IMPACT OF DESIGN AND CONSTRUCTION ERRORS ON THE STRUCTURAL RELIABILITY OF STEEL INDUSTRIAL BUILDINGS

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Abstract

Errors in design and construction critically undermine the structural reliability of industrial buildings, putting property, the environment, and human safety at risk. In this regard, the present research work is intended to investigate how such mistakes influence the performance of the main structural components and the stability of steel industrial buildings. Detailed finite element analysis was performed using DIANA FEA for solid modeling and SAP2000 for beam modeling to assess global structural performance. This includes, among others, the insufficiency of local reinforcement in compressed members and eccentricity in column connections. It was performed to analyze the local and global buckling behaviors, deviations in symmetry, and inefficiency of the bracing systems. Consequently, it reveals a significant reduction in load-bearing capacity due to reinforcement deficiencies in the compressed elements and eccentricity, while a structural loss in integrity becomes highly significant at symmetry deviations, especially in horizontal loads. This study provides critical insights into mitigating design and construction errors to enhance the reliability of industrial steel buildings.

Keywords: steel structures, finite element method, finite element analysis, structural performance, structural design, reinforcement, buckling, symmetry deviation

I. Introduction

Steel structures have many advantages in the construction of industrial buildings in terms of tensile strength, ductility, and durability. That is, their versatility, design flexibility, and efficiencies realized from pre-engineered building systems make them necessary to meet the demand for industrial requirements. The possibility of covering large areas without interruptions, with reasonable consumption of materials and resistance in the case of earthquakes, makes steel superior to other materials for construction in this industry [1]. The environmental benefits of industrial building construction are necessary, and steel structures are imperative. The sustainability and recyclability of steel, coupled with the industry's efforts to reduce carbon emissions and increase energy efficiency, reflect a greener construction sector. By leveraging the many advantages that steel has to offer, today's construction industry has the possibility of constructing environmentally responsible buildings that will meet any modern industrial process while minimizing the impact on the planet [2]. Although they offer several advantages for industrial building construction, steel structures encounter problems that should be considered during the planning, design, and maintenance stages. It is through the knowledge of these disadvantages associated with steel, such as its susceptibility to corrosion, fire resistance concerns, and initial cost considerations, that appropriate strategies for maximizing its benefits and mitigating the associated drawbacks can be designed by the project stakeholders at different levels. These shortcomings have been addressed earlier, making steel versatile, resilient, and able to stand up to or meet all modern requirements expected of contemporary building projects in any industrial construction [3].

Steel structures of large-span industrial buildings were designed as volumetric systems. In the soil, all columns were fixed to foundations, and the top was fixed to trusses, thus creating a transverse frame. Transverse frames are linked together by means of bracings and purlins, and such a design ensures volumetric rigidity of the frame. Large-span industrial buildings are designed to accommodate industrial processes and to shelter them from open environments. Therefore, the structures of industrial buildings are designed to withstand environmental impacts such as snow, wind, dust, seismic, and service loads considered by industrial operations inside these buildings, including live loads, crane loads, explosions, and temperature fluctuations [4].

The design of large-span industrial buildings and their loads and influences are regulated by national codes and general engineering practices [5]. Errors during the design and construction phases of large-span industrial buildings may contribute to the disruption of industrial operations, jeopardy for personnel health, and the total failure of structures. According to statistics, errors causing the malfunctioning and collapse of industrial buildings can be divided into two groups: errors during the design phase and errors during the construction phase. According to statistics from to 1993-2004, the following average distribution of causes of accidents was observed: defects in construction and erection works, 44%; violation of operation rules, 24%; poor quality of materials, 15%; excess loads and external impacts, 5%; erroneous design decision making, 4%; and other causes, 8%. Thus, approximately 60% of accidents are associated with the construction stage, including critical defects in the construction facilities and materials used [6]. The distribution of defects in a construction can be represented by the following statistics [7]: By the reasons of defect origin: design error - 4%; poor quality of materials and products - 17%; low quality of installation work - 42%; operation deficiency - 18%; combination of reasons - 19% [6].

Other researchers have focused on the structural failure of steel structures due to (quasi-static) loads during the erection-construction phase, service phase, and fatigue failure. In most cases, structural failure is caused by gross human errors. Human errors in the execution of steel structures have been identified as a cause of failure in comparison to failures originating from errors in the design process. The primary source of fatigue destruction in steel structures is insufficient welding size or inadequate quality of the welding joints. According to previous studies, fatigue cracks nearly always originate at the toes of a weld, namely at the transition point between the weld and base material, or at the edges between individual weld passes [8-10]. Although this may be accounted for by the combinations of an unlucky set of variations influencing parameters affecting the actions and responses of the structures in probabilistic and semi-probabilistic design methods, significant human error is the main cause of the failure of steel structures.

