is used in many dedicated studies.

Krishnamurthy and collaborators [8–10] achieved a broad study to inspect the behavior of that connection sort using finite element analysis. They proposed formulas to simulate the general rotational behavior, which pointed to a thinner end-plate than that was achieved previously

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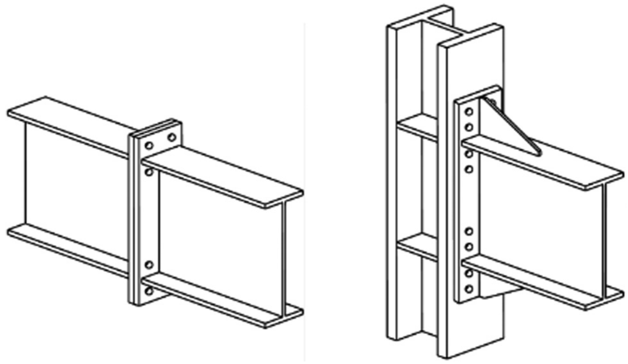

Figure 1: Schematic of a standard end-plate connection.

using realistic formulas. Ghassemieh et al. [11] considered the performance of an eight-bolted connection. They used experimental methods in conjunction with finite element analysis and achieved a good correlation between them. Kukreti and co-workers [12–14] presented a finite element-based methodology to describe the M–Φ conduct of the extended end-plate joint, and they performed experiments to validate it.

Bahaari and Sherbourne [15–19] proposed analytical expressions for the design of this connection type based on finite element analysis successions. They account for all major conductor effects, such as material nonlinearity and contact phenomena. Bursi and Jaspart [20,21] proposed a new finite element proposal for the conduct of the end-plate connection. Choi and Chung [22] presented a three-dimensional (3D) finite element simulation that considered different sources of nonlinearities, such as elastoplasticity and interaction, for a comprehensive study of the conductivity of end-plate connections.

Mays [23] used the finite element method (FEM) for designing a model for bolted extended end-plate connections with an unstiffened column and employed yield line theory to create outline methods that verified a great relationship with finite element analysis and experimental results. Maggi et al. [24] used ANSYS software for modeling experimental end-plate models, and they investigated the finite element results with experimental static loading. Gang et al. [25] described numerous experimental bolted end-plate joints and analyzed them using finite elements; they also discussed the extended end-plate design phases.

Mofid et al. [26] introduced a new analytical method based on yield line theory to demonstrate connection behavior. A good agreement between the experimental data and the presented technique was shown. Raasheduddin [1] conducted an analytical and finite element technique to determine the appropriate end-plate thickness and bolt diameter. A worthy compromise between the proposed analytical formulation and finite element analysis results has been reached.

It was still critical to be aware of the plastic rotation assessment of connections. Numerous techniques for anticipating the connection's performance were presented in Mazzolani and Piluso in 1996 [27].

The prediction of extended end-plate joints' capacity is significant for designing ductile moment-resisting steel assemblies. Distinctive methodologies can be utilized for anticipating connections, including empirical, analytical, finite element, and experimental counting [28,29].

Finite element analysis is quickly replacing analytical methods, particularly for issues that must be comprehended and are changed in accordance with assignments. As indicated by Mays [23], this technique may well utilize the conductivity of end-plate joints under various loading conditions. In the present paper, a 3D FEM has been produced to portray the conductivity of the spliced beam to the column via the extended end-plate connection.

This study presented a 3D numerical analysis to mimic the nonlinear conductivity of the beam-to-beam extended end-plate connections. In a parametric study, the influence of the bolt and end-plate size on connection conduct and failure sequence was investigated using two end-plate test setups. Stresses and forces in the connection parts were measured and discussed at various loading levels throughout the study.

2 Finite element modeling

The existing design practice required that connections in steel structures be verified prior to usage. The effective use of a finite element can offer a base for qualifying a connection pattern. After creating the finite element numerical model, the results must be validated. The work done by Sumner [7,30] was adopted herein to validate the proposed FEM model.

The developed model was a simply supported beam consisting of two beam sections linked at the middle of the span by an extended end-plate by bolts under the influence of a pure bending moment. The primary goal was to investigate the effect of bending moments as well as the performance of thick and thin plates aimed at being weaker than the jointed beams (Figure 2).

