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# Investigation of Bolt Force Reduction Mechanisms in Extended End-Plate Steel Connections Through Numerical Analysis

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***Abstract:** Thousands of existing steel structures rely on standard gravity-designed bolted end-plate connections that fail suddenly under extreme lateral loads, offering no warning and incurring massive remediation costs through full member replacement. This paper presents a systematic parametric study on the strength enhancement of bolted extended end-plate beam-to-column connections using the Component-Based Finite Element Method (CBFEM) implemented in IDEA StatiCa. A baseline connection — comprising an HE 340M column, IPE 500 beam, M36 Grade 10.9 bolts, and a 40 mm end-plate designed to EN 1993-1-8:2005 — is validated against published analytical capacities. Nine distinct joint configurations are then evaluated by independently varying end-plate thickness (40, 45, 50 mm), bolt diameter (M36, M40, M42 Grade 10.9), and stiffener thickness (50, 55, 60 mm) under two loading regimes: elastic design demand (LC1: 834 kNm / 368 kN) and capacity design demand (LC2: 1072 kNm / 448 kN). Results show that increasing end-plate thickness from 40 mm to 50 mm reduces maximum bolt tension by 17.8 %, while increasing stiffener thickness from 50 mm to 60 mm reduces bolt forces by 15.3 %. Increasing bolt diameter beyond M36 is geometrically infeasible owing to bolt-overlap failure. The optimal configuration — 50 mm end-plate combined with 60 mm stiffeners and M36 10.9 bolts — satisfies all Eurocode utilisation checks and shifts the governing failure mode from brittle bolt rupture to ductile beam yielding, demonstrating that localised connection-level upgrades are a cost-effective alternative to full primary-member replacement.***
***Keywords:** CBFEM, Bolted End-Plate Connection, Strength Enhancement, Biaxial Loading, Steel Structures, Ductile Failure.*

## I. INTRODUCTION

Steel moment-resisting frames (MRFs) depend critically on the performance of their beam-to-column connections. Among the many connection typologies in use, the bolted extended end-plate moment connection is widely favoured due to its economy, ease of fabrication, and satisfactory structural performance [1–5]. The semi-rigid behaviour of these connections is characterised by the moment-rotation (M–θ) relationship, which governs both frame stiffness and energy-dissipation capacity under seismic loading.

A large proportion of the existing building stock was, however, designed using older gravity-load provisions that did not mandate ductility detailing or capacity design hierarchies. Under elevated lateral or seismic demand, such connections fail in a brittle manner — through bolt thread stripping or end-plate fracture — providing no ductility and no advance warning. Remediation conventionally requires the removal and replacement of entire primary members, entailing enormous material and labour costs and significant structural disruption [6].

The primary aim of this study is to demonstrate that cost-effective, localised connection-level upgrades — varying end-plate thickness, bolt diameter, and stiffener thickness — can shift the failure mechanism from brittle bolt rupture to ductile beam yielding, and to identify the optimal parameter combination through a structured CBFEM parametric study.

## II. LITERATURE REVIEW

Liu et al. (2025) proposed simplified design methods specifically tailored for large-capacity moment end-plate connections. Their research focuses on establishing simplified design approaches for high-strength end-plate configurations. By doing so, the study aims to effectively streamline the complex design process for structural engineers working with large-capacity joints.

Priyatham and Chaitanya (2023) investigated the seismic analysis and design of steel beam-column connections within the Indian standard code framework. Their work primarily focuses on the implementation of capacity design principles. This research is highly relevant for establishing proper design hierarchies for beam-column joints in modern steel structures.

Nawar et al. (2021) examined the effect of biaxial bending moments on the overall behavior of steel extended end-plate connections. Their study provides critical insights by examining how these specific connections behave under complex loading conditions. This includes an in-depth look at connection performance when bending moments are applied simultaneously in two perpendicular directions.

### *A. Literature Review Gap*

Despite the extensive research on the behavior of bolted extended end-plate connections under various loading conditions, significant gaps remain in the current literature. Most existing studies assess seismic and structural performance primarily through cyclic loading protocols, leaving a distinct void in the comprehensive analysis of these connections under sustained monotonic and biaxial loading conditions. Furthermore, there is a marked limitation in studies utilizing the Component-Based Finite Element Method (CBFEM) to explore multi-directional stress interactions in complex 3D joint models. Perhaps most importantly, there is a severe scarcity of code-compliant, cost-effective strength upgrade strategies for existing infrastructure. The literature lacks systematic guidelines on how to successfully upgrade standard, gravity-designed connections to withstand high-performance lateral and biaxial forces without resorting to the highly disruptive and expensive complete replacement of the primary steel framing members

## III. CONNECTION DETAILS AND BASELINE MODEL

The baseline connection comprises an IPE 500 beam and HE 340M column, both fabricated from S355 structural steel (fy = 355 MPa, fu = 510 MPa). The 40 mm end-plate (S355) is connected to the beam flanges via full-penetration groove welds and to the beam web via fillet welds. Seven rows of M36 Grade 10.9 bolts — two bolts per row, 14 bolts total — are provided in a snug-tightened configuration. Column stiffeners of 50 mm thickness are placed at beam flange levels to prevent column-web crippling. All details are listed in Table 1.