Geis et al. studied 1029 cases of snow-induced building failures in the United States from 1989 to 2009 and 91 international cases in 16 countries across four continents from 1979 to 2009 [11]. Paper archives provided data for this study from 1345 articles from 883 distinct sources. They found that the primary causes of snow-related building failure were excessive snow (89% of total incidents), rain-on-snow events (13% of total incidents), and building problems (9% of incidents). As buildings age, their structural members deteriorate and may be damaged. A higher percentage of incidents was attributed to building problems in older buildings: 28% of historic buildings and 26% of middle-aged buildings, while only 5.4% of new buildings were part of the U.S. dataset. Other contributing factors listed in the surveys included melting and drifting snow, drainage, and people living on the roof. In addition, Alinaitwe and Ekolu identified several causes for the collapse of structures during the construction phase, including poor materials and workmanship, design and construction errors, lack of professional supervision of site work, improper implementation of construction methods, and neglecting design approval procedures. They argued that construction failures could be prevented if proper procedures were followed during the design, construction, and operation of structures [12].

Oloyede et al. conducted extensive research and collected data by administering questionnaires to professionals in the building industry, including contractors, builders, architects, estate surveyors and valuers, civil engineers, electrical engineers, structural engineers, and town planners [13]. The data were analyzed using descriptive and analytical statistics. Their findings indicated that the causes of building collapse included soil type, poor building design and planning, use of low-quality building materials, and employment of incompetent craftsmen, resulting in poor workmanship, weak supervision, natural disasters, and corruption.

It is envisioned that active involvement and quality input by professionals in the building industry from design to construction, including supervision at each stage, would be vital to ensure adherence to standards and procedures [14]. It is becoming increasingly difficult to learn from mistakes in the prevention, detection, and limitation of design errors. Kamara et al. [15]. observed that most failures and their ensuing damage are rooted in planning, design, construction, and use errors rather than in construction material variability, strengths, and structural loads. It should be borne in mind that design is the first step in construction, and design faults have been the fundamental cause of many disasters, leading to the death and injury of workers and the public [15].

Recent advances in computer-aided design and high-strength materials have optimized modern structures compared with their ancestors. However, optimization reduces the inherent margin of safety; thus, modern structures have a minimal excess capacity to handle unexpected loads. Therefore, modern structures are prone to unexpected loading [16]. Gross and McGuire also indicated that new construction forms contributed to increased vulnerability, which reduced costs but sacrificed the strength and continuity inherent in older construction forms [17].

A few studies have examined inaccurate results for the numerical modeling of truss nodes. Truss collapse was found to be a result of incorrect node design. The control results of numerical calculations by analytical calculations of statically determinate systems were proposed, which will help determine the error at the initial stages of design [18].

The possible modes of structural failure can be broadly classified into three types: large local plasticity, instability, or fracture. Large local plasticity indicates that deformation of the material occurs over and beyond the elastic limit and is often accompanied by the prospect of large plastic deformation. Instability is the occurrence of a sudden change in the response initiated by a load, which results in bifurcation, branching, or non-bifurcation. It is usually initiated by direct tensile rupture, fatigue failure, and brittle fracture and is a critical failure mode. In real structural failures, a mixture of these failure modes occurs, with special consideration given to the interaction between the large local plasticity and instability. Normally, instability precedes or acts alternatively to plastic action, leading one member to have less strength and stiffness under given circumstances into a hinged mechanism. Both failures can be better explained by examining the load-displacement behavior characteristics of each type. Depending on the type of member, loading conditions, and type of support, the load-deflection curves or instantaneous stiffness of the member are perhaps the single most important parameters used in the prediction of load-carrying capacity and stability [19].

These notable cases can serve as valuable tools in engineering education; however, they might misleadingly suggest that only large and uncommon structures are prone to failure. Moreover, incidents that do not result in the loss of human lives, which constitute the majority, can offer valuable insights into engineering malpractice, demonstrating that any area of engineering practice can present complex challenges [18]. The general structural performance of steel industrial buildings can primarily be evaluated by analyzing the individual structural elements and overall structural performance. A case study was conducted to assess the performance of compressed structural elements and the overall performance of the structure.

II. Methodology

This study examined two cases of errors and their impact on the structural performance of industrial buildings. In the first case study, construction errors, such as a lack of local reinforcement, large eccentricities, and symmetry deviations, are analyzed; in the second case, design errors, such as insufficient bracings and insufficient cross-section size, are analyzed.
An industrial steel building, length 23.5 m, free span 20 m, clear height 20m, was designed and constructed in the Republic of Azerbaijan in Baku city. The purpose of this building is to serve as a painting and blasting chamber for the ship hull blocks. The building is designed and constructed as a volumetric structure. The transverse frame consisted of steel columns from the H section of 300 × 300 × 17 mm and a truss. The truss elements were all double angles: upper and lower chords 75x75x6 mm, vertical bars 75x75x6 mm, bracing 75x75x6 mm and 50x50x6 mm. Purlins Channel 150x75x10 mm and an angle of 95 × 75 × 7 mm. Vertical bracing between columns from cross angle + 100 × 100 × 10 mm, vertical bracing between trusses from double angle 75 × 75 × 6 mm. The steel used had a yield strength of 235 MPa and tensile strength of 400 MPa for all elements. The building was considered under the Azerbaijan National Construction and Design codes and standards [20].