2.1 Modeling of the beam, end-plate, and bolt

The finite element program ABAQUS (6.14) was employed to build the model, as is shown in Figure 3. Steel beams,

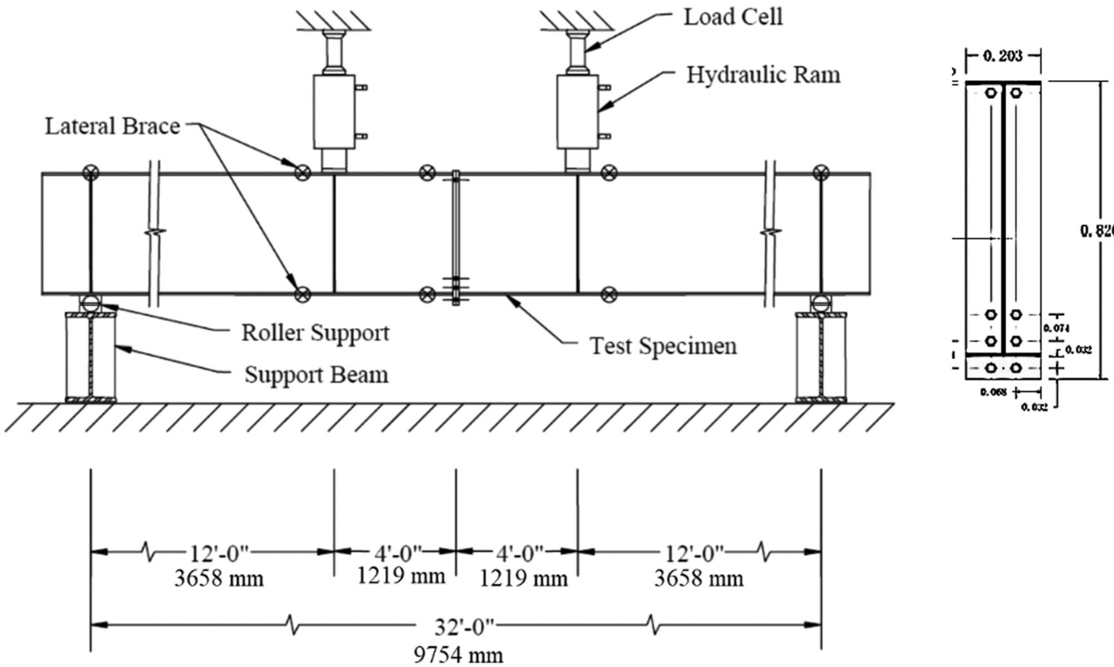

Figure 2: Schematic design for the typical experimental specimen.

bearing pads, end plates, and stiffeners are all modeled using continuum 3D quadratic elements with plasticity-incorporated properties. The bolts were modeled using 20-node brick elements with plasticity properties to represent the 3D stresses that were initiated on the bolts. The accuracy of finite element results is influenced by meshing; using a fine mesh that offers truthful results with less execution time is adopted in the model after inspecting altered mesh sizes. The model part instances method was used to create the bolt mesh shown in Figure 3. The model boundary conditions were applied, agreeing with those obtained from the experimental setup.

A hard "TIE" constraint is utilized to model welding (i.e., solid connection) among various parts of the beam and end plate. The contact surface interaction was estimated to have a friction coefficient of 0.30 for interfaces between the steel extended plate and the bolt head/nut, while the tangential behavior using the penalty stiffness formulation (small sliding) was used.

Steel was implemented as an isotropic elastic–plastic response in tension and compression. The nonlinear material properties of steel were imposed on the component by demonstrating two distinct multi-linear stress–strain relationships: the first was used to represent both the built-up

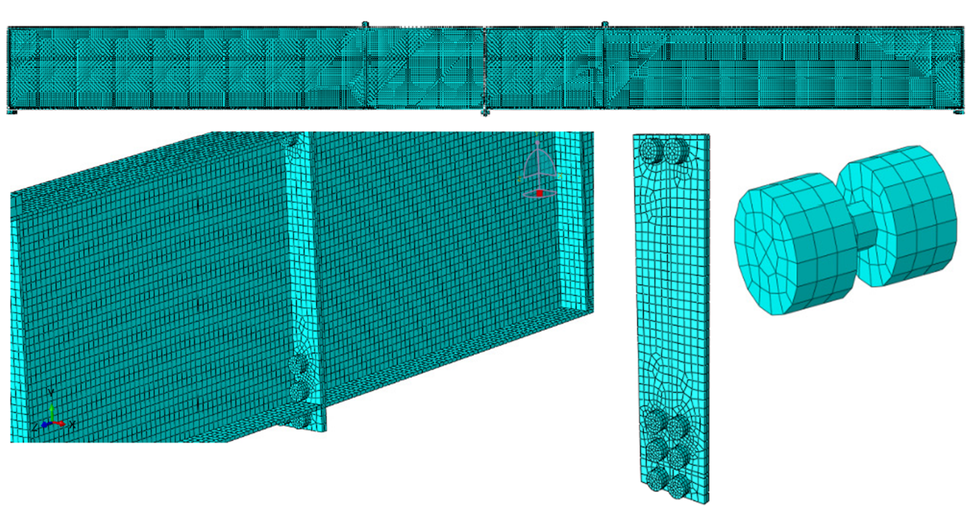

Figure 3: Component representation for FEM.