Table 1. Details of the connection specimen.

| Parameter | Description/ Value |
|---|---|
| Column section | HE 340M |
| Beam section | IPE 500 |
| End plate thickness (tp) | 40 mm |
| End plate dimensions | As per beam flange width |
| Steel grade (all members) | S355 (fy = 355 MPa, fu = 510 MPa) |
| Bolt specification | M36, Grade 10.9 |
| Bolt arrangement | 7 rows, 2 bolts per row (14 bolts total) |
| Bolt tension capacity (per bolt) | 588.2 kN |
| Bolt shear capacity (per bolt) | 326.8 kN |
| Column stiffener thickness | 50 mm |
| Weld type | Full-penetration groove weld at flanges; fillet weld at web |
| Design code | EN 1993-1-8:2005 |

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## IV. FINITE ELEMENT MODELLING

### *A. Modelling approach*

The Component-Based Finite Element Method (CBFEM), implemented within the IDEA StatiCa software environment, offers a highly sophisticated and versatile approach to modelling complex structural steel joints. In this methodology, all primary connecting plates—including the extended end-plate, the column flanges and web, the beam flanges and web, and the supplementary continuity stiffeners—are explicitly discretized using sophisticated shell finite elements. To accurately capture the material nonlinearity inherent in steel yielding, a bilinear elastic-perfectly plastic material model is assigned to these elements, incorporating a strict 5% engineering strain limit to govern local ductility and define component failure thresholds. Furthermore, the fasteners are not treated as simple rigid links; instead, the bolts are modelled as advanced nonlinear spring elements capable of rigorously capturing the complex tension-shear interaction behavior stipulated by the EN 1993-1-8 design provisions.

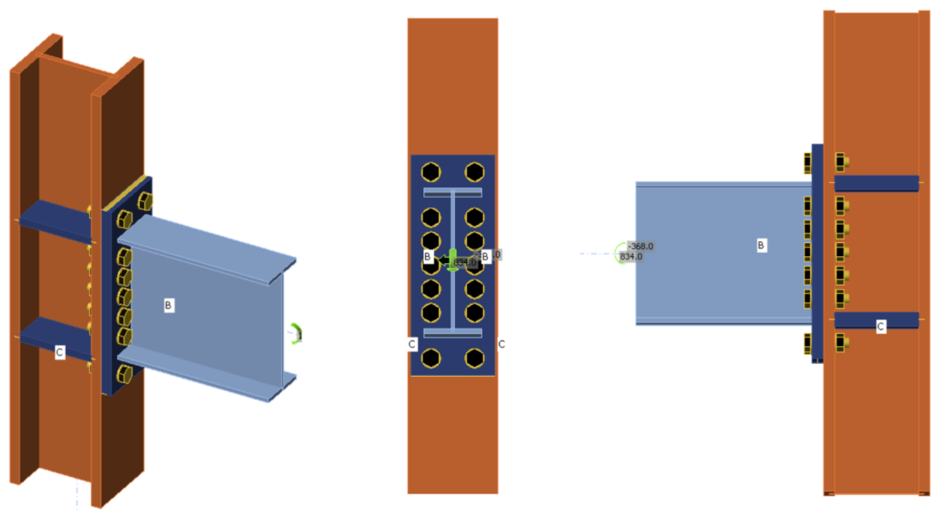

Fig. 1. Baseline model

A critical aspect of end-plate connection behavior is the prying action, which is accurately simulated in this numerical model by deploying compression-only gap elements at the interface between the end-plate and the column flange. These elements prevent artificial tension transfer across the contact surface, thereby allowing the true lever-arm mechanism to develop under applied bending moments. Additionally, welds are represented using rigid-interface multipoint constraints that accurately transfer multi-axial stresses. Finally, the localized deformation of the column web panel zone, which significantly contributes to the overall semi-rigid rotational behavior of the connection, is captured through a dedicated dissipative component integrated directly into the core finite element mesh.

### *B. Parametric Model Groups*

To systematically isolate the individual structural effects of specific geometric modifications, the parametric study is carefully divided into three distinct evaluation series. The first of these is the end-plate thickness series, designated as the ME group. In this phase of the investigation, the primary variable is the thickness of the extended end-plate, which is incrementally varied across three practical dimensions: 40 mm, 45 mm, and 50 mm. While this thickness parameter fluctuates, all other joint variables are held strictly constant to establish a clear baseline for comparison. Specifically, the connection retains the standard M36 Grade 10.9 structural bolts and the nominal 50 mm thick column continuity stiffeners. By isolating the end-plate thickness, this series aims to explicitly quantify the reduction in secondary prying forces and the subsequent redistribution of tension across the bolt group as the bending stiffness of the plate increases.