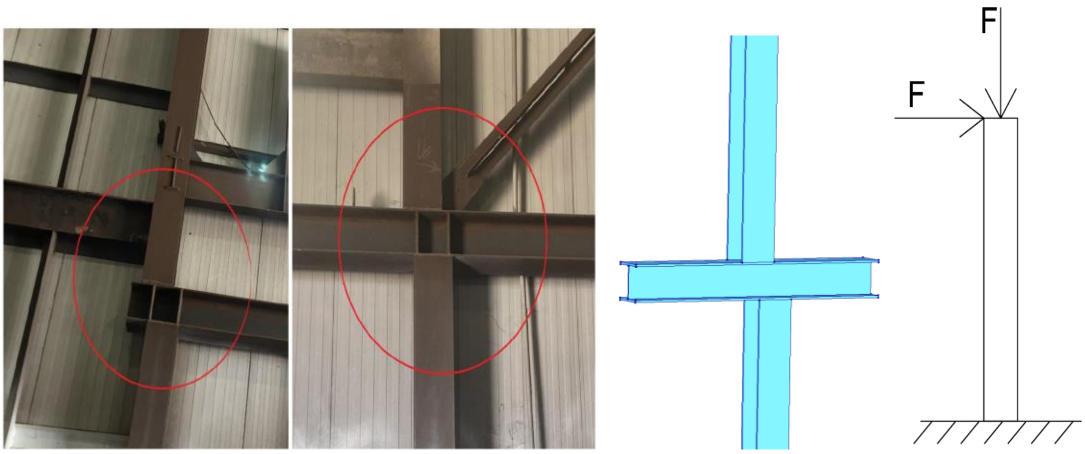
2.1. Lack of local reinforcement and eccentricity connections in column

In engineering practice, tall steel columns are reinforced with brackets (Fig. 1). To study the performance of the columns, FE models were created and subjected to 3D performance analyses. The first model was created based on the actual on-site conditions without reinforcement (Fig. 1). The second model was created with brackets installed every 1 m [21,22].

Large eccentricities in the column end-to-end connection introduce two forces that create moments. This moment induced additional stresses in the column, which were superimposed on the axial compressive stress. To study the performance of the columns, three FE models were created and subjected to 3D performance analyses. The first model was created according to actual onsite conditions with an eccentricity of 100 mm (Fig. 2a). The second model was created according to the actual onsite conditions with an eccentricity of 150 mm (Fig. 2b). The third model was created without eccentricity (Fig. 1b). In both cases, the boundary conditions were applied as a fixed supported column with axial and lateral forces at the top tip, as shown in (Fig. 2c).

(a) (b) (c)

Figure 1. *Column of industrial building: (a) actual conditions on site; (b) FE model of steel column without reinforcement; (c) FE model of steel column with reinforcement.*

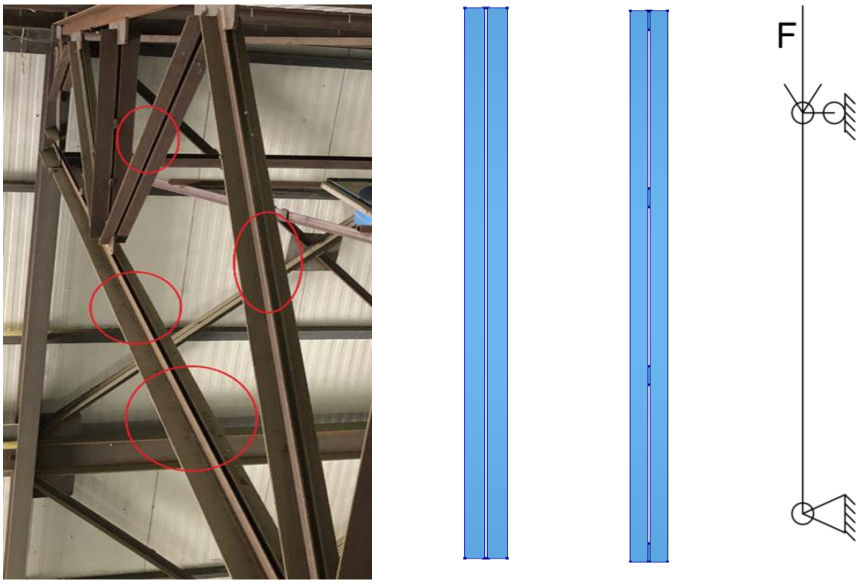


(a) (b) (c)

