ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
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PARAMETRIC STUDY ON SHEAR CAPACITY OF PLATE GIRDERS WITH WEB OPENINGS

A Thesis in Structural Engineering

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A Thesis
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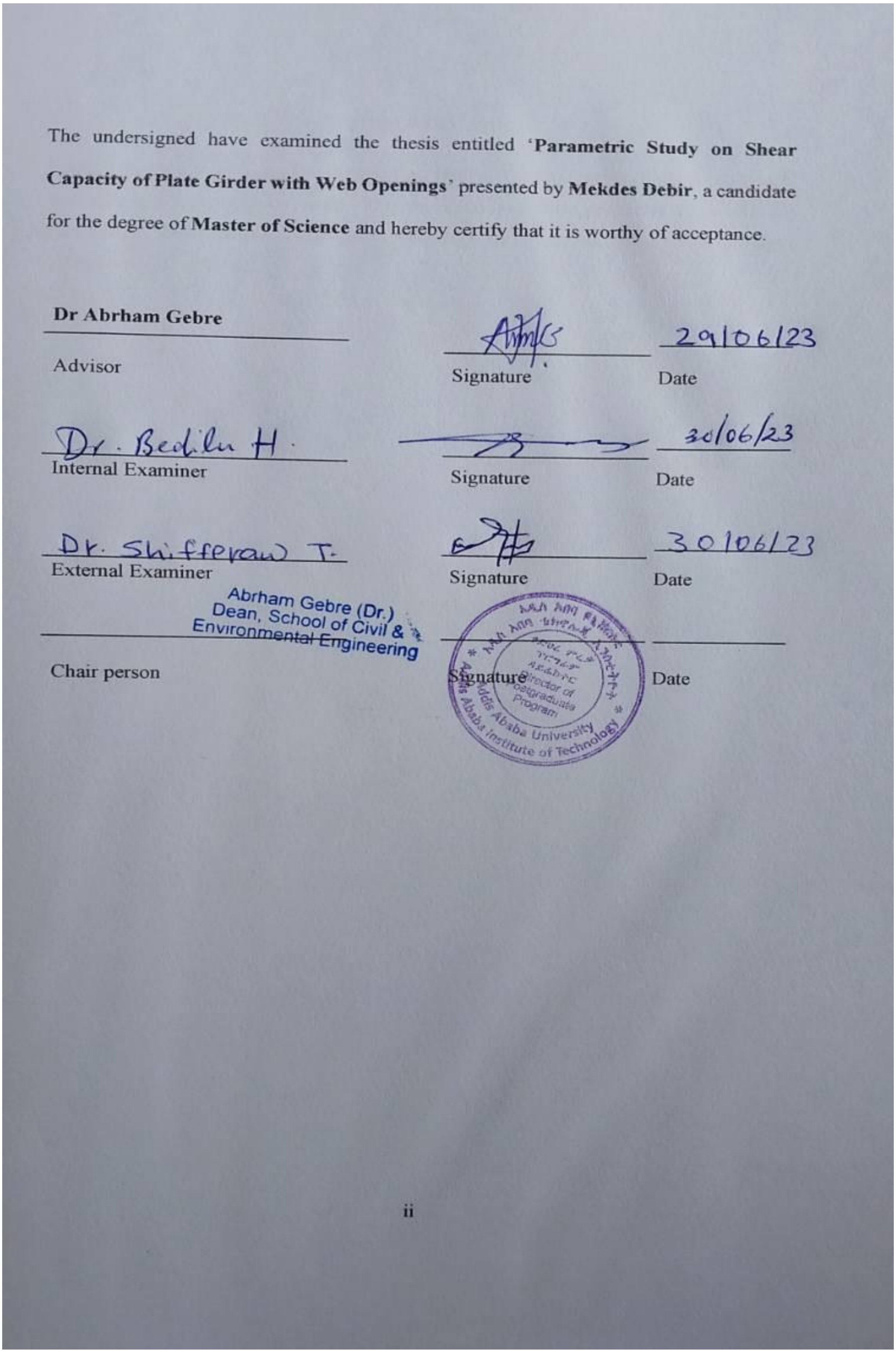

The undersigned have examined the thesis entitled '**Parametric Study on Shear Capacity of Plate Girder with Web Openings**' presented by **Mekdes Debir**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I certify that research work titled “parametric study on shear capacity of plate girder with web openings” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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ABSTRACT

This thesis studies parametrically the ultimate shear capacity of plate girders with web holes. The variables include the span length, the opening depth, the opening number, the opening location, and the opening form. This study will provide guidelines for which form of openings to use, how deep to cut the web, where to cut the web, and how many openings to use when web openings are required to pass utility lines and for inspection purposes without adversely reducing the plate girder's ultimate capacity. The finite element software, ABAQUS, was employed as a practical design tool. The accuracy of the developed model is evaluated by comparing its predictions with tests that were carried out experimentally on plate girders, which showed a good agreement in terms of deformations and ultimate loads.

The results demonstrated that the behavior of plate girders with web openings is significantly influenced by the shape and depth of the opening, particularly when the opening depth exceeds 50% of the web depth and the opening takes on a square shape. SPGs with two holes each situated up to two times the web depth from supports showed no discernible impact on the ultimate capacity. Increasing the number of openings resulted in additional mid span deflections and a considerable reduction in ultimate capacity. The depth of the opening was found to be the most influential parameter in the sensitivity analysis. The ultimate capacity of the weakest SPGs with web openings is greatly increased by providing steel plates around the openings.

Keywords: Steel plate girder (SPG), Web openings, Ultimate capacity, Parametric study

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Lord, I wanted to thank you for everything you have given me. It may not always be a trouble-free or challenge-free life, but I am grateful that it is my life, and with your support, it has shaped me into the person I am today. I can face the future with confidence, and I know that you are with me always.

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LIST OF ABBREVIATIONS

a - Spacing between transverse stiffeners

c – Outstanding width of flange

d - Depth of Web

D - Depth of Girder

do - Opening Depth

E - Modulus of Elasticity

E ‘- Tangent Modulus

Fy - Yield Stress

Fu - Ultimate Stress

L - Span Length

LVDT- Linear Variable Differential Transformer

SPG - Steel Plate Girder

tf -Thickness of Flange

ts -Thickness of Stiffener

υ - Poisson’s Ratio

τ - Shear stress

CHAPTER 1 INTRODUCTION

1.1 Background

In the absence of structural steel, the world would be very different. There would be no skyscrapers creating unique patterns against the sky. Buildings would only have a few floors and would make up extra square footage by expanding in length and width. Cities would grow considerably more rapidly than they do now. Structures made of materials other than steel could not endure earthquakes and other severe weather that the earth can produce. Main advantages of steel structures include safety, reduced construction cost, future adaptability, high quality construction, serviceability, design flexibility and sustainability.

The steel plate girder is a flexural member that is constructed via bolting, welding, or riveting. Typically, the SPGs consist of two flange plates and one web plate that are joined together to form an I-shaped cross-section. In addition to performing appropriately under the imposed stresses, SPGs are more affordable than rolled steel beams. The top and bottom steel flange plates' primary purpose is to withstand bending moments. The axial tensile and compressive stresses that result from the bending action is what give rise to this resistance. whereas web plates preserve relative separation between flanges and resist shear stresses. The thin steel web experiences shear buckling as a result of being under enormous shear forces.

The SPGs' web is used in thinner thickness than their flanges, which reduces their weight and has a good effect on their structural performance. On the other hand, due to the thin web, the SPGs buckle or yield under the shear stresses before reaching their maximum flexural strength (Bahrami, A., Najarnasab, 2020).

SPGs are frequently used as primary structural components in

- Bridges and overpasses with comparatively large spans
- Ordinary buildings with longer spans
- Frames of industrial structures

Large construction depths between floors may result from placing large ducts and pipes beneath structural steel beams and girders in building construction. Creating openings in the webs of the beams and girders is the most popular solution to provide the required space for service. Web holes on these girders are utilized in highway bridge construction to allow space for service, inspection, and maintenance. These openings can have lengths that are two or three times the opening depths and occasionally have depths that are up to 60% of the beam depth. There are many applications for circular and rectangular openings. In order to reduce or eliminate stress concentration, rectangular openings typically have curved edges (N. E. Shanmugam, Lian and Thevendran, 2002).