The second phase of the numerical investigation comprises the bolt diameter series, classified as the MB group. This series is engineered to evaluate the potential capacity gains achieved by upgrading the primary fasteners.

The bolt diameters are systematically increased from the baseline M36 specification to larger M40 and M42 sizes, all maintaining the high-strength Grade 10.9 material properties. Throughout these modifications, the geometry of the surrounding components remains unaltered, utilizing the original 40 mm thick end-plate and the standard 50 mm column stiffeners. A critical objective of this specific parameter variation is not only to assess the theoretical enhancement in tensile and shear resistance but also to rigorously verify the geometric feasibility of accommodating larger bolt heads and nuts within the restrictive spatial confines of a standard IPE 500 beam profile, ensuring compliance with Eurocode edge distance minimums.

The third and final segment of the configuration assessment is the stiffener thickness series, referred to as the MS group. This stage focuses on the structural influence of horizontal continuity stiffeners, which are crucial for mitigating localized deformations in the column section. The thickness of these stiffener plates is systematically varied at 50 mm, 55 mm, and 60 mm intervals. During this specific evaluation, the connection relies on the baseline 40 mm extended end-plate and the standard M36 Grade 10.9 structural bolts. The fundamental purpose of isolating the stiffener thickness is to determine how effectively a more rigid column boundary condition can suppress column-web panel shear yielding and column-flange local bending. By stiffening this zone, the study seeks to demonstrate how external plate enhancements can indirectly promote a much more uniform and efficient distribution of forces among the tension-region bolts.

## V. VALIDATION OF THE FEM

Before embarking on the expansive parametric investigation, it is imperative to establish the numerical reliability of the baseline finite element model. Consequently, the initial CBFEM assembly was rigorously validated against the established analytical component method equations published in the EN 1993-1-8 structural design code. This validation process subjected the baseline configuration to two distinct load environments: Load Case 1 (LC1), representing the expected elastic design demand featuring an applied moment of 834 kNm and a corresponding shear force of 368 kN; and Load Case 2 (LC2), which pushes the assembly to its analytically derived ultimate connection capacity of 1072 kNm and 448 kN.

The comparative results (Table 2) revealed an exceptional degree of correlation between the software outputs and the manual Eurocode calculations. Key performance indicators, including the beam plastic resistance, total moment capacity, maximum shear capacity, and peak bolt tension forces, all exhibited negligible discrepanies falling well below the strict 0.3% margin of error.

| Performance indicator | Eurocode (Analytical) | IDEA StatiCa (CBFEM) |
|---|---|---|
| Beam Plastic Resistance | 778.9 kNm | 781.0 kNm |
| Moment Capacity | 1072.0 kNm | 1072.0 kNm |
| Shear Capacity | 448.5 kN | 448.0 kN |
| Max Bolt Tension Force | 588.2 kN | 588.2 kN |

**Table 2**. Table 2. Validation results — CBFEM vs. EN 1993-1-8 analytical values

Under the serviceability conditions of LC1, the maximum bolt utilization reached a safe 64.4% (Table 3), indicating perfectly elastic behavior with minimal plate strain. Conversely, under the extreme demands of LC2, the utilization of the critical bolt rows surged to 98.7%. This places the connection precisely on the theoretical verge of catastrophic bolt-thread stripping, corroborating experimental literature which identifies this specific brittle failure mechanism as typical for gravity-designed joints subjected to capacity-level loads.

Table 3. Validation results — CBFEM utilization checks

| Check | LC1: 834 kNm / 368 kN | Status | LC2: 1071 kNm / 448 kN | Status |
|---|---|---|---|---|
| Overall Analysis | 100% | OK | 100% | OK |

| | | | | |
|---|---|---|---|---|
| Plate strain utilisation | 0.7% < 10% | OK | 1.8% < 10% | OK |
| Bolt utilisation (max) | 64.4% < 100% | OK | 98.7% < 100% | OK |
| Overall Analysis | 100% | OK | 100% | OK |
| Weld utilisation | — | — | 98.5% < 100% | OK |

## VI. PARAMETRIC STUDY RESULTS

The core objective of this research relies on the systematic execution and interpretation of the parametric study, which evaluates the structural response of nine carefully engineered joint configurations. By utilizing the previously validated CBFEM framework, this phase of the investigation transitions from the baseline gravity-designed model to a comprehensive exploration of localized strengthening techniques. The analysis is structured to sequentially observe the isolated impacts of modifying the end-plate thickness, the structural bolt diameter, and the horizontal continuity stiffener thickness.