Figure 2. *Column connection eccentricity: (a) actual conditions on site, (b) FE model and (c) analysis scheme*

2.2. Lack of Gusset plates in the compressed members of the truss

A double-angle truss, also known as a double-angle truss, uses pairs of angle sections connected to form truss members. Joining plates (gusset plates) between the angles are used to connect these members at the joints, ensuring structural integrity and proper load distribution. Compressed truss members are particularly susceptible to buckling. Gusset plates provide lateral support to these members, reducing their effective length and increasing their buckling resistance. Without the gusset plates, the unsupported length of the compressed members increases, making them more prone to buckling under axial loads. The insufficient use of gusset plates may compromise the overall stability of the truss. To study the performance of all the compressed members, they were FE modeled and subjected to 3D performance analyses. Two different models were created for each compressed element: one model expressing real onsite conditions without gusset plates and the other with gusset plates. The boundary conditions in both cases were applied as a pin-supported element with an axial force applied at one end, as shown in (Fig. 3).



(a) (b) (c) (d)

Figure 3. *Compressed truss elements: (a) actual condition on site, (b) without gusset plate, (c) with gusset plate and (d) analysis scheme*

Finite element analyses (FEA) of the structures were conducted using DIANA FEA software. The most comprehensive analysis was provided by the 3D solid element model (Fig. 4a), which examined both the longitudinal and transverse behavior and accounted for the entire three-dimensional response [21,22]. The steel material model was based on the Von Mises plasticity with linear plastic hardening (Fig. 4b). based on the experimental properties [23].

In the analysis stage, an increment in the point load was applied to the steel column up to the maximum tensile stress. Features associated with the load-displacement behavior were modeled in solid modeling, whereby at every step, 100 increments of point load equal to a factor of 3.5, were applied in the numerical models of the behavior under study for load–displacement behavior. The mesh topology used in the analysis was quadratically shaped. For the analysis of the columns, a mesh size ranging from 50 to 60 mm was employed, while a finer mesh size of 25 mm was utilized for the analysis of other structural elements. Essentially, it results in a spurious outcome when an incremental primary method is used through the DIANA program, unless very small steps are taken. This is because nonlinear systems are very sensitive and taking larger steps in an incremental analysis result in numerical instability and a loss of convergence. This limitation can be determined using an iterative approach. In the numerical simulation, the applied loads on the elements increased gradually up to the point of the tensile stress. First, the response of the column was linear elastic; there was a direct relationship between any applied load and the deflection thereof. An updated arc-length control method for the normal plane is used to capture the nonlinear response of the column. This control mechanism ensured that matching of the analysis was performed for the reply because the element exhibited nonlinear behavior, such as peak tensile stress.

The control method allows the increment of the load at each step to adjust to the evolving element behavior in a dynamic manner through its automatic load step-scaling function. Thus, adaptive scaling improved the nonlinear analysis convergence and made it possible to simulate the reactions of the columns more realistically at failure. The numerical analysis in the present study was performed by employing the Newton-Raphson method adopted with the Line Search Approach, with a selected default value of 25 iterations. The selected numerical technique was unconditionally stable and numerically efficient for analysis.

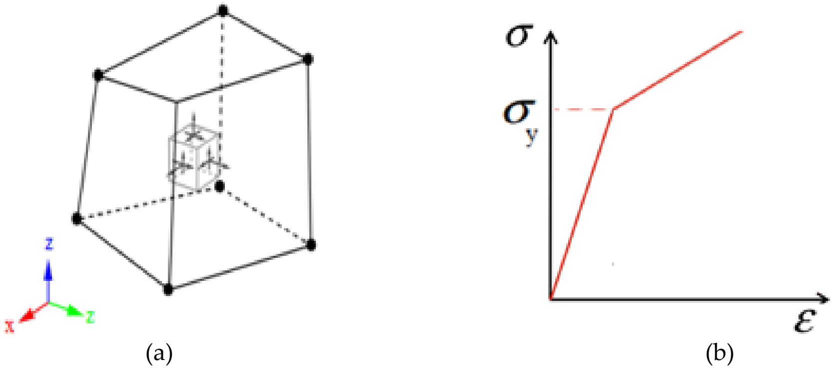


Figure 4. *Element and material model for the FE analysis: (a) 3D solid model, (b) Steel model*

2.3. Insufficient bracings and insufficient cross sections size

Inadequate and insufficient bracings were determined by inspection of the blasting chamber. Insufficient bracings and cross-sectional sizes in industrial steel structures can lead to serious structural issues, compromising the overall stability of the structure. This may increase vulnerability to buckling under loads. Without adequate bracing, the structure may experience excessive deflection, which can lead to misalignment and damage to structural and nonstructural elements. The bracings reinforcing the lower cord of the truss were completely missed. Vertical bracings connecting trusses in the middle of the span were applied incorrectly only in the 1st step, whereas vertical bracing between columns was applied in the 2nd and 4th steps. To study the impact of insufficient bracings and insufficient cross-section size on the performance of industrial buildings, two models were created using SAP 2000 software. The first model was created following actual on-site conditions with insufficient bracings and insufficient cross-sectional size. The second model was created with proper bracing according to AzDTN II-23-81 [20]. Both models were exposed to 3D performance analyzes. Pushover analyses were conducted for the overall structure, and the reading point for the results of the pushover curve was considered at the top of the column.