Greater deflections than equivalent beams with solid webs are caused by the introduction of an opening in the girder's web, which alters the stress distribution inside the member and influences the behavior of its collapse. The plastic deformations that take place at the openings will determine the plate girder's strength. Thus, one of the crucial factors in the construction of modern structures is the effective use of beams and plate girder sections with web holes.

The presence of web openings reduces plate girders' local buckling and shear capacity. The shear and bending capacities of the beam typically decrease as the height of the web opening increases. Alternately, increasing the web opening length decreases the perforated section Vierendeel collapse capacity by increasing the local Vierendeel bending force acting on the "Tees," but has little effect on the beam shear and bending capacities (Rodrigues et al., 2014). These factors served as the driving force for the current investigation, which used FE models calibrated using test and numerical data. A thorough parametric analysis of beams with web openings that was centered on the profile size, web opening depth, location, and shape, among other factors, was made possible by the correctness of the results. The effectiveness of plates welded at the opening region, transverse stiffeners, and longitudinal stiffeners was also examined in this work.

1.2 Statement of the problem

When a big load needs to be carried with more stability and fatigue resistance, plate girders are required. Solid plate girders prevent utility lines such as huge pipes, hydraulic and ventilation pipes, electric wires, and air conditioning ducts from passing through industrial or high-rise buildings. As a result, more room will be required, increasing the floor height will add extra expense, and integrating holes in plate girder webs will be a solution. Web openings in the girders of highway bridges are also required to create space for services, inspections, and maintenance. Web openings drastically lower the ultimate shear capacity of the web; as a result, it is important to carefully consider where to place openings, what shapes and sizes to employ, and how to improve the strength after constructing the opening. In order to address the aforementioned issue, this study employs working numerical investigations and parametric studies employing three different spans with various opening positions, hole sizes, number of openings, and opening shapes for a total of 45 simply supported SPGs.

1.3 Objectives of the study

1.3.1 General objective

The main objective of this work is to investigate and compare, through finite element analysis, the ultimate shear capacity of perforated steel plate girders of different spans with circular and square openings varying in size and location.

1.3.2 Specific objective

- Studying the effect of shape and size of the web opening on the ultimate shear capacity of SPG and identifying effective shape and size.
- Assess the results of variable opening locations from the support and indicate appropriate locations to cut.
- Investigate the effect of number of openings on the shear capacity of SPG.
- Considering various methods for increasing the capacity of perforated SPG and recommending better corrective measures.
- Performing a parametric study and identifying the most sensitive parameter
- Developing a general formula to calculate the reduced shear capacity of a steel plate girder

1.4 Significance of the Research

Designers and practitioners can potentially benefit from the output of this research by getting essential guidelines about where to cut a web, which shapes and sizes are effective, and what to do when the capacity of the web decreases due to the openings in the case of the design of structures (bridges, buildings, cranes, and so on) for shear using plate girders.

1.5 Scope and limitations of the study

To explore the ultimate shear capacity of SPG with web openings, three simply supported SPGs with various span lengths of 15m, 20m, and 25m were employed using the commercial FE package ABAQUS (SIMULIA 2022). These SPGs had circular and square openings, and a monotonic load was applied at the center of the top flange.

Calibrated numerical models were used to carry out wider-ranging parametric studies to investigate the effect of location, depth, and shape of the opening on the ultimate shear capacity of the SPG. The study's findings only apply to welded I sections; rolled I sections are not included. The study took into account the same material properties for webs and flanges, and only shear capacities were studied.

1.6 Organization of the Research

This thesis is composed of five chapters, including the current chapter as the introduction about the SPG with web openings, and explains why the study is needed, what the objectives are, and what we expect as the output. Chapter 2 provides a survey of pertinent articles and quotes from a wide range of related works that have already been published in the field of study. The materials, methodologies, and analysis procedures used to complete the study are presented in chapter 3. The fourth chapter attempts to give a detailed description of how the software is calibrated against experimental work and examines the results of the current numerical analysis, including the parametric study. The study's conclusions and recommendations are presented in Chapter 5 as a final section.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

Plate girders are manufactured sections used to carry significant vertical loads over long spans where the associated bending moments are greater than the moment resistance of readily accessible rolled sections. The plate girder is a built-up beam that, in its most basic form, consists of two flange plates that are fillet welded to a web plate to create an I-section as shown in figure 2.1. The top and bottom flange plates' main job is to withstand the axial compressive and tensile forces brought on by the applied bending moments; the web's major job is to withstand shear. In fact, certain rules of practice base design on this division of structural action. By extending the distance between them, the necessary flange areas for a given bending moment can be decreased. Thus, it is advantageous to increase the distance between flanges for an inexpensive design. The web thickness should be decreased as the depth rises in order to minimize the girder's self-weight; however, this makes plate girders more susceptible to web buckling issues than rolling beams (Davison & Owens, 2008).

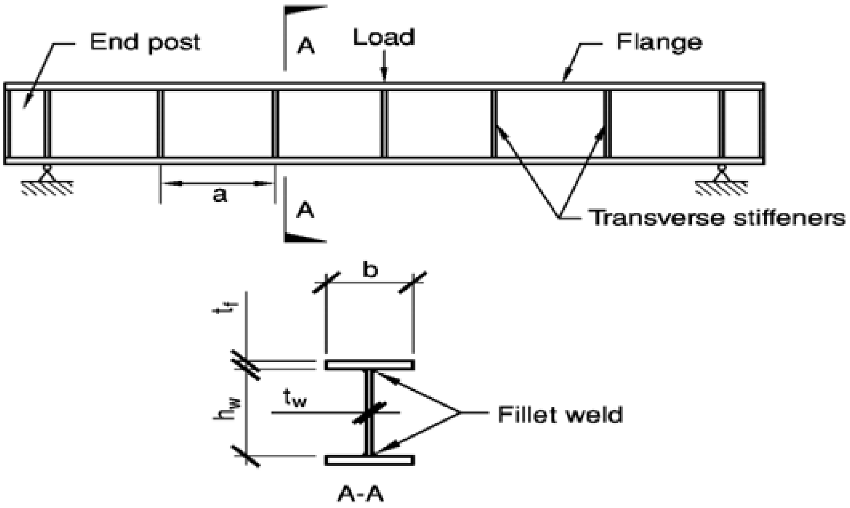


Figure 2-1 Elevation and cross section of typical plate girder

The designer of a plate girder is free to select components that are a practical size, but they must also offer connections between the web and the flanges. The manufacturing of plate girders allows for a special degree of flexibility, and the cross section can vary along the length of the structure. By altering the flange areas and maintaining the same girder depth, it is possible to supply the precise amount of steel needed at each segment throughout the length of the girder. Alternatively put, it can be molded to resemble the actual bending moment curve. The engineer's imagination is thus given an abundance of opportunities by a plate girder (Subramanian, 2010).

A reduction in steel consumption results from this modification of the cross section. A girder with an excessive number of cross section configuration changes may represent one that uses the least amount of metal, but it may not be the most cost-effective one because of the higher production costs associated with extensive variation in plate sizes. During the design stage, while determining the component sizes, this element needs to be given extra consideration (Welded Steel Structures," 1953).

When utilized to build rail road bridges in the late 1800s, plate girders gained widespread acceptance. To create plate girders of the necessary size, the plates were linked together using angles and rivets. Due to their aesthetics, superior quality, and affordability, welded plate girders began to replace riveted and bolted plate girders in the developed world by the 1950s (Patidar and Harne, 2017).

The SPGs are extensively used as primary structural components in the construction of bridges and overpasses with relatively high spans, as well as large-span residential buildings and the frames of industrial structures (Bahrami and Najarnasab, 2020).

For highway bridges with spans of 24–46 m and railway bridges with spans of 15–40 m, plate girders are often more cost-effective. If the bridges are continuous, it might be extremely competitive for considerably longer spans. In buildings, plate girders are also employed to support heavy concentrated loads. Such circumstances may occur if a wide hall on the ground floor of a multi-story building is required, yet there are no interfering columns (Subramanian, 2010).

In the design of a bridge girder plate, stress, bending moment, and shear force increase while deflection decreases if the span is maintained constant while the web thickness varies in increasing order. When the span varies in increasing order while the web thickness remains constant, stress, bending moment, shear force, and deflection all increase (Patidar and Harne, 2017).