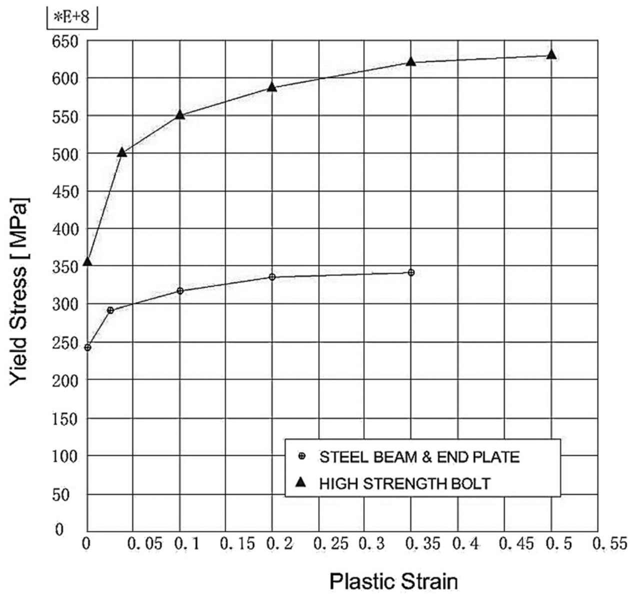

Figure 4: Customized models for use with steel components.

beam and end-plate materials and the second to epresent both the built-up beam and end-plate materials for the steel bolt. The yield stress-plastic strain relation for entirely spliced joint parts was simulated by utilizing the model shown in Figure 4.

2.2 FEM validation

To validate the numerical model exposed earlier, the numerical results were compared with Sumner [7,30] specimens by means of the moment strength, moment along with displacement and end-plate separation, and bolt response.

From the beams that were tested by Sumner [7,30] under monotonic loading, six beams were utilized as benchmarks, and the details of those beams are shown in Table 1. The beams' configuration attributes are demonstrated separately in Figure 2.

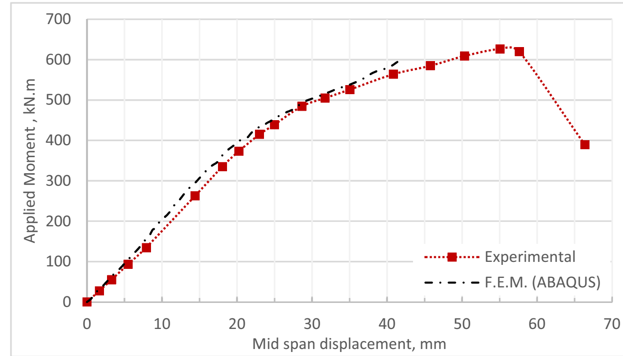

Figure 5: Experimental and finite element moment versus mid-span deflection for test A (thin plate).

A worthy relationship concerns the experimental and finite element responses at the first loading stages rather than in the final stages. The local deformations of the plate, bolts, and end-plate separations are shown in Figures 5–14. The results of the FEM are compared to the results of the experiment, and it is shown that the FEM accurately simulates the connection's conductivity. Table 2 shows the

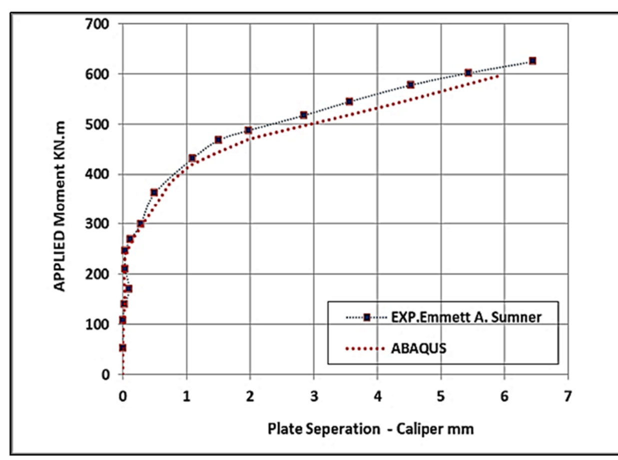

Figure 6: Experimental and finite element moment versus plate separation for test A (thin plate).

Table 1: Specimen components and properties

Test identification	Extend-plate thickness (mm)	Inner pitch, P_{fi} (mm)	Outer pitch, P_{fo} (mm)	Gage, g, g_o (mm)	Flange width (mm)	Beam depth, h (mm)	Bolt diameter, d_b (mm)	Bolt grade
A-MRE 1/2–3/8–3/4–30	10	32	32	90	203	762	20	A325
B-MRE 1/2–3/4–3/4–30	20	32	32	90	203	762	20	A325
B1-MRE 1/2–3/8–3/4–30	20	125	32	90	203	762	20	A490
C-MRE 1/2–3/4–1/2–30	12	125	32	90	203	762	20	A325
D-MRE 1/2–3/4–3/4–30	20	125	32	90	203	762	20	A325
D1-MRE 1/2–3/4–3/4–30	20	125	32	90	203	762	20	A490