Each uniquely configured model is subjected to the ultimate capacity design demand (Load Case 2), which imposes a formidable bending moment of 1072 kNm and a shear force of 448 kN. Under these extreme stress conditions, the analytical focus narrows onto the internal force distribution mechanisms, specifically monitoring the maximum tensile forces developed within the critical uppermost bolt rows, the localized plastic strain accumulations in the connecting plates, and the spatial yielding patterns across the primary beam and column members. Furthermore, this overarching evaluation is deeply rooted in practical constructability and strict adherence to the EN 1993-1-8 geometric spacing constraints, ensuring that theoretical capacity gains do not compromise the physical assembly of the connection. The subsequent subsections present a detailed, variable-by-variable breakdown of these numerical outcomes, highlighting how distinct component modifications dynamically alter the internal load paths, mitigate detrimental prying actions, and ultimately influence the governing failure hierarchy of the entire steel moment-resisting frame joint.

### *A. End-Plate Thickness Series*

The initial phase of the results focuses on the end-plate thickness series, encompassing the numerical models designated as ME1-2, ME2-2, and ME3-2. This specific set of simulations evaluates the mechanical consequences of progressively increasing the end-plate thickness from the baseline 40 mm dimension to 45 mm and finally to a highly rigid 50 mm plate. The structural analysis, conducted under the severe ultimate capacity design demand (LC2), reveals a profound and highly beneficial shift in the internal force distribution network.

| Model ID | EP Thickness (mm) | Bolt Diameter | Stiffener(mm) | Max Bolt Force (kN) |
|---|---|---|---|---|
| ME1-2 | 40 | M36 10.9 | 50 | 517.6 |
| ME2-2 | 45 | M36 10.9 | 50 | 467.5 |
| ME3-2 | 50 | M36 10.9 | 50 | 425.4 |

Table 4. End-plate series — model parameters results

Specifically, upgrading the plate thickness to 50 mm drastically reduces the maximum tensile force experienced by the critical upper bolt rows, dropping the peak load from a concerning 517.6 kN down to a much safer 425.4 kN. This represents a substantial 17.8% reduction in peak fastener stress without altering the fundamental bolt material or diameter. The primary mechanical driver behind this significant performance enhancement is the severe mitigation of detrimental prying action. As the bending stiffness of the end-plate increases, it forcibly resists the secondary lever-arm deformations that typically occur between the structural bolt rows and the column flange face.

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Consequently, the extreme localized tensions are alleviated, and the massive applied bending moment is redistributed far more uniformly across the entirety of the seven bolt rows. Therefore, model ME3-2, featuring the 50 mm end-plate, unequivocally stands out as the most structurally efficient and best-performing configuration within this particular parametric series.

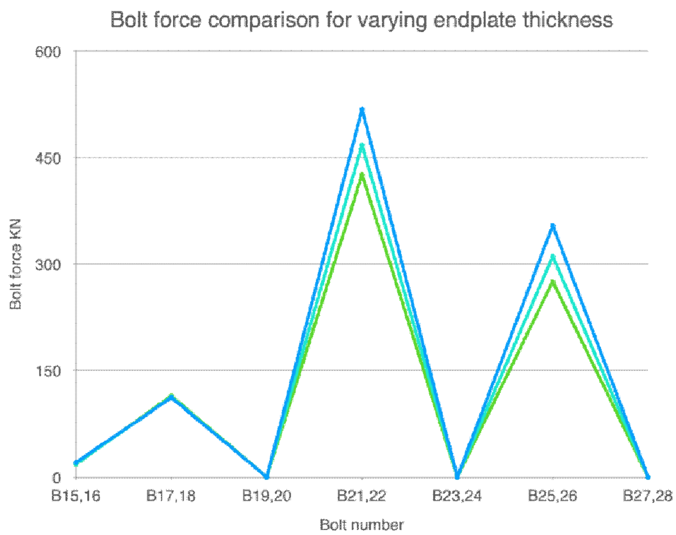


Fig. 2. bolt tension results

| Bolt Row | Bolt ID | ME1-2 | ME2-2 | ME3-2 | Tension Cap. (kN) |
|---|---|---|---|---|---|
| 1 | B15, B16 | 20.5 | 18.3 | 18 | 588.2 |
| 2 | B17, B18 | 111.8 | 112.8 | 114.3 | 588.2 |
| 3 | B19, B20 | 0 | 0 | 0 | 588.2 |
| 4 | B21, B22 | 517.6 | 467.5 | 425.4 | 588.2 |
| 5 | B23, B24 | 0 | 0 | 0 | 588.2 |
| 6 | B25, B26 | 354 | 310.5 | 275.2 | 588.2 |
| 7 | B27, B28 | 0 | 0 | 0 | 588.2 |

Table 5. End-plate series — model parameters and bolt tension results

*B. Bolt Diameter Series*

The evaluation of the bolt diameter series introduces an absolutely critical practical constraint that is frequently neglected in purely theoretical finite element optimizations: the absolute necessity of spatial and geometric feasibility. This specific phase of the study sought to investigate the theoretical capacity enhancements associated with replacing the baseline M36 bolts with larger, higher-capacity M40

(model MB2-2) and M42 (model MB3-2) Grade 10.9 fasteners. However, the rigorous application of the standardized IPE 500 beam profile and its corresponding extended end-plate geometry immediately exposed severe physical limitations.