2.1.1 Advantages of plate girders

There are various benefits of employing plate girders stated, but the main ones stated by (Garg, 2017)include:

- Quick transportation
- Prompt assembly
- Greater carrying capacity
- longer fatigue life and greater stability
- The primary components dimensions can be altered to fit the precise bending moment and shear as required by the design (Ghosh Utpal ,2016)

2.2 Shear strength of Plate Girders

The web of the SPGs is made up of a large low-thickness steel sheet surfaces, and it is encased between the top and lower flanges and vertical stiffeners. SPGs' web is thought to be lighter than their flanges causing the web of SPGs buckles or yields under shear stresses before reaching their maximum flexural strength (Bahrami and Najarnasab, 2020).

The depth to thickness ratio of the web and the spacing of the provided intermediate stiffeners both affect the plate girder's shear strength. Plate girder webs are typically transversely strengthened. This aids in raising the webs' overall shear resistance. The depth of the webs causes issues in plate girders. Adding stiffeners is one approach to increase a thin web plate's ability to support loads; choosing the proper kind of stiffening is crucial when designing a plate girder. A plate girder can fail in two different ways: yielding of tension flange and buckling of compression flange. There are several ways that the compression flange can buckle, including vertical buckling into the web, flange local buckling, and lateral torsional buckling (Subramanian, 2010).

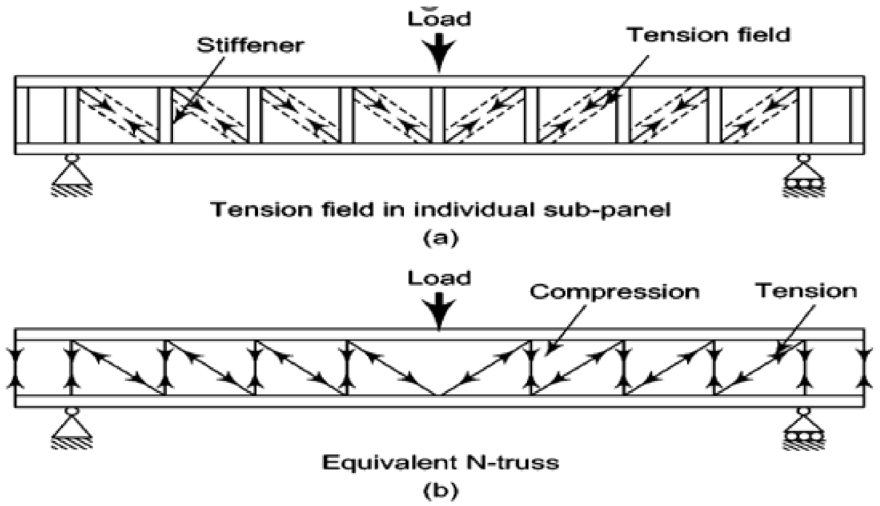


Figure 2-2 Tension field action and Equivalent N-truss

The tension field action in a panel is shown in Figure 2.2 (a). The web loses its ability to withstand the diagonal compression as it starts to buckle. The transverse stiffeners and the flanges receive the diagonal compression after that. The stiffeners sustain the vertical component of the diagonal compression, whereas the flanges obstruct the horizontal component. Only the diagonal stress is resisted by the web, and this behavior of the web is known as tension field action. As seen in Figure 2.2-b, the behavior is quite similar to that of a Pratt truss, in which the diagonals carry the tension and the vertical elements carry compression. Only as the web begins to buckle do we recognize the contribution of the tension field action. The web's overall strength is made up of the pre-buckling strength created as a result of tension field action. Thus, the use of appropriately spaced stiffeners, referred to as intermediate stiffeners, can be employed to improve the web's shear capacity and create tension field action. Instead of resisting the applied loads, the primary function of these stiffeners is to give the web rigidity (Subramanian, 2010).

To shield the web from direct compressive loads, additional stiffeners known as bearing stiffeners are placed at concentrated load sites. They can also serve as intermediate stiffeners at the same time. Concentrated loads applied to the top flange can cause side-sway web buckling, web crippling, and web yielding. This type of buckling happens when the tension flange buckles laterally due to the compression in the web. The relative movements of the flanges should be constrained by lateral bracing to avoid this. When designing welds, general welding techniques are used to connect the components. (Subramanian, 2010).

There are two parts to the web's shear capacity, namely, strength prior to the onset of buckling and strength following the buckling. When a stiffened web panel is subjected to an increase in shear load, the web panel buckles (**see Figure 2.3).** This load does not represent the web's full shear capability. Depending on the tension field action, the load can still be increased and the web panel will continue to support more weight. A portion of the buckled web bears the stress of the load. This tension member action is across the web panel in an angled direction to the web panel diagonal shown in figure 2.4 (Subramanian, 2010).

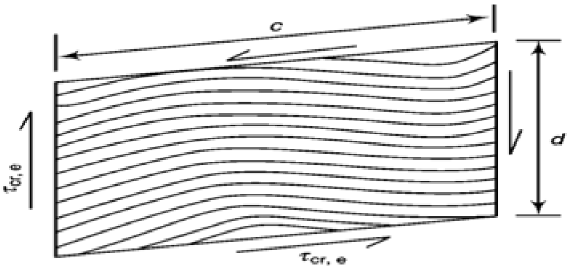


Figure 2-3 Buckling of the web panel

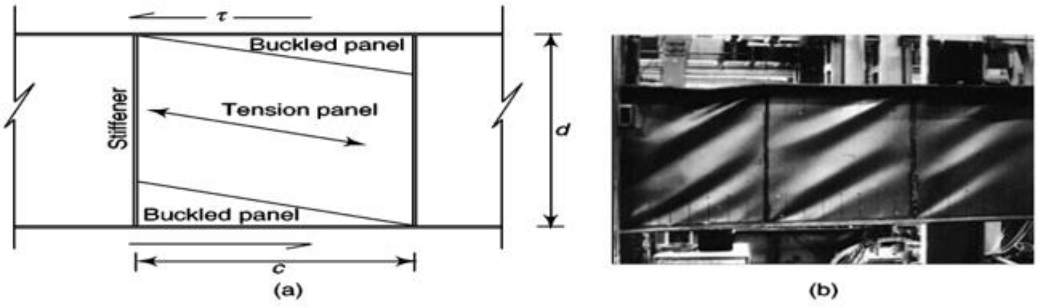


Figure 2-4 (a) panel showing tension field and (b) tension field in a plate girder

At this stage, the girder behaves like an N-truss (Figure. 2.2), with the buckled web resisting the tension and the flanges carrying the compression forces. Tension field action is the name given to this additional reserve strength. It should be noted that the shear capacity is limited to the strength before buckling if there are no intermediate stiffeners or if their spacing is too wide. In these cases, tension field action cannot occur. The stiffener spacing ‘a’ affects the shear-induced buckling and post-buckling behavior of the web. The next section describes the various shear resistance stages. Each web panel on a long span plate girder may have a unique combination of bending moments and shear forces. Panels close to the support will be primarily subject to shear, whereas those close to the center will be primarily subject to bending forces (Subramanian, 2010).

The nominal shear strength of webs with or without intermediate stiffeners under buckling can be calculated using the Simple post-critical method or Tensile field theory. The simple post-critical method is a general method that can be used to design all types of girders. The simple post critical method can be applied to I-section girder webs with or without intermediate transverse stiffeners, as long as the web has transverse stiffeners at the supports. Tension field action is limited to a specific range of girders and produces efficient girder designs by utilizing the post-buckling reserve of resistance. According to tension field action theory, three factors contribute to the predicted shear strength: the first is the primary buckling strength of the plate, the second is due to tension field action of the web, and the third is due to the plastic moment capacity of the flanges (Subramanian, 2010).

Two flanges, a web, and many transverse stiffeners are welded together to create modern plate girders. Web plates preserve the space between the flanges and resist shear, while flanges resist applied moment. Compared to the usual flange forces, the induced shearing force is practically lower. Therefore, deep girders are frequently used to achieve a high strength to weight ratio. This calls for a thick, deep web whose weight is decreased by doing so. In the design process, a number of instabilities are taken into account, including shear buckling of web plates, lateral torsional buckling of girders, compression buckling of webs, flange-induced buckling of webs, and local buckling and crippling of webs. The thinness of web plates causes them to buckle during the initial phases of loading. Shear buckling and web part failure are thus critical design considerations for plate girders. To boost their buckling strength, webs are frequently strengthened with transverse and, in certain circumstances, longitudinal stiffeners (Dabaon M.A. and EL Hadidiy A.M., 2014)

2.2.1 Stiffeners

The web in a plate girder is frequently made very thin to maximize weight economy. When the webs are unable to support the load, a variety of stiffeners are used to strengthen and stabilize them. Stiffeners are secondary plates or sections that are attached to beam webs or flanges to stiffen them against out-of-plane deformations.