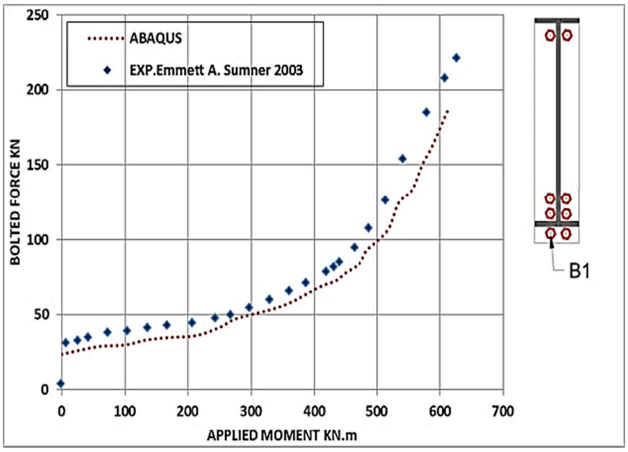

Figure 7: Experimental and finite element bolt force versus moment for test A (thin plate).

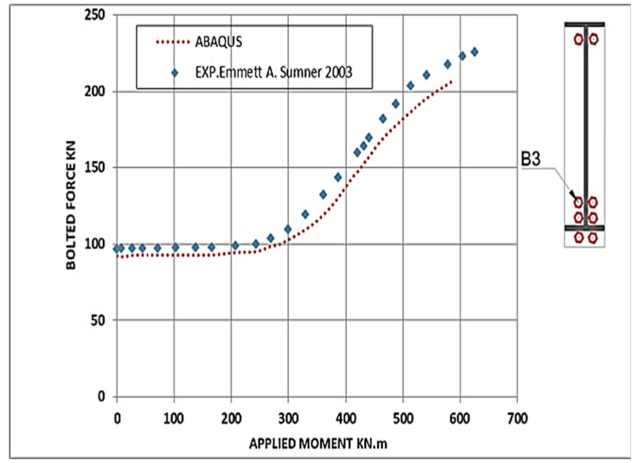

Figure 9: Experimental and finite element bolt force versus moment for test A (thin plate).

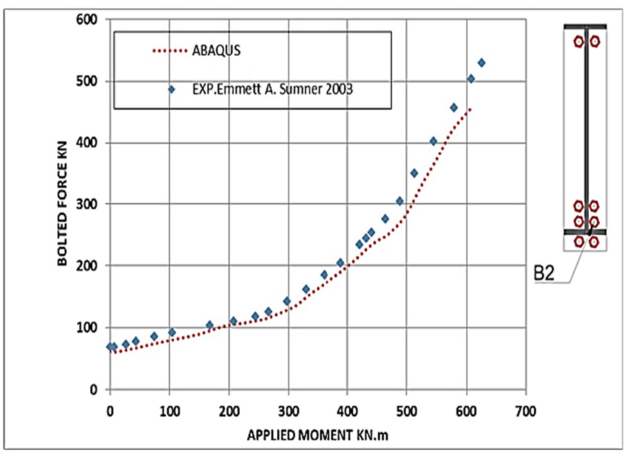

Figure 8: Experimental and finite element bolt force versus moment for test A (thin plate).

experimental moment resistance compared with finite element results.

As the analysis's divergence does not specify the real failure at each time, processes for checkups were adopted to recognize the criteria that caused the connection failure. The criteria were observing the stress and strain of bolts, determining that connected beam displacement should not exceed 20 times the allowable, and observing local buckling of the beam flange.

In place of the failure modes that occurred in the experiments and the finite element analysis, two kinds of failure modes were detected: thin (weak) and thick (strong) end-plate connections. The fundamental characteristics of the weak plate connection behavior were the yielding of the end-plate, which was followed by the tension rupturing of the bolt. Finite element analysis shows thin-plate behavior in two

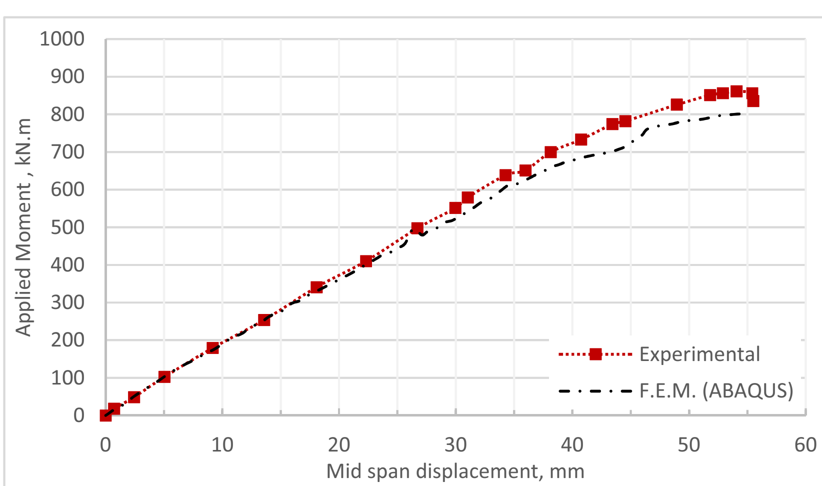

Figure 10: Experimental and finite element moment versus mid-span deflection for test B (thick plate).