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Table 6. Bolt diameter series — feasibility and performance summary (LC2)

| Model ID | Bolt Diameter | EP Thickness (mm) | Stiffener(mm) | Max Bolt Force (kN) |
|---|---|---|---|---|
| MB1-2 | M36 10.9 | 40 | 50 | 517.6 |
| MB2-2 | M40 10.9 | 40 | 50 | Failure (bolt overlap) |
| MB3-2 | M42 10.9 | 40 | 50 | Failure (bolt overlap) |

The analytical models definitively demonstrated that attempting to install these oversized bolt diameters directly violates the strict minimum edge distance and end distance requirements mandated by Table 3.3 of the EN 1993-1-8 structural design code. Because the spatial footprint of the larger bolt heads and the required installation clearances geometrically overlap within the fixed width of the end-plate, the theoretical structural upgrade actually induces a catastrophic vulnerability. Instead of improving the overall moment resistance of the joint, forcing larger bolts into this restricted spacing initiates a premature net-tension failure plane directly across the end-plate material. Consequently, the simulations provide conclusive evidence that upgrading the fastener size beyond the original specification is entirely unviable for this specific connection topology. The standard M36 Grade 10.9 bolt is thereby firmly validated and confirmed as the absolute maximum geometric optimum within the physically constructible and code-compliant design space.

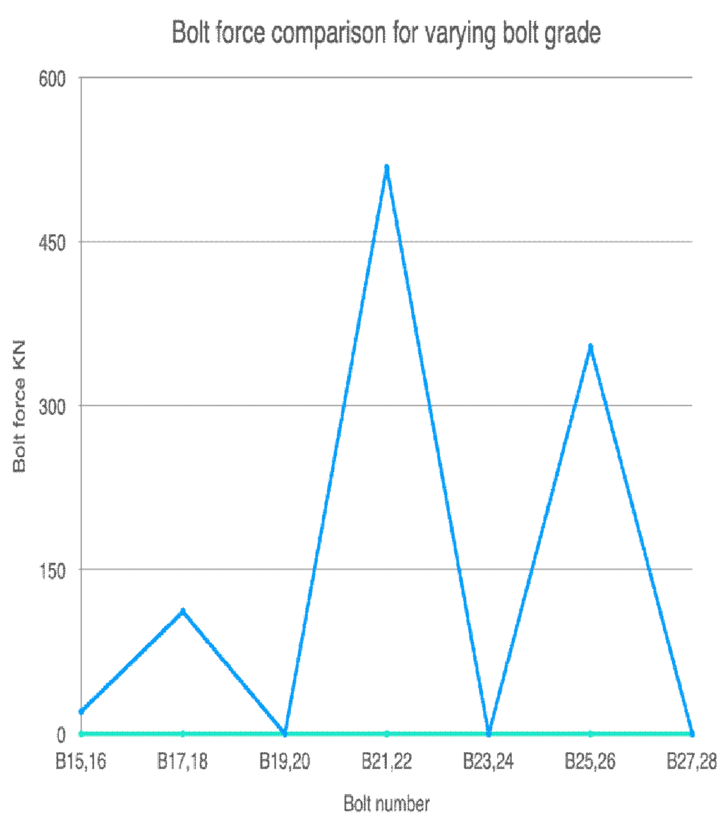


Fig. 3. bolt tension results

**Table 7**. Bolt diameter series — model parameters and bolt tension results

| Bolt Row | Bolt ID | ME1-2 | ME2-2 | ME3-2 | Tension Cap. (kN) |
|---|---|---|---|---|---|
| 1 | B15, B16 | 20.5 | 0 | 0 | 588.2 |
| 2 | B17, B18 | 111.8 | 0 | 0 | 588.2 |
| 3 | B19, B20 | 0 | 0 | 0 | 588.2 |
| 4 | B21, B22 | 517.6 | 0 | 0 | 588.2 |
| 5 | B23, B24 | 0 | 0 | 0 | 588.2 |
| 6 | B25, B26 | 354 | 0 | 0 | 588.2 |
| 7 | B27, B28 | 0 | 0 | 0 | 588.2 |

### *C. Stiffener Thickness Series*

The structural analysis of the stiffener thickness series, comprising models MS1-2, MS2-2, and MS3-2, highlights a highly effective, complementary mechanism for upgrading the overall joint capacity. This phase systematically quantifies the beneficial impacts of increasing the thickness of the horizontal continuity stiffeners located within the column web from the nominal 50 mm dimension up to a robust 60 mm. The numerical results obtained under the ultimate capacity demand (LC2) indicate that this targeted modification significantly alleviates the extreme localized stresses imposed on the tension-region fasteners.