In this study, the method used for static pushover analysis was nonlinear static pushover analysis, which was conducted with the nonlinear version of SAP2000 software [26] applied to three-dimensional structural models. The model was designed as a beamline. The analyses were performed under displacement-controlled conditions until the specified displacement level was reached in the direction of the control and at the control point. In the case of SAP2000, the frame elements were modeled as linear elastic line elements, whereas the nonlinear force–displacement behavior for individual frame elements was represented by hinges through a series of linear segments [27, 28]. Hinge properties can be considered either as default or completely definable by the user [29-31]. Numerically, for user-defined steel moment and PMM hinges, the yield moment, yield rotation, and axial force-bending moment interaction diagrams could have been computed based on the section and material properties [32,33]. Thus, although user-defined moment-rotation relationships can result in various plastic rotation capacities and strain hardening ratios, the default steel moment and PMM hinges were used in this study for the pushover analyses of steel frames because of their simplicity. By default, pushover analyses using SAP2000 prefer to consider default hinge properties based on the ATC-40 and FEMA-273 criteria [34-36]. This is because specifying the cross-sectional properties of all members of a structure can make pushover analysis challenging to implement, especially when dealing with a three-dimensional structure.

2.4. Symmetry deviations

A blasting chamber was constructed with symmetrical deviations at each step (Fig. 5). Symmetry deviations in the steps of industrial building steel structures can have significant implications for the stability, safety, and functionality of the structure. Asymmetry can lead to an uneven load distribution, causing certain parts of the structure to bear more weight than designed, potentially leading to structural failure. Deviations can affect the dynamic response of a structure to loads such as wind or seismic activities, increasing the risk of resonant vibrations or instability. To study the impact of symmetry deviation on the performance of industrial buildings under seismic and wind loads, two models were created using the SAP 2000 software. The first model was created based on actual on-site conditions with symmetrical deviations. The second model was created without any deviation in symmetry. Both the models were subjected to 3D performance analyses.