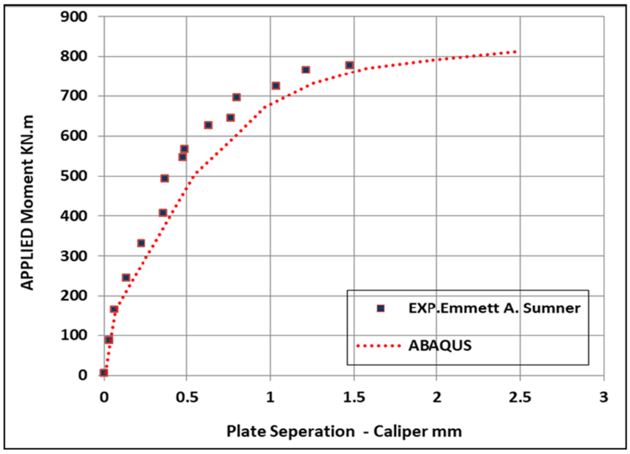

Figure 11: Experimental and finite element moment versus plate separation for test B (thick plate).

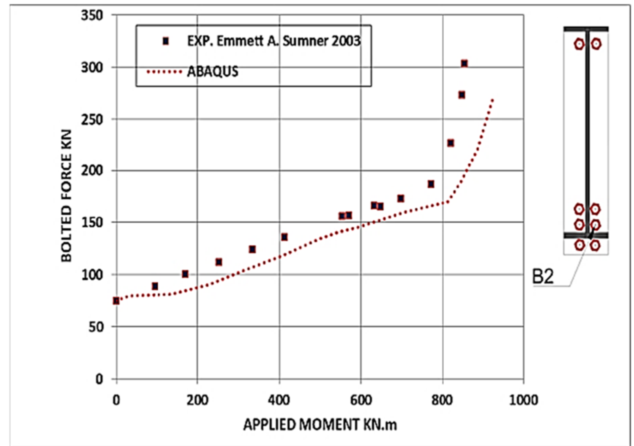

Figure 13: Experimental and finite element bolt force versus moment for test B (thick plate).

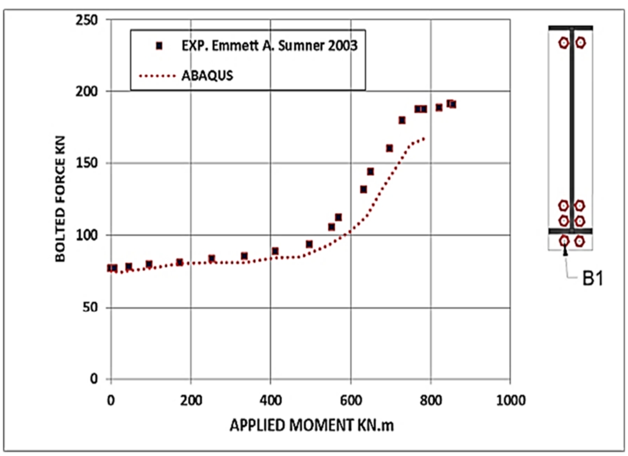

Figure 12: Experimental and finite element bolt force versus moment for test B (thick plate).

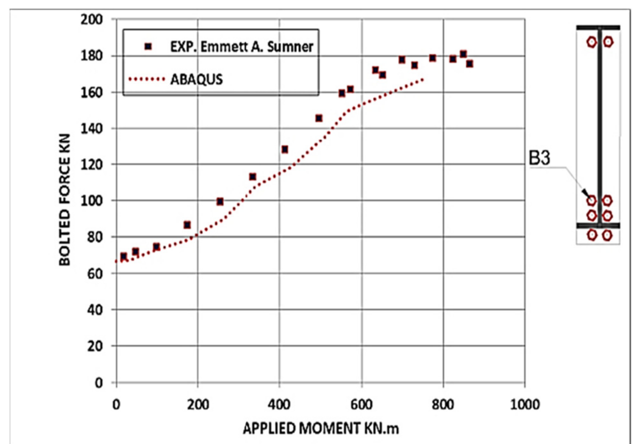

Figure 14: Experimental and finite element bolt force versus moment for test B (thick plate).

specimens' models confirmed by the inelastic load mid-span displacement response and the large end-plate separations.

In thick-plate connections, the bolt tension ruptures before the beginning of the end-plate yielding. The observed finite element connections show a small amount of inelastic response owing to the bolt yielding without any end-plate yielding, which shows typical thick-plate behavior. Table 3 compares the experimental failure mode with the results of the finite element model. The local deformations are shown in Figures 15–17.