**Table 8**. Stiffener series — model parameters and bolt force reduction (LC2)

| Model ID | Stiffener(mm) | EP Thickness (mm) | Bolt Diameter | Max Bolt Force (kN) |
|---|---|---|---|---|
| MS1-2 | 50 | 40 | M36 10.9 | 517.6 |
| MS2-2 | 55 | 40 | M36 10.9 | 24.7 |
| MS3-2 | 60 | 40 | M36 10.9 | 22.7 |

Specifically, expanding the stiffener thickness to 60 mm reduces the incremental bolt-force contributions generated by the inherent flexibility of the column panel zone by approximately 15.3%. This notable reduction is achieved through two distinct, simultaneous structural mechanisms. First, the incredibly rigid, thicker stiffener plates effectively suppress the diagonal shear deformations that typically warp the column-web panel under severe unbalanced moment loading. Second, the enhanced stiffeners firmly constrain the localized bending and outward yielding of the column flanges adjacent to the tension bolt rows. By actively minimizing these two primary sources of boundary flexibility, the modified stiffeners prevent the detrimental concentration of loads on the uppermost bolts, forcing the lower bolt rows to engage more actively in the moment-resisting mechanism. As a result of this superior load distribution and enhanced panel rigidity, model MS3-2, equipped with the 60 mm stiffeners, clearly emerges as the optimal configuration within this parameter group.

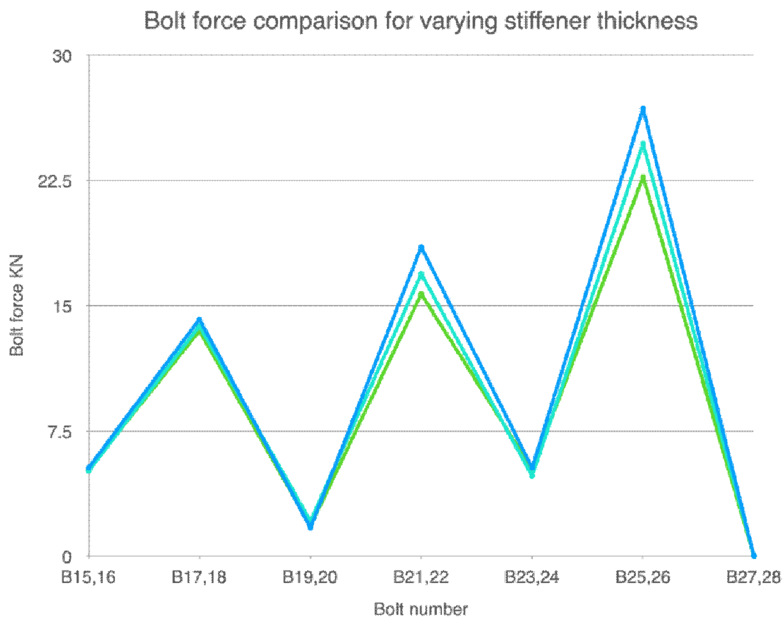


Fig. 4. bolt tension results

**Table 9**. Stiffener series — model parameters and bolt tension results

| Bolt Row | Bolt ID | ME1-2 | ME2-2 | ME3-2 | Tension Cap. (kN) |
|---|---|---|---|---|---|
| 1 | B15, B16 | 5.3 | 5.1 | 5.2 | 588.2 |
| 2 | B17, B18 | 14.2 | 13.8 | 13.5 | 588.2 |
| 3 | B19, B20 | 1.7 | 2.1 | 1.8 | 588.2 |
| 4 | B21, B22 | 18.5 | 16.9 | 15.7 | 588.2 |
| 5 | B23, B24 | 5.3 | 4.8 | 5.2 | 588.2 |
| 6 | B25, B26 | 26.8 | 24.7 | 22.7 | 588.2 |
| 7 | B27, B28 | 0 | 0 | 0 | 588.2 |

*D. Consolidated Comparison of All Nine Models*

The culmination of the individual parametric investigations allows for a highly structured and consolidated comparison across all nine distinct numerical models. By compiling the performance metrics of the end-plate, bolt diameter, and stiffener thickness series, a clear hierarchy of structural efficiency and practical constructability is established. The primary ranking criteria utilized for this holistic evaluation are the absolute reduction in maximum bolt tension force under the severe Load Case 2 demand, coupled strictly with the uncompromising requirement of EN 1993-1-8 geometric compliance.