The stiffeners are classified based on their role in strengthening the web and are listed below (Garg, 2017).

- A load carrying stiffener prevents the web from buckling locally due to any concentrated load. Bearing stiffeners, as a result, transfer heavy reactions or concentrated loads to the entire depth of the web. They are installed in pairs on the web of plate girders at unframed girder ends and where concentrated loads are required. They should fit tightly against the loaded flanges and extend as far as possible out to the edges of the flange plates.

- An intermediate transverse web stiffener primarily improves the shear buckling strength of the web. They continue to be effective after the web buckles by providing tension field anchorage and, finally, by preventing the flanges from moving towards one another. Non-bearing stiffeners and stability stiffeners are other names for them.
- To improve economy, intermediate stiffeners are sometimes alternated on each side of the web, or they are all placed on one side to improve aesthetics (Subramanian, 2010)
- Torsional stiffeners are installed at supports to prevent girders from twisting.
- The diagonal stiffener provides local web strengthening under shear and bending conditions.
- The tension stiffener transmits tensile forces from the flange to the web.
- A longitudinal stiffener increases the web's buckling resistance.

Table 2-1 Types of stiffeners

Types of stiffeners		
1	Intermediate Transverse and Longitudinal Web Stiffeners	Enhancement of buckling strength of buckling strength of slender web due to shear
2	Load-carrying Stiffeners	Prevention of local buckling of web due to concentrated loading
3	Bearing stiffener	Prevention of local crushing of web due to concentrated loading
4	Torsional Stiffener	Provision of Torsional restraint at supports
5	Diagonal Stiffener	Provision of local reinforcement to web under shear and bearing
6	Tension Stiffener	Transmission of tensile forces to web through flange

2.2.2 Resistance stages of web panels

A plate girder's failure mode will be heavily influenced by the panel aspect ratio ($\frac{c}{d}$) and the web slenderness ratio ($\frac{d}{t_w}$) when the applied load is raised, where c is the clear distance between vertical stiffeners, and d and t_w are the clear depth and the thickness of the web panel respectively. The web will fail by yielding in shear when the panel is stocky, and this is determined by the theoretical shear yield strength. However, for the majority of practical plate girders, web panels are often thin and tend to buckle before yielding. The unbuckled stage, the post-buckled stage, and the collapsed stage are the three distinct phases of a web panel's general behavior (Vimonsatit, Tan and Ting, 2009).

2.2.2.1 Unbuckled stage

Prior to buckling, equal tensile and compressive principal stresses are developed in the plate. A principal tensile stress of magnitude τ will act throughout the entire web if a uniform shear load is applied to it. This stress condition will persist until the applied shear stress reaches the critical shear strength τc_r, which may be calculated using Timoshenko and Gere's traditional stability theory for plates (Timoshenko and Gere James M., 1985).

Complementary shear stresses are created when a square web plate is exposed to vertical shear in order to maintain the plate's equilibrium. The plate consequently creates diagonal compression and tension. Consider a small element E that is in equilibrium and experiencing shear stress τ inside the web plate (see Figure 2.5). The element experiences principal tension in direction BD and principal compression in direction AC. The plate will eventually buckle in the direction of the compressive diagonal AC as the applied stress is gradually increased. The plate is unable to withstand a further rise in compressive stress. The corresponding shear stress τ in the plate is known as the panel's 'elastic critical shear stress' τ_{cr}. If the plate's boundary conditions are known, the value of τ_{cr} can be simply calculated using classical stability theory. However, knowing the true boundary conditions of the plate provided by the flanges and stiffeners is quite challenging. As a result, conservatively, simply supported conditions are assumed.

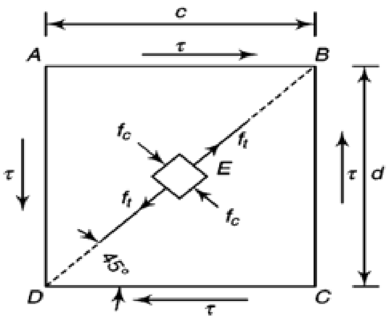


Figure 2-5 Unbuckled shear panel ($\tau < \tau_{cr}$) (Subramanian, 2010)

It should be noted that if no intermediate stiffeners are present or their spacing is large, tension field action cannot occur and shear capacity is limited to the strength before buckling (Subramanian, 2010).

2.2.2.2 ***Post-Buckled Stage***

When the critical shear strength is reached, the web can no longer withstand any increase in shear load. The mobilization of tensile membrane stress σt in the web's diagonal band will provide additional shear force (Vimonsatit, Tan and Ting, 2009).

In Figure 2.6, the magnitude of the tensile membrane stress is denoted by σt, and its yielding inclination to the horizontal is denoted by ɵ. Because the girder's flanges are flexible, they will bend inwards under the pull of the tension field (Narayanan et al., 1981)

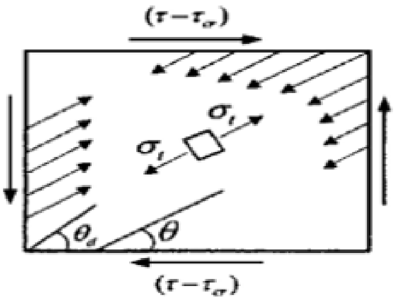


Figure 2-6 Post buckled stage (Narayanan et al., 1981)

The flange rigidity appears to have little effect on the post buckling strength of web panels; however, it affects the elastic shear buckling strength of web panels as it affects the degree of constraint at the flange-web juncture (Lee and Yoo, 1998).

Plate girders rely on the webs' post-buckling strength. The major planes would be inclined to the longitudinal axis of the member at high shear places in the girder web, often around the supports and the neutral axis. The major stresses along the principal planes would be diagonal tension and diagonal compression. Though diagonal tension is not a concern, diagonal compression causes the web to buckle perpendicular to its action (Subramanian, 2010).

2.2.2.3 ***Collapsed Stage***

When the top and bottom flanges have enough hinges to cause the plate girder to fail, the diagonal yield zone and web panel work together to create a plastic sway mechanism as shown in figure 2.7. The additional shear load that the web panel must endure before collapsing is calculated by taking into account the virtual work that was done to the sway mechanism (Vimonsatit, Tan and Ting, 2007).

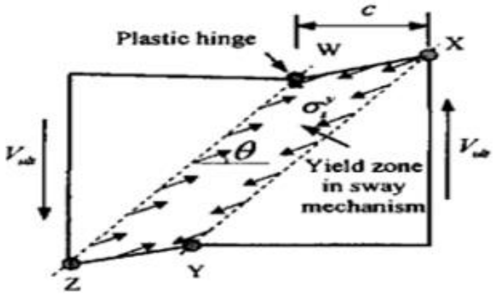


Figure 2-7 Collapsed stage (Vimonsatit, Tan and Ting, 2007)

2.3 Openings in a Plate Girder's Web

Openings in the webs of plate girders are required to facilitate inspection and service. The addition of openings in a web causes stress distribution within the member and reduces its strength and stiffness (Al-Mazini, 2016).

For ducts, cables, and other services, as well as to simply lighten the load, openings in steel plate girders may be necessary (Chung, Liu and Ko, 2001).

Girders with web openings are more sensitive to changes in web slenderness and flange stiffness ratios than solid web girders (Al-Mazini, 2016).

The high strength properties of structural steel cannot always be used to best advantage due to limitations on maximum allowable deflection. As a result, several new methods for increasing the stiffness of steel members without increasing the weight of the steel required have been developed. One of these solutions was a beam with a web opening. The shape of the web opening will be determined by the designer's preference as well as the purpose of the openings (Jichkar, Arukia and Pachpor, 2014).

As chung et al. stated the presence of web openings in steel beams introduces three distinct failure modes at perforated sections

- Shear failure as a result of reduced shear capacity,
- Flexural failure as a result of insufficient moment capacity,
- The 'Vierendeel' mechanism, as depicted in Figure 2.8, is caused by the formation of four plastic hinges in the tee-sections above and below the web openings as a result of the Vierendeel action, i.e., the transfer of lateral shear force across a web opening (Chung, Liu and Ko, 2001).