Table 2: Experimental and finite element (ABAQUS) results

Test identification	Experimental applied moment (kN m)	Finite element results moment (kN m)	Finite element/Exp.
A-MRE 1/2–3/8–3/4–30	625	600	0.96
B-MRE 1/2–3/4–3/4–30	856	803	0.93
B1-MRE 1/2–3/8–3/4–30	1,016	980	0.96
C-MRE 1/2–3/4–1/2–30	651	655	1.01
D-MRE 1/2–3/4–3/4–30	755	704	0.93
D1-MRE 1/2–3/4–3/4–30	843	805	0.95
		MVR	0.96

Table 3: Experimental specimen and finite element model failure modes

Test identification	Finite element results moment (kN m)	Experimental failure modes		Finite element failure modes	
		Bolt	End-plate	Bolt	End-plate
A-MRE 1/2–3/8–3/4–30	600		Yielding		Yielding
B-MRE 1/2–3/4–3/4–30	803	Rupture		Rupture	Yielding
B1-MRE 1/2–3/8–3/4–30	980	Rupture		Rupture	
C-MRE 1/2–3/4–1/2–30	655	Rupture	Yielding	Rupture	Yielding
D-MRE 1/2–3/4–3/4–30	704	Rupture		Rupture	
D1-MRE 1/2–3/4–3/4–30	805	Rupture		Rupture	

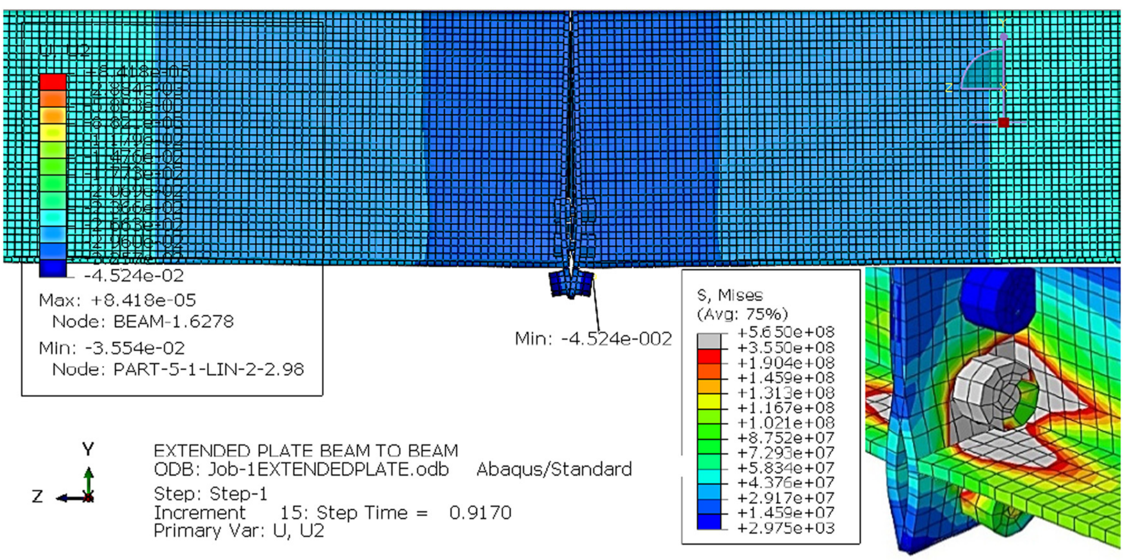

Figure 15: Abaqus' typical deformations of the thin-plate model.