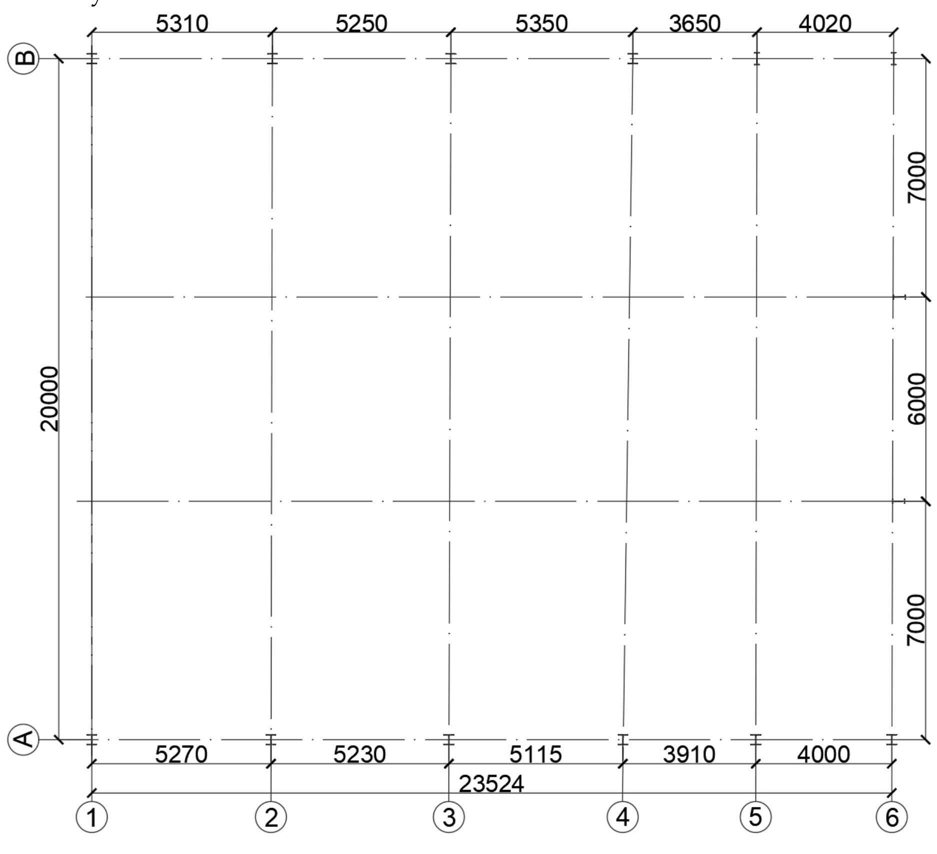


Figure 5. *Plan of existing Blasting Chamber*

III. Results and Discussion

In this research, using detailed case studies, the finite element modelling method was used to assess the load-carrying elements of an existing industrial building under several variables. The analyses focused on the load-bearing elements of existing structures. First, the current conditions of the industrial building were carefully checked, and the geometric and material properties of the structural elements were detailed. Subsequently, a data-driven finite element model was created, and performance analyses were performed under various loads. These were conducted for case studies representing the worst-case scenarios of different loads commonly encountered in industrial buildings. Detailed analyses were performed for the response of the load-bearing elements concerning various parts of the structure and conditions relevant to events when the critical performance limits were exceeded. The results from the analysis clearly demonstrate the performance of the structure under load and the capability of the structure to maintain its integrity. Specifically, cases of overloading and scenarios in which the structural responses exceeded the safety limits for deformation and stress distributions were investigated

3.1. Columns

The deficiencies of the local reinforcement and eccentric column connections were studied in this work. FE models were generated to investigate the column performance, and 3D performance analyses were carried out. First, a model without strengthening was created using existing site conditions. In the second model, strengthening was performed using additional brackets every 1 m. The results of the analyses showed that the peak tensile stress in the column without reinforcement was 400 kN, whereas that in the column with local reinforcement was approximately 500 kN. It can be seen from the load–displacement curve that after reaching the peak tensile stress, the column with local reinforcement did not collapse; although the displacement increased, it was still able to bear 500 kN. In contrast, for a reinforced column, a linear decrease in the bearing capacity with a peak value of the tensile stress was observed beyond the reinforced area, which was compensated by plate reinforcements, thereby improving the functioning of the column (Fig. 6). Hence, the lack of local reinforcement can reduce the structural performance of the column by approximately 20%.

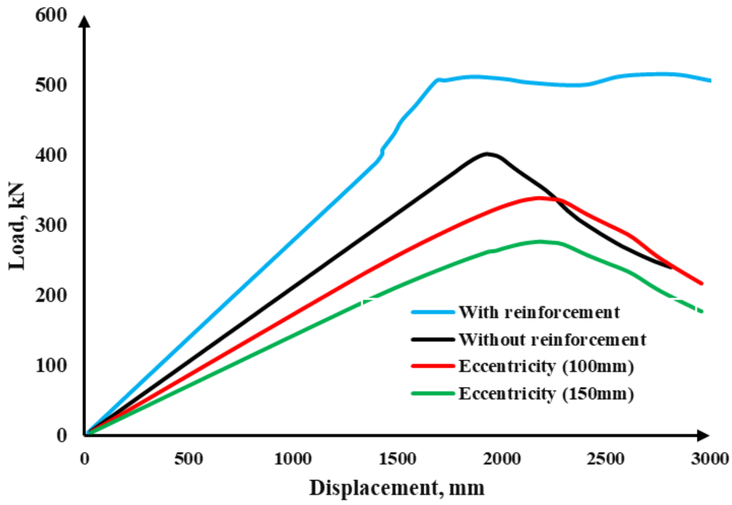
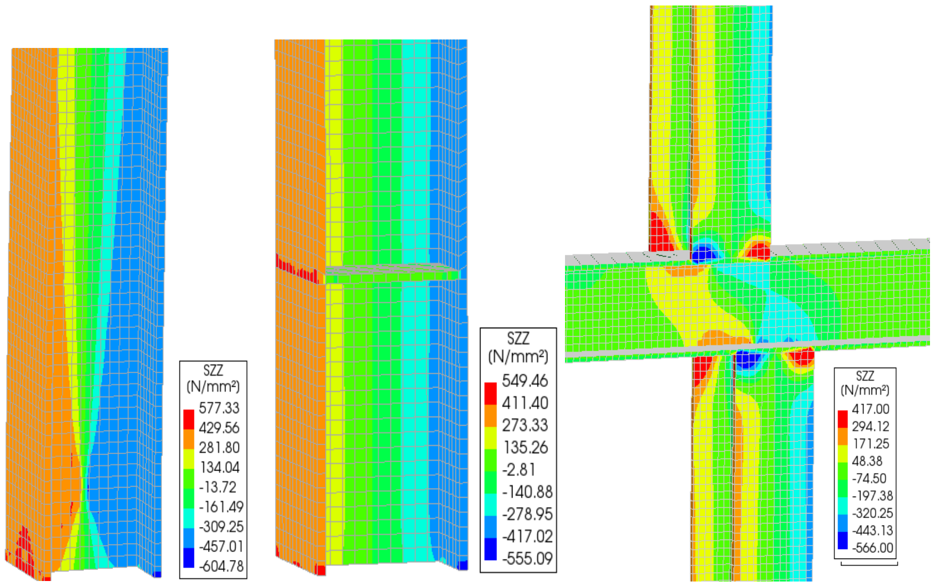