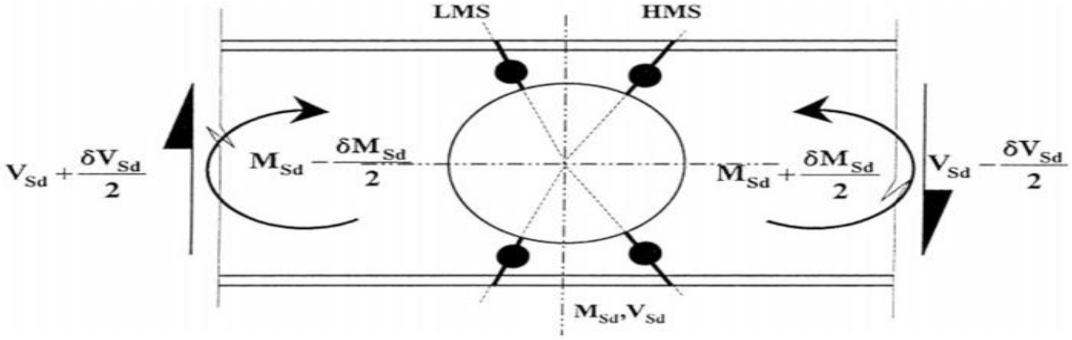


Figure 2-8 Vierendeel mechanism around the circular web opening

The main advantage in the construction is acknowledged to be the simplicity with which utilities, such as hydraulic and ventilation pipes, as well as electrical cables, can be integrated inside the structural depth of the beams. In contrast to the traditional plain webbed beam usage, where services are supported underneath the beam, the impact is a reduction in the overall building height due to the drop in the floor to ceiling height for each storey level. This results in a more efficient and cost-effective construction technique. Additionally, based on the advancement of long span composite systems, there have been notable advancements in the structural design of commercial multi-story buildings in recent years. By reducing the number of columns, long span beams provide the benefit of internal planning flexibility, which reduces the need for foundations and speeds up and lowers the cost of construction. Long span beams are more competitive in the market, especially when produced for parking garages. Even though there are numerous alternatives to solid web beams, such as stub girders, trusses, etc., beams with web openings can still be competitive. With the placement of services within the girder depth at the most suitable locations, this type of construction maintains a reduced depth (Morkhade and Gupta, 2015).

The SPGs take up a lot of space inside the structures because of their deep design, which is in opposition to the aesthetics of the structures' architecture. Researchers have suggested employing SPGs with openings in their web as a strategy to make the best use of this area. But these openings have exacerbated structural issues like decreased strength, localized buckling, and instability (Bahrami and Najarnasab, 2020).

In highway bridges, web holes on girders are utilized to allow space for service, inspection, and maintenance. These openings can have lengths that are two or three times the opening depths and occasionally have depths that are as large as 60% of the beam depth. Openings that are round and rectangular are widely applied. In order to reduce or completely remove stress concentration, rectangular openings typically have curved edges (Shanmugam et al., 2002).

Buckling of web posts may be critical in beams with multiple web openings when the openings are closely spaced. Furthermore, additional deflection due to the presence of web openings should be taken into account (Chung, Liu and Ko, 2001).

Chung et al. discovered that the reduction in shear capacity of the perforated section is more important than the reduction in moment capacity for steel beams with circular web openings. The precision of the shear capacity assessment of the perforated sections is crucial for the accurate prediction of the load carrying capacities of steel beams with circular web holes. It is shown that the structural penalty on the load carrying capacity of steel beams brought on by the presence of web openings may be fully minimized via the sensible design of the web openings in terms of the positions along beam span as well as the opening sizes (Chung, Liu and Ko, 2001).

Gabar investigated three steel plate girders' structural behavior under shear. The reference plate girder (G) is the first one; it is constructed without web openings. The second plate girder (GO) is made with a circular opening at the center of each web panel; its diameter is 60 percent of the web depth. The third plate girder (GOR) has reinforced strip welded around the circular web openings. The panel's aspect ratio is 1, and they are all the same size. Using the package software program (ANSYS V.11), a nonlinear finite element analysis is performed for the tested plate girders. Additionally, parametric study is carried out with variable reinforcement size around the web openings. The generalization that central openings reduce the shear strength of steel plate girders is supported by experimental work. As a result, with reduced shear load, the web panel of the plate girder

(GO) exhibits significant deformation. Compared to identical panels with circular cutouts without reinforced strips, the strength is increased by 32% by the reinforcement surrounding the circular hole. The parametric investigation reveals that increasing the thickness of the reinforcing strip around the opening has a bigger impact on shear strength than increasing its breadth (Gabar and Hamoodi, 2013).

Bahrami et al. analyzed SPGs with elliptical web holes. The SPGs were examined using ABAQUS, a finite element program. The analyses of the SPGs considered a number of variables. The factors included the number of openings, how they were arranged, how much of the steel frame was used, how much of the diagonal steel stiffeners were used, and how thick the steel frame and diagonal steel stiffeners were. These variables' effects on the effectiveness of the SPGs were assessed. It was discovered that increasing the number of openings had a negative impact on ultimate strength. The placement of the SPGs' openings has a significant impact on how well they work. It is preferable to have horizontal openings rather than vertical ones. The final strength is increased by using the steel frame around the openings. The optimum strength is also obtained by using the steel stiffeners. Additionally, the ultimate strength is increased by thickening the steel frame and/or diagonal steel stiffeners. However, increasing the frame thickness has a greater overall impact on raising ultimate strength than increasing the thickness of the stiffeners. The SPGs' failure modes were assessed (Bahrami & Najarnasab, 2020).

Shanmugan et al. presented six girders with web slenderness ratios of the web plate d/t ranging from 50 to 300, to determine the effect of web slenderness on the ultimate load carrying capability of plate girders with center circular holes. To investigate the impact of flange stiffness on the girders, five additional models were examined. The finite element software program ABAQUS is used to analyze 20 plate girders with centrally located circular openings and 12 plate girders with centrally located rectangular holes. At the center of the top flange, a vertical concentrated load was applied. The relationship between the ultimate load of plate girders with solid web and central circular openings is constant regardless of changing flange stiffness ratio, $\frac{b_f}{t_f}$, because no appreciable change in the non-dimensionalized ultimate load is seen in the other case. The ultimate load capacity of girders with larger $\frac{b_f}{t_f}$ ratios is relatively constant; this may be because the contribution of the flanges to resisting the load relative to the web is minimal in this situation (N. E. Shanmugam, Lian and Thevendran, 2002).

Rodrigues et al. indicated three separate failure modes, including shear, bending, and Vierendeel collapses, are introduced by the existence of web opening in steel beams. Due to the development of four plastic hinges on the "Tees" created by the lateral shear stress transferred via the web opening, the Vierendeel collapse happens. In general, it is simple to assess the perforated sections' shear and bending moment capabilities. Due to the concurrent existence of axial, shear pressures brought on by the global bending stresses operating at the perforated section, it is challenging to measure the "Tees" bending resistance. Additionally, four plastic hinge forms are directly related to the proper assessment of the collapse mechanism, which necessitates the application of plastic design principles. The shear and bending capacities of the beam are typically reduced as the height of the web opening increases. Alternately, increasing the web opening length decreases the perforated section Vierendeel collapse capacity by increasing the local Vierendeel bending force acting on the "Tees" but has little effect on the beam shear and bending capacities. The effectiveness of longitudinal stiffeners welded at the opening region was also examined, as were the advantages of utilizing an appropriate edge concordance radius in beams with rectangular and square openings (Rodrigues *et al.*, 2014).

Al-mazini et al. tested the structural response of seven plate girders under two concentrated static loadings risen just enough to cause collapse. One of the girders had a flat web, therefore, it served as a control girder. The six girders were separated into three groups: plate girders with circular openings, square openings, and rectangular openings in the center of the web. The plastic hinge develops close to the line where the web and flanges meet, and the buckling load roughly runs transverse to the web. For webs with openings, particularly those with rectangular opening webs, shear stresses decreased. For plate girders with rectangular holes, lower critical shear stress values can be obtained, especially in circumstances where the slenderness ratio is high. Additionally, as the hole size increases, the shear stress reduces. Shear stresses diminish as the web's width-to-height ratio rises, especially for thin webs with high slenderness values (Al-Mazini, Alhamaidah and Zewair, 2021).