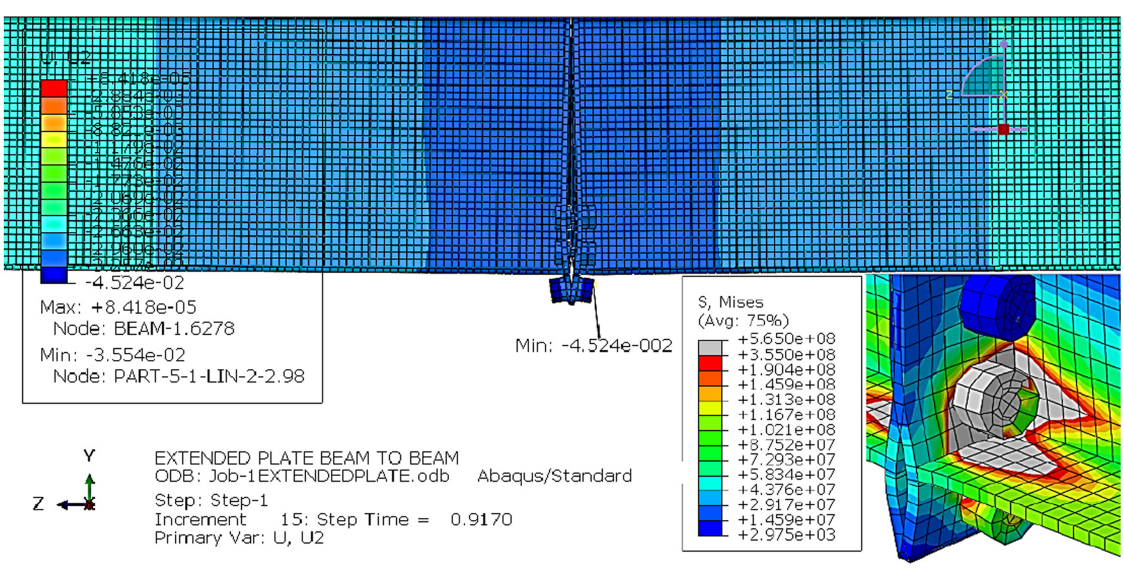

Figure 16: Abaqus' typical deformations of the thick-plate model.

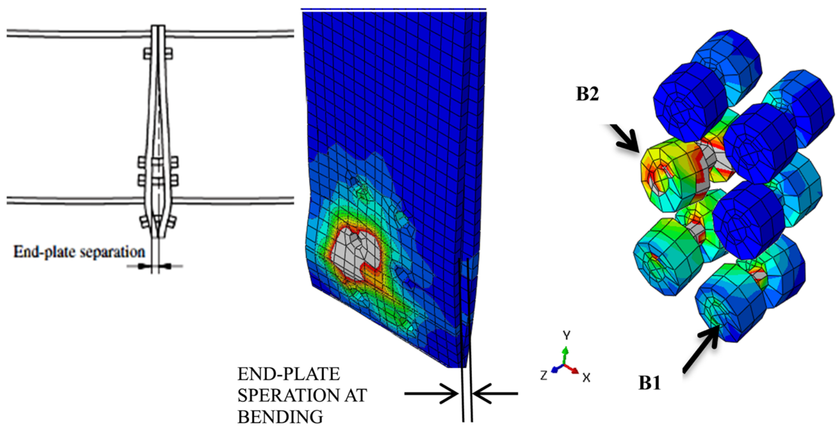

Figure 17: Enlarged deformation of plates and bolts.

2.3 Finite element results

Considering in detail test A, which serves as an ideal model for the conduct of a thin-plate connection, and test B, which serves as an ideal model for the behavior of a thick-plate connection, in order to emphasize the connection conduct if the end-plate was thin or thick.

Figure 5 illustrates the conduct of the connection in test A and displays the applied moment versus deflection in the center of the beam. It demonstrates that while the finite element analysis and loading were highly compatible early in the test, this compatibility was degraded later due to high yielding in end-plate and bolts. Test A shows a typical thin-plate connection conduct as the end-plate yields followed by rupturing in bolts. Figure 6 depicts the inelastic end-plate partition conduct and large end-plate separation preceding failure. Figures 7–9 depict the bolt's conduct plot and the sharp increase in bolt conduct indicates yield in a bolt on the verge of failing, as the moment turns out to be enormous the bolt will rupture by tension.

Figure 10 illustrates the conduct of the connection in tests and displays the applied moment versus deflection in the center of the beam. It demonstrates that the finite element analysis and loading are highly compatible in all loading stages.

Test B shows a typical thick-plate connection conduct as the bolts were ruptured before the end-plate yields were launched. The numerical models reveal that an intense strength moment, M_{max}, was required to cut off the bolts, which is approved by the adopted bolt force conduct model. The partition conduct is also shown in Figure 11, where a much larger separation is seen. Figures 12–14 display the bolt's conductance; every bolt achieved the numerical load, Pt, before connection failure. This is also an important perception to approve the used bolt force behavior. Figure 16 displays a snapshot of a thick-plate model and, subsequently, finite element analysis. The bolt failure and the absence of yields in the end-plate were clear. Figure 17 displays the bolts and plates deforming because of the tension in flanges.

3 Parametric study

The end-plate thickness and bolt diameter, which are regarded as the two most crucial factors in the bearing capacity of a joint of this kind, were taken into account in a parametric study using the developed finite element

Table 4: Different parameter values utilized in this study

Parameter	Designated value
End-plate thickness, t_p (mm)	8, 10, 12, 14, 16, 18, 20
Bolt diameter, d_b (mm)	12, 14, 16, 18, 20, 22

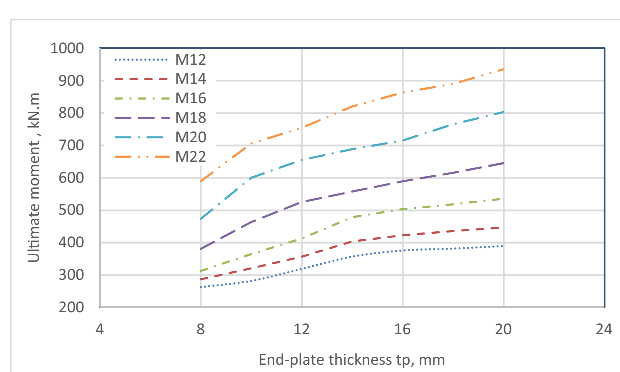

Figure 18: Ultimate moments vs end-plate thickness, a parametric study.