Figure 6. *Load displacement curve of column*

Large eccentricities in the end-to-end connections of the column generated large moments, which caused additional stresses on the column, adding to the original axial compressive stress. Column case studies under different applied eccentricity conditions and reinforcements returned results showing their behavior and structural performance. For example, as shown in Figure 6, in a similar column, the capacity decreased from 400 kN to 300 kN with an eccentricity of 100 mm, whereas for the other column with an eccentricity of 150 mm, the capacity decreased drastically to 240 kN. These produced moments, owing to the eccentricities, increased the stresses inside the column to dangerous values. Columns without reinforcement with 100 mm eccentricity lost 40% of their bearing capacity compared to the properly reinforced column without eccentricities. Columns without reinforcement with 150 mm eccentricity lose 52% of the bearing capacity compared to the properly reinforced column without eccentricities.

Figure 7 shows the stress distribution in the column for the three cases. The installation of the local reinforcement prevented the column web from buckling. Reinforcements provided additional support for the web, which helped distribute stress in a more uniform manner. Figure 7a shows that in the absence of reinforcement, the stress is concentrated in the web of the column. This occurred because the column lacked additional reinforcement, which increased its rigidity, allowing the stress to be distributed more evenly. Consequently, areas with high stress values developed weak points that were susceptible to damage and eventual collapse. In contrast, Figure 7b shows that in the reinforced column, the stress distribution between the two flanges is even. The stress spread uniformly over the cross-section, reducing the likelihood of local stress concentrations. This improves the overall stability of the column and minimizes the risk of buckling or other failures. Consequently, higher loads can be applied to the reinforced column, which remains structurally stable for a longer period.



(a) (b) (c)

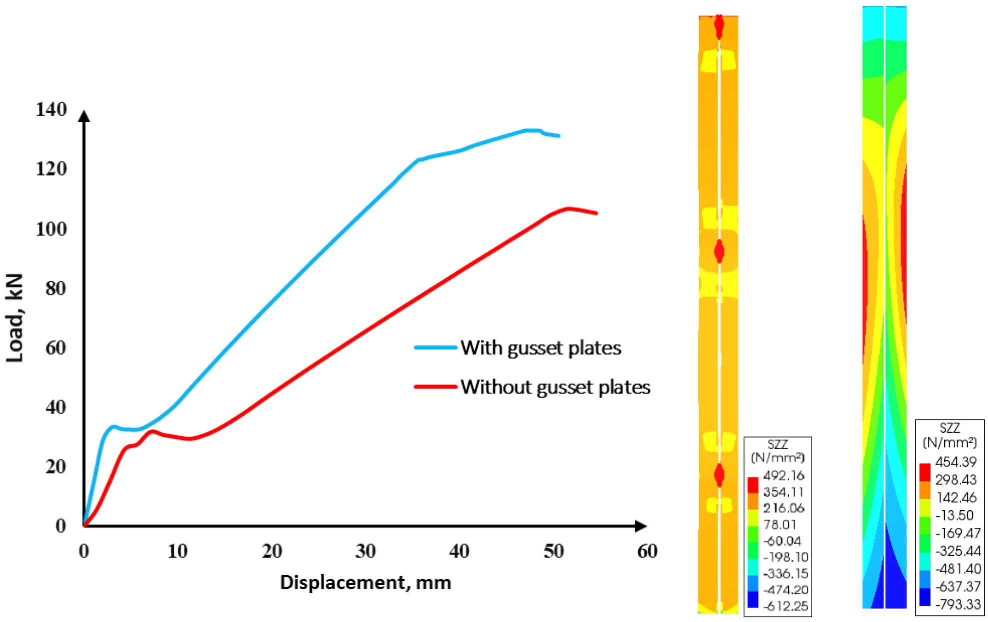
Figure 7. *Stress distribution diagram: (a) actual on-site conditions, (b) with reinforcement, (c) large eccentricities column*

In the case of column end-to-end connections with eccentricity (Fig. 7c), the stress distribution at the critical section became somewhat complicated. Eccentricity is considered to be the lateral deviation of the load path from the central axis of the column, leading to additional stresses owing to the bending moment created. At the failure point, a highly irregular stress distribution prevails, leading to a compound failure mode. Consequently, several types of failures can occur simultaneously or sequentially. The added eccentricity further complicates the already intricate stress distribution, making it more difficult to predict and mitigate the potential failure zones.