Azmi et al. illustrated the ultimate load behavior of perforated steel plate girders with inclined stiffeners. A single concentrated load applied at the center of the girder span was used to test ten simply supported plate girders until they failed. This study's primary objective was to examine the impact of various inclination degrees on the post-buckling behavior of intermediate stiffeners. The test series took into account the intermediate stiffeners' inclination angles of 90°, 75°, 60°, 45°, and 30° as measured from the bottom flange. Due to the usage of inclined stiffeners, the diagonal length of the compression and tension flanges are now uneven, complicating the behavior of web panels. In order to assess the effectiveness of such girders in terms of load carrying capacity, load-deflection behavior, and failure characteristics, an experimental series on perforated plate girders with inclined stiffeners must be conducted. The study's main discovery is that by reducing the angles of the intermediate inclination stiffeners, the final capacity of the test girders improved by 92 percent. The loss in load carrying capacity caused by the presence of web openings in plate girders was therefore restored. It was clear that intermediate inclined stiffeners significantly contributed to boosting the load carrying capacity by raising some internal forces placed on the web panels. By applying some internal stresses to the web panels, it was clear that intermediate inclined stiffeners significantly increased the load carrying capacity. As a result, the loss in load carrying capacity caused by the presence of plate girder web apertures was recovered. The tension bandwidth increased in comparison to standard girders with rectangular web panels because trapezoidal-shaped web panels were made at the end of the web panels by the addition of intermediate inclined stiffeners. As a result, greater tension forces can be applied during the post-buckling period (Azmi *et al.*, 2017).

In the study's scenarios, the presence of a large square web opening with a depth equivalent to 0.67 of the web depth reduced strength by nearly 80%. Additionally, the stiffening of the web opening resulted in an 83 percent increase in strength (Dabaon M.A. and EL Hadidiy A.M., 2014).

It is shown that beams with square and rectangular opening heights of 75% of total girder depth have severely degraded structural capacities. The beams with rectangular holes had the lowest ultimate loads. That is, they cost 30% less than identical beams with square or round openings. Two unique collapse modes were observed in beams with circular holes.

The beams with openings 0.15 L from the supports collapsed due to a load application point bending failure (Rodrigues *et al.*, 2014).

The existence of a web opening results in a loss of strength due to a lack of resistance in the web panel to out of plane deformation. The reduction in strength caused by the presence of a circular cutout with a diameter of 60% of the web depth is 51%. The shear strength of this girder is recovered in the web panel of a plate girder with reinforced strip welded around the circular web apertures. When compared to similar panels with circular cutouts that do not have reinforced strips, the reinforcement surrounding the circular opening enhances the strength by 32% (Gabar A. and Hamoodi J., 2013).

According to the prior literatures, studies on the ultimate shear capacity of perforated steel plate girders subjected to shear loading using the parameters considered in this study, namely span length, opening shape, opening size, number of openings, and location of opening, are rare. This study, however, is one of the few available.

CHAPTER 3 METHODOLOGY

3.1 Introduction

This chapter describes the methodologies used in conducting a parametric study on the shear capacity of plate girders of various shapes, sizes, and locations of web openings using numerical method.

It is well understood that experimental tests provide the most reliable results and are used to reflect nearly actual failure modes and loading resistance. When extensive parametric studies are required to cover longer spans, different opening locations, opening types and sizes, requiring time and a large working area, performing experimental tests would be extremely difficult. This limitation encourages the use of advanced computer simulations, particularly finite element modeling.

The finite element method is a numerical analysis technique that is used to obtain approximate solutions to a wide range of engineering problems. Although it was initially designed to study stresses in complex airframe structures, it has since been extended and applied to the broader field of continuum mechanics. It is gaining popularity in engineering schools and industry due to its versatility and adaptability as an analysis tool (Ratheesh and Chacko, 2019).

The numerical simulations developed in this research program are carried-out by using general-purpose finite element software ABAQUS/CAE software package (SIMULIA 2022).

3.2 Preliminary proportioning of plate girder

Proportioning the section entails determining the flange and web sizes and determining whether or not these sizes provide the required strength.

A plate girder's general dimensions are determined by its optimal behavior. The design of a plate girder begins with the values of live load shears, bending moments, and estimated dead loads. Following the selection of the girder depth, the web can be designed to resist shear while keeping the minimum thickness requirements in mind. The flanges provide the majority of the moment of resistance in an I-section plate girder, with the web contributing very little. A trial section must be assumed initially in the design of a plate

girder, and the amount of load to be carried includes the girder's self-weight as well. The simplified flange area method can be used to obtain a preliminary estimate of the girder weight (Subramanian, 2010) .
The following are the functional requirements of a plate girder:

- Strength to withstand bending moment
- Vertical stiffness to meet any deflection limitation (adequate moment of inertia)
- Lateral stiffness to prevent lateral-torsional buckling of the compression flange
- Strength to carry shear (adequate web area)
- Stiffness to improve the web's buckling or post-buckling strength

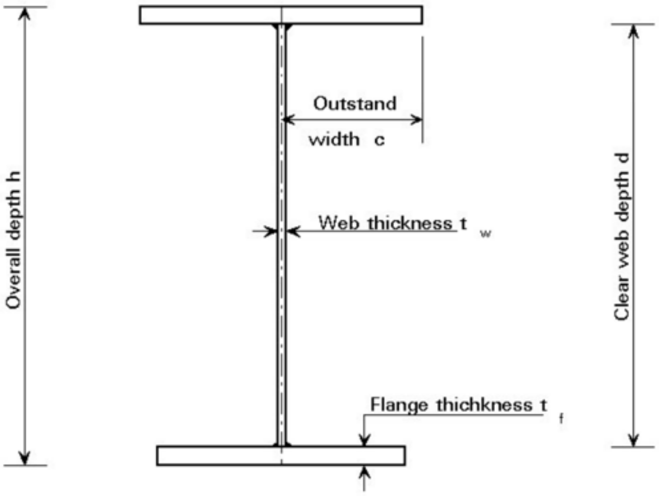


Figure 3-1 Plate girder proportions

Although more slender cross-sections are permitted, the slenderness of plate girders used in buildings should not exceed the Class 3 cross-section restrictions (given in Clause 5.5 of EN 1993-1-1). Plate thickness selection and cross-sectional buckling are linked. If the plates are too thin, stiffening may be required to restore proper stiffness and strength(European Committee for Standardization, 2005) .
Preliminary proportioning of plate girders in this study is done based on AASHTO and Eurocode 3 so that the general conclusions serve for bridge and buildings.

3.2.1 Span to Depth ratios

Plate girders with constant or variable depth can be produced at a low cost using modern fabrication techniques. Constant-depth girders have historically been more common in buildings, but as designers demonstrate a greater willingness to modify the steel structure to accommodate services, this may change.

For constant depth beams used in simply supported non-composite girders, the recommended span to depth ratio is between 12 and 20. For all spans in this study, d=L/20 is used.

3.2.2 Web depth and Thickness

The selection of a near practical optimum depth is the most important parameter for the design of a steel girder bridge. As a first step, the minimum steel plate girder depth for tangent girders can be determined using the span-to-depth ratios in AASHTO Table 2.5.2.3.3-1. It may appear reasonable to choose the deeper web depths based on engineering mechanics principles. However, the structure depth will be limited in most projects due to minimum vertical clearance or hydraulic requirements.

The girder's web primarily gives it shear strength. Because the web makes only a small contribution to the bending resistance, it should have the smallest thickness possible to adhere to the web depth to thickness ratio limits of $\frac{d}{tw} \leq 150$ for webs without longitudinal stiffeners and $\frac{d}{tw} \leq 300$ for webs with longitudinal stiffeners, respectively (Article 6.10.2.1) (American Association of State Highway and Transportation Officials, 2005).

In general, the practical optimum depth will be 5% to 15% greater than the minimum depth required. For example, the minimum steel plate girder depth for a 15m span length (L) simple span steel plate girder bridge is =0.033*L=0.033*15000=495 mm.

The web thickness shall not be less than 7mm, according to AASHTO 6.7.3.