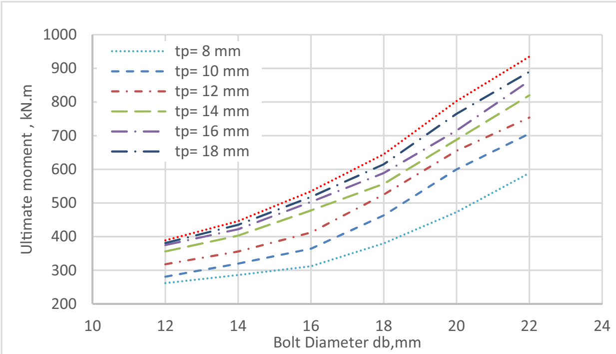

Figure 19: Ultimate moments vs bolt diameter, a parametric study.

model, as shown in Figure 2. Table 4 displays the various parameter values employed in this study.

3.1 End-plate thickness, t_p

Figure 18 plots the ultimate moment vs end-plate thickness relative to the selected bolt diameter range (12–22 mm). The connection displays thin-plate behavior for smaller bolt diameters (12 and 16 mm), with yield and ultimate moment increasing by 153% as plate thickness increases from 8 to 14 mm.

For higher bolt sizes (18–22 mm), the ultimate moment increased in the range 169–158%, as end-plate thickness increased from 8 to 20 mm. The behavior changes in thick plates, and the increase from 18 to 20 mm barely affects the ultimate moment. When the behavior shifts to a thick plate, the increase in the end-plate thickness has little to no impact, and the connection behavior is controlled by the bolts yielding and rupturing.

3.2 Bolt diameter, d_b

The ultimate moments versus bolt diameter concerning the chosen end-plate thickness range of 8–20 mm are shown in Figure 19. It was found that at thinner thicknesses (8 and 12 mm), there is a 9% increase in the yield moment as the diameter increases from 12 to 22 mm; the primary yielding here is caused solely by the end-plate. The ultimate moment of the connection shows an increase of around 238% as the bolt size is increased from 12 to 22 mm.

Ultimate moment increases by nearly 158% when the bolt diameter increases from 16 to 20 mm, although the yield moment values do not show such an increase. This is because the end-plate has a post-yielding capacity that is rather substantial, which enables bolts to have higher capacities than they otherwise would. Because the capacity of the end-plate has already been reached its maximum, increasing from 20 to 22 mm in diameter, the percentage increase was slighter.

As the bolt diameter increases from 12 to 22 mm, the ultimate moments for thicker end plates (16–20 mm) increase by around 230–240%, with about an 80–100% increase in the yield moment. When the diameter of the bolt was increased from 20 to 22 mm, it was discovered that the yield moment changed very little. This is because these connections operate as thick plates for diameters lower than 20 mm, and the yielding is only caused by bolts; for bolt sizes beyond 20 mm, the connections behave as thin plates, and the compliance is only caused by end-plate yielding.

4 Conclusions

This study presented a finite element simulation using the commercial software ABAQUS to investigate the performance of bolted steel beam-to-beam extended end-plate connections subjected to monotonic loading.

Connection performance was simulated excellently, and the ultimate numerical moment associated with six empirically tested beams fluctuated from 0.93 to 1.01 with an average deviation of 0.96. Two kinds of failure modes were detected: thin (weak) and thick (strong). The connection displays thin-plate behavior for smaller bolt diameters (12 and 16 mm), with yield and the ultimate moment increasing by 153% when the end-plate thickness increases from 8 to 14 mm. For bigger bolt sizes (18–22 mm), the ultimate moment increases by 169–158% when the end-plate size increases from 8 to 20 mm. When conduct shifts to thick-plate conduct, the increase in end-plate thickness has little to no impact, and the connection behavior is controlled by the bolts yielding and rupturing.

The yield moment increases when the bolt diameter increases from 12 to 22 mm, with a 9% increase at thinner thicknesses (8 and 12 mm). The ultimate moment of the connection increases by up to 158% when the bolt diameter increases from 16 to 20 mm. When the bolt diameter increases from 12 to 22 mm, the ultimate moments for thicker end plates (16–20 mm) increase by 230–240%, with an 80–100% increase in the yield moment. The yield moment barely vagaries when the bolt diameter is increased from 20 to 22 mm.

Conflict of interest: The authors state no conflict of interest.

Data availability statement: Most datasets generated and analyzed in this study are comprised in this submitted manuscript. The other datasets are available on reasonable request from the corresponding author with the attached information.

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