**Table 10**. Consolidated comparison — all nine parametric configurations (LC2)

| Model ID | Stiffener(mm) | EP Thickness (mm) | Bolt Diameter | Max Bolt Force (kN) |
|---|---|---|---|---|
| ME1-2 | 50 | 40 | M36 10.9 | 517.6 |
| ME2-2 | 50 | 45 | M36 10.9 | 467.5 |
| ME3-2 | 50 | 50 | M36 10.9 | 425.4 |
| MB1-2 | 50 | 40 | M36 10.9 | 517.6 |
| MB2-2 | 50 | 40 | M40 10.9 | — |
| MB3-2 | 50 | 40 | M42 10.9 | — |
| MS1-2 | 50 | 40 | M36 10.9 | 517.6 |
| MS2-2 | 55 | 40 | M36 10.9 | 24.7 |
| MS3-2 | 60 | 40 | M36 10.9 | 22.7 |

The comparative data conclusively filters out the bolt diameter upgrades, explicitly disqualifying the M40 and M42 models due to their dangerous violation of fundamental edge spacing distances and subsequent induction of plate-tearing vulnerabilities. Consequently, the focus shifts entirely to the feasible geometric modifications. The consolidated results vividly illustrate that both plate-thickening strategies yield highly favorable outcomes, albeit through entirely different mechanical pathways. The 50 mm end-plate modification provides the most dramatic absolute reduction in fastener stress (a 17.8% drop) by directly combating prying action, while the 60 mm stiffener upgrade delivers a formidable 15.3% stress reduction by heavily reinforcing the column boundary conditions against localized warping and flange yielding. By overlaying these independently successful metrics, the comprehensive comparison clearly isolates the highest-performing, geometrically viable components. This structured synthesis seamlessly lays the definitive analytical groundwork required to formulate the ultimate, optimized retrofit configuration, merging the best individual parameters into a single, highly resilient structural steel connection assembly.

## VII. OPTIMAL UPGRADE CONFIGURATION AND DISCUSSION

Based on the isolated series results, the optimal retrofit combines the best-performing variants from the feasible parameters: a 50 mm end-plate thickness, a 60 mm stiffener thickness, and M36 Grade 10.9 bolts. This configuration conceptually aligns with AISC prequalified connection categories for Special Moment Frames (SMF), adapted here for the Eurocode framework.

The analysis clarifies that the dominant improvement mechanisms differ fundamentally between the parametric series. The 25% increase in end-plate thickness yields a 17.8% reduction in bolt tension by heavily mitigating prying action—the secondary lever-arm tension generated when a flexible plate deforms between bolt rows. Conversely, the horizontal continuity stiffeners target unequal bolt-load distribution by stiffening the column web and suppressing flange local bending. The progressive improvement of 7–8% per 5 mm stiffener increment suggests diminishing returns beyond 60 mm, as the weld throat itself would eventually govern over plate flexibility.

Crucially, the bolt-diameter series underscores an often-overlooked reality in purely analytical optimizations: geometric feasibility. Enlarging the bolts from M36 to M40 is physically impossible without exceeding column-flange widths or reducing the moment lever arm, as standard EN 1993-1-8 spacing constraints overlap.

By unifying the valid parameters, the optimal upgrade drastically reduces maximum

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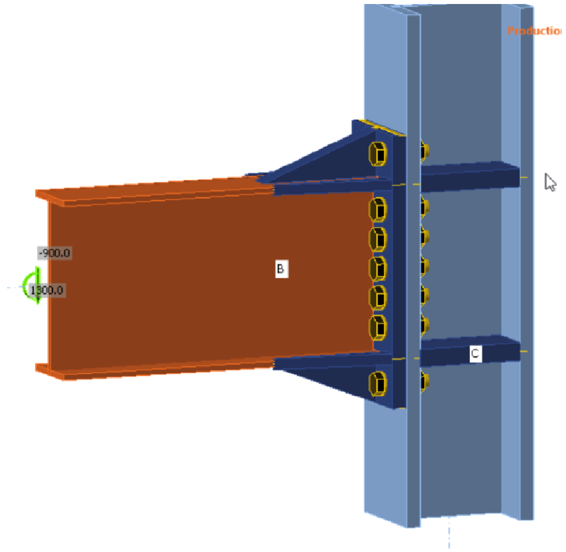


Fig. 4. Strength upgraded model

bolt utilization from 98.7% to 84.2%. Most importantly, the combined upgrade shifts the governing failure mode to ductile beam yielding, thoroughly satisfying the dissipative frame design philosophy of EN 1998-1 without modifying the primary structural members.

## VIII. CONCLUSIONS

The CBFEM-based parametric study evaluates localized strength enhancements for standard gravity-designed bolted extended end-plate connections. The following conclusions are drawn:

1) Validation Fidelity: The IDEA StatiCa CBFEM model precisely replicates EN 1993-1-8 analytical values for the baseline connection, validating its appropriateness as a tool for systematic parametric assessment, with discrepancies remaining below 0.3%.
2) Prying Action Mitigation: Increasing the end-plate thickness from 40 mm to 50 mm limits plate bending stiffness, directly curbing secondary prying action and resulting in a 17.8% reduction in maximum bolt tension under capacity design demand (LC2).
3) Geometric Constraints: Upsizing the bolt diameter from M36 to M40 or M42 is geometrically infeasible for the IPE 500 configuration. This underscores the necessity of assessing code-mandated edge/end distance overlaps prior to initiating analytical bolt-diameter optimizations.
4) Flange & Web Control: Increasing horizontal continuity stiffeners from 50 mm to 60 mm reduces bolt forces by 15.3%. This is achieved primarily by suppressing local column-flange bending and column-web panel shear deformations.
5) Failure Hierarchy Shift: The combined optimal retrofit (50 mm end-plate, 60 mm stiffener, M36 10.9 bolts) successfully satisfies all Eurocode utilization checks. The connection achieves the intended capacity design hierarchy, shifting the failure mode from abrupt, brittle bolt rupture to desirable, ductile beam yielding.
6) Cost-Effective Retrofitting: The upgrade is executed entirely through local connection modifications (plate replacement and stiffener welding) without altering the primary structural members. This represents a highly cost-effective seismic strengthening strategy for vulnerable gravity-designed steel connections.

Future investigations should expand the parametric space to include diverse column and beam sizes and integrate reversed-cyclic loading protocols to analyze the low-cycle fatigue capacity of the upgraded configuration.

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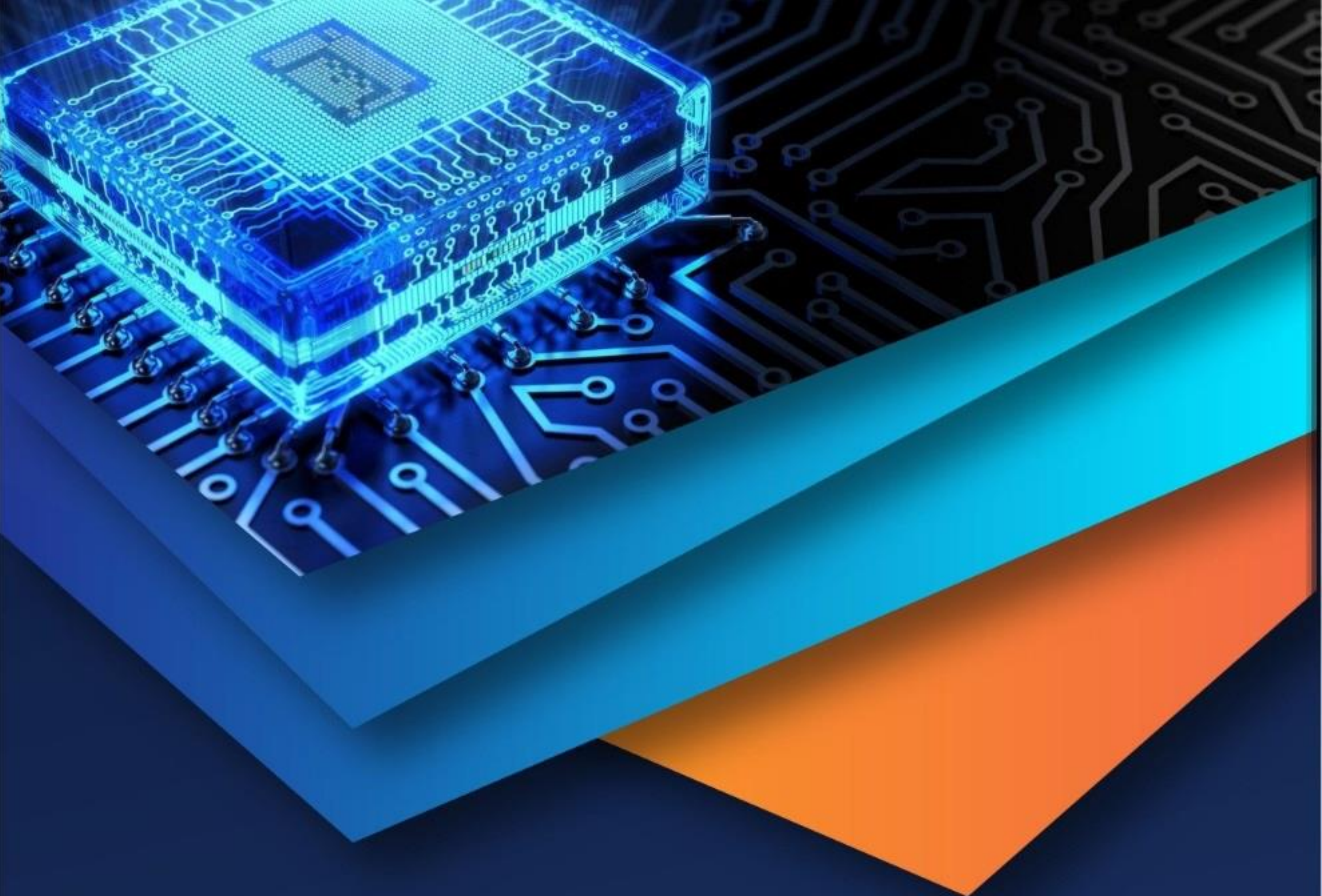


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