The web depth (d) must be as large as possible to produce the lowest axial flange force for a given bending moment. To reduce self-weight, the web thickness (t_w) must be kept as low as possible. As a result, the web plate is often of slender proportions and prone to buckling at relatively low values of applied shear.

According to Eurocode 3 (EN 1993-1-5:2006), the web's depth to thickness ratio (d/t_w) must be less than $\frac{72\varepsilon}{\eta}$ in order to prevent web buckling; if not, the web must be tested for shear buckling resistance and given transverse stiffeners at the supports (European

Committee for Standardization, 2005). Depth to thickness ratio (d/t_w) 62.5 is used for all spans in this study.

Where $\varepsilon = \sqrt{\frac{235}{fy(\frac{N}{mm2})}}$, η- Factor depending on steel grade, 1.2 for steel grade up to and including S460, and for higher steel grades η=1

In general, a web is dimensioned using a depth-to-width ratio that takes buckling into account. In terms of flange cross section, the web is typically over dimensioned; that is, in some situations, the material strength of the web cannot be fully exploited, and so some material of the web can be conserved, for example, via web holes (Wu, Li and Zhou, 2018).

3.2.3 Flange width and thickness

The flange force and material yield stress define the required flange area. The flange thickness, t_f, will usually meet the Eurocode 3 (Table 5.3.1) requirements for Class 3 (semi-compact) sections, i.e., $\frac{c}{tf} \leq 14\varepsilon$, where c is outstanding width of flange. According to Eurocode 3, 5.5.2 classification Class 3 cross-sections are those in which the stress in the steel member's extreme compression fiber, assuming an elastic distribution of stresses, can reach the yield strength, but local buckling is likely to prevent the development of plastic moment resistance.

AASHTO 6.10.2.2 requires compression and tension flanges to be proportioned so that

- the flange will not distort excessively when welded to the web $\frac{b_f}{2t_f} \leq 12$ and
- $\frac{d}{bf} \leq 6$, so that the stiffened interior web panels can develop post buckling shear resistance due to tension field action.

Where

- b_f= full width of flange (in)
- t_f= thickness of the flange (in)
- t_w= thickness of the web (in)
- d= web depth (in)

For non-composite girders, the flange width is usually between 0.3 and 0.5 times the section depth (0.4 is most common). These guidelines can still be used for preliminary compression flange sizing on simply-supported composite girders. Tension flanges can have their width increased by 30% according to BS EN 1993-1-1.

To ensure that some restraint will be provided by the flanges against web shear buckling, thickness of the flange must be greater than 1.1tw ($t_f \geq 1.1 t_w$) according to AASHTO (6.10.2.2-3).

3.3 Modelling and Validation

A model should be created with a specific purpose (or application) in mind, and its validity should be determined in relation to that purpose. If the goal of a model is to answer a variety of questions, the model's validity must be determined for each question. The level of accuracy required should be specified before beginning model development or very early in the model development process. For these tests, the model is viewed as an input-output transformation.

It was required to ensure that the model collapse was not caused by lateral torsional buckling or local buckling at the load application point or supports during the numerical examination. The lateral displacements of the top flange along the beam span were limited, and transverse stiffeners were used at the load application point and supports.

Validation is carried out during the development of a simulation model in order to provide an accurate and convincing model. For the same set of input conditions, the validation test compares the outputs of the system under examination to the outputs of the model. Data obtained while observing the system must be available to conduct this test. The output of the primary interest model should be used as the performance metric (Robert, 2011)

In this study Finite element models were developed to simulate the structural behavior of steel beams with web openings and the models were calibrated against experimentally tested SPG by Yatim et al. in their paper called "Performance of steel plate girders with inclined stiffeners". As a result, tested and simulated SPGs behaved similarly under load and provided consistent shear capacity. The following chapter will go into greater detail.

3.4 Parametric Study

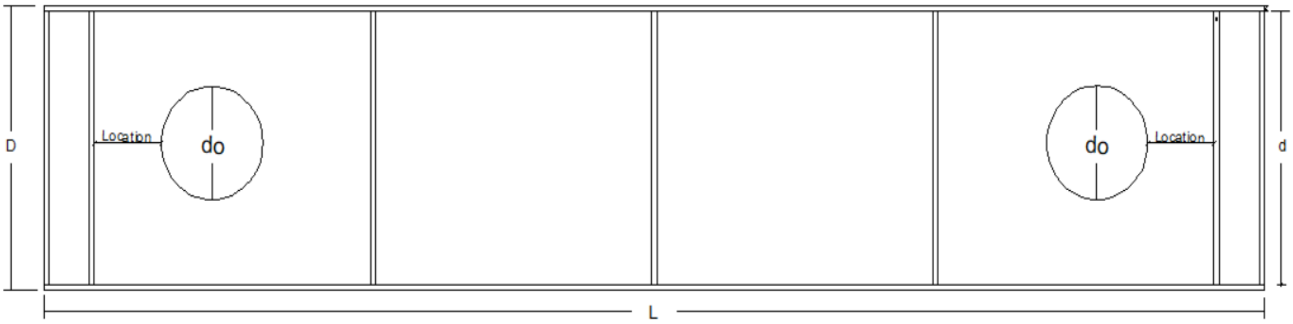
Following the validation of the finite element method's accuracy in predicting load-carrying capacity, a parametric study is conducted to evaluate the effect of various parameters on the shear capacity of steel beams with circular and square web openings having different sizes.

The study considers simply supported beams with spans of 15m, 20m, and 25m subjected to concentrated load positioned at the center of the top flange and mechanical properties as shown in Table 3.2. All of the models used the same material properties. To avoid premature failure in the analysis, load-bearing stiffeners are provided under concentrated load and at supports. The opening diameters considered in the study are 0.2d, 0.5d, and 0.7d, with a slenderness ratio of the plate girder (d/t_w) 62.5 and thickness of flange (t_f) 10mm for all the tested spans.

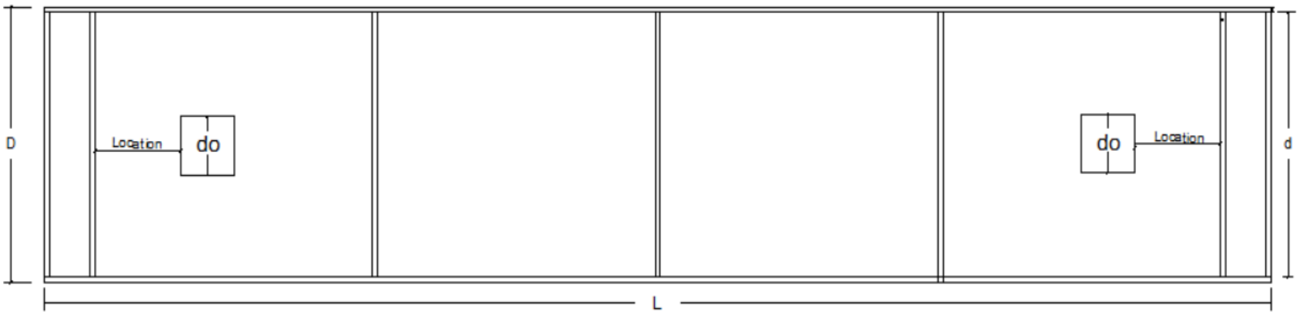
The study took into account the following parameters:

- Span length of the beam (15m, 20m, 25m)
- Shape of opening (square and circular)
- Depth of opening (0.2d, 0.5d, 0.7d):

  (The opening depth should not exceed 70% of the section depth (David, 2003))
- Location of opening (distancing 0.2d, 0.5d, d, 1.5d, 2d and 5.5d from the support)
- Number of openings (2, 4, 6, 8, and 10 openings)



(a) SPG with circular web opening



(b) SPG with square web opening

Figure 3-2 Sample geometries for the parametric study

Table 3-1 Terminology in SPG geometry and proportions

L	Span length	15m, 20m, 25m
d	Web depth	L/20
d_o	Opening depth	0.2d, 0.5d, 0.7d
b_f	Width of flange	0.4d
t_f	Thickness of flange	b_f/12
d/t_w	Depth to thickness of web ratio	62.5
t_s	Thickness of stiffener	10mm

Table 3-2 Mechanical properties of steel

Mechanical properties of steel	
Young's Modulus, E (GP_a)	200
Yield Stress, f_y (MP_a)	275
Ultimate Stress, f_u (MP_a)	360
Poison's Ratio, υ	0.3
Tangent modulus, E' (MP_a)	0.01*E