3.2. Truss compressed elements

In this study, detailed analyses were conducted to examine the performance of a double-angle truss. The analysis results explained the differences in the structural performance of the compressed elements with and without gusset plates. A significant increase in the buckling resistance of the elements was observed when they were supported by gusset plates and the integrity of the structure was retained. The obtained results for the load displacement showed that the rigidity value in the elastic region of the curve was higher when the truss was provided with gusset plates. For the models without gusset plates, the effective length of the elements was increased, which already brought danger by way of buckling and risked a decrease in structural stability. Specifically, as shown in Figure 8a, the behavioral change in both trusses came at 38-40 kN of load. In terms of performance, one can see that the element with gusset plates stood at a maximum force of 125-130 kN compared to the one without plates, which only reached a maximum of 105-110 kN. This proves that the gusset plates significantly improved the buckling resistance and increased the rigidity of the structure.

As shown in Figure 8b, the double-angle profiles with the gusset plates have an even stress distribution along the element. The application of the gusset plates significantly increased the buckling resistance by approximately 15-16%. In the truss elements without gusset plates, as shown in Figure 8c, buckling started at specific points of the element, and the profiles did not work efficiently together. This causes high stresses in some areas and buckling problems in others within compressed elements.



(a) *(b)* *(c)*

Figure 8. *Load displacement curve (a), stress distribution diagram of compression truss elements with (b) and without (c) gusset plates*

3.3. Symmetry deviation and inadequate bracing

The results of the static pushover analysis for the current state of the steel industrial building compared to its design according to standards provide valuable information about the volumetric performance of the structure. It has been observed that the structural performance of existing steel building is 550 kN. This value increased to 690 kN in the building designed according to the standards. This indicated a performance difference of approximately 20% (Figure 9). Furthermore, the load-displacement curve shows a significant difference in the formation of the initial hinges in both structures. In the examined existing structure, the first hinge formation in the first column occurred at a load of 450 kN, whereas in the building designed according to the standards, this value increased to 540 kN. This demonstrated an average performance difference of 17% between the two structures. Despite these differences, the collapse modes of both buildings exhibited relatively consistent behavior and similar patterns. These results demonstrate that designing according to the standards significantly improves the overall performance of the structure and provides a safer building.

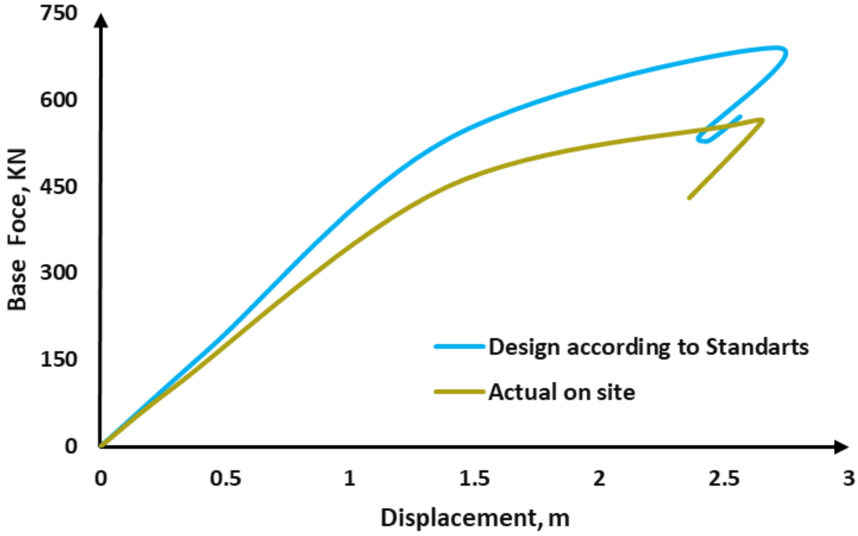


Figure 9. *Pushover curve of the overall steel structure*

IV. Conclusion

This study presents the impact of design and construction errors on the structural reliability of industrial steel buildings, focusing on critical issues, such as insufficient local reinforcement, eccentric connections, symmetry deviations, and inadequate bracing systems. The results indicate the serious consequences of these errors at both the individual structural component and overall building performance level. In the case study, the bearing capacity was reduced by approximately 20% in tall columns owing to insufficient local reinforcement. In the case of no reinforcement with an eccentricity of 100 mm, the bearing capacity was reduced by approximately 40% compared with properly reinforced columns without eccentricity. This was further reduced by approximately 52% for a 150 mm eccentricity. Furthermore, the proper application of gusset plates in the compressed truss elements increased the buckling resistance by approximately 15–16%. Deviations in symmetry and scanty bracing systems resulted in significant structural performance when horizontally loaded. An industrial building built with such a scarcity performed 20% below the standards. These directly point to an urgent need to pay more emphasis in detail by design and during construction to establish how reliably safe such structures in the form of steel industrial buildings can be achieved, keeping up with common flaws in integrity and performance expectations.

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