Table 3-3 SPG labels with definitions

SPG Label	Definition
WO	Without web opening
C-OP-1	Circular opening of size 0.2*depth of web
C-OP-2	Circular opening of size 0.5*depth of web
C-OP-3	Circular opening of size 0.7*depth of web
S-OP-1	Square opening of size 0.2*depth of web
S-OP-2	Square opening of size 0.5*depth of web
S-OP-3	Square opening of size 0.7*depth of web
15m-WO-OP	15m span plate girder without web opening
20m-C-OP-1	20m span with circular opening of diameter 0.2*d
25m-S-OP-2	25m span plate girder with square opening of size 0.5*d

Table 3-4 Geometric properties of the parametric Study

NO.	Span (mm)	SPG Label	Depth of Web, d (mm)	Web thickness, tw (mm)	Thickness of Flange, Tf (mm)	Width of Flange, bf (mm)	Opening depth (mm)	Opening location from support (mm)	Number of openings
1	15000	WO	750	12	25	300	0	750	2
2	15000	C-OP-1	750	12	25	300	150	750	2
3	15000	C-OP-2	750	12	25	300	375	750	2
4	15000	C-OP-3	750	12	25	300	525	750	2
5	15000	S-OP-1	750	12	25	300	150	750	2
6	15000	S-OP-2	750	12	25	300	375	750	2
7	15000	S-OP-3	750	12	25	300	525	750	2
8	20000	WO	1000	16	34	400	0	1000	2
9	20000	C-OP-1	1000	16	34	400	200	1000	2
10	20000	C-OP-2	1000	16	34	400	500	1000	2
11	20000	C-OP-3	1000	16	34	400	700	1000	2
12	20000	S-OP-1	1000	16	34	400	200	1000	2
13	20000	S-OP-2	1000	16	34	400	500	1000	2
14	20000	S-OP-3	1000	16	34	400	700	1000	2
15	25000	WO	1250	20	42	500	0	1250	2
16	25000	C-OP-1	1250	20	42	500	250	1250	2
17	25000	C-OP-2	1250	20	42	500	625	1250	2
18	25000	C-OP-3	1250	20	42	500	875	1250	2

19	25000	S-OP-1	1250	20	42	500	250	1250	2
20	25000	S-OP-2	1250	20	42	500	625	1250	2
21	25000	S-OP-3	1250	20	42	500	875	1250	2
22	15000	C-OP-2	750	12	25	300	375	150	2
23	15000	C-OP-2	750	12	25	300	375	375	2
24	15000	C-OP-2	750	12	25	300	375	1125	2
25	15000	C-OP-2	750	12	25	300	375	1500	2
26	15000	S-OP-3	750	12	25	300	525	150	2
27	15000	S-OP-3	750	12	25	300	525	375	2
28	15000	S-OP-3	750	12	25	300	525	1125	2
29	15000	S-OP-3	750	12	25	300	525	1500	2
30	15000	S-OP-3	750	12	25	300	525	4100	2
31	20000	S-OP-2	1000	16	34	400	500	1000	4
32	20000	S-OP-2	1000	16	34	400	500	1000	6
33	20000	S-OP-2	1000	16	34	400	500	1000	8
34	20000	S-OP-2	1000	16	34	400	500	1000	10
35	15000	S-OP-2	750	12	25	300	375	750	4
36	15000	S-OP-2	750	12	25	300	375	750	6
37	15000	S-OP-2	750	12	25	300	375	750	8
38	15000	S-OP-2	750	12	25	300	375	750	10
39	25000	S-OP-2	1250	20	42	500	625	1250	4
40	25000	S-OP-2	1250	20	42	500	625	1250	6
41	25000	S-OP-2	1250	20	42	500	625	1250	8
42	25000	S-OP-2	1250	20	42	500	625	1250	10

Sensitivity analysis of parameters is also done. The science of determining the amount of variation a system has in response to a specific range(s) of input is known as sensitivity analysis.

Benefits of sensitivity analysis

- Allows the decision maker to see how a change in one of a model's values affects the optimal values of the objective function while keeping all other parameters constant.
- Increases the decision maker's understanding of the sensitivity of the optimal solution to changes in various parameters of a problem.
- Allows for the rapid examination of changes as a result of improved information about a problem or a desire to understand the potential impact of proposed changes.
- In addition to identifying the most sensitive parameter, remedial measures for the weak plate girders selected from each span are recommended.

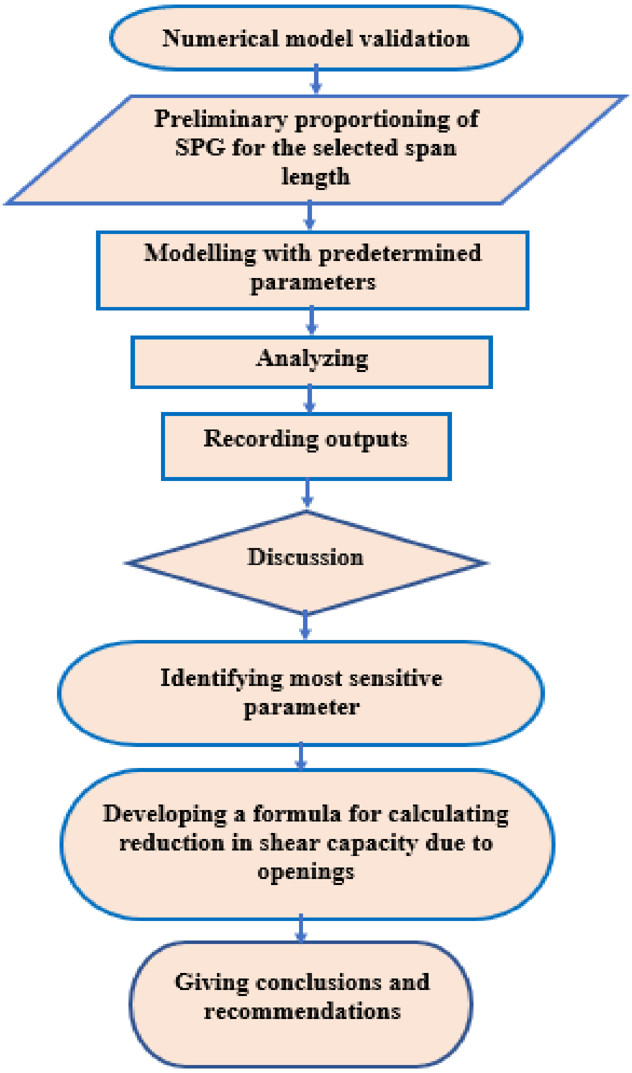


Figure 3-3 Summary for Methodology

CHAPTER 4 NUMERICAL SIMULATION, RESULT AND DISCUSSION

4.1 Numerical Model Simulation and Validation

To demonstrate the modeling accuracy, an experimental test of SPG performed by Yatim et al. titled "Performance of Steel Plate Girders with Inclined Stiffeners" was simulated and validated.

4.1.1 Experimental testing of SPG

Yatim et al. presented the ultimate performance of thin-webbed plate girders with stiffeners at 90°, 75°, 60°, 45°, and 30° inclination angles measured from the bottom flange. The plate girder webs were made slender, making them prone to buckling even at low shear but prevented from failure due to lateral torsional buckling. Variations in load carrying capacity, load-deflection response, and failure characteristics were all studied.

4.1.1.1 General Properties of Tested SPG and Test Set Up

Five plate girders were built using mild steel plates of Grade 43A, which is equivalent to Grade S275 and meets the requirements of code BS 4360: Specification for weldable structural steels. All girder specimens had the same basic configurations in order to have a constant span length, L= 2200 mm, web thickness, t_w = 2 mm, web depth, d = 500mm, flange width, b_f = 100 mm, flange thickness, t_f = 10mm, web slenderness ratio, d/t_w = 250, and aspect ratio, b/d= 1.04. The width of the web measured across the centroid of the trapezoidal-shaped panel is denoted by the notation b.

To avoid local failure of the lower flange and web, the girders were placed in the self-straining test rig over strong, simple supports at their end bearing stiffeners. Deflection was measured at various points using general purpose LVDT transducers connected to a data logger. The test setup is shown in figure 4.2.

Under a central concentrated load applied at the mid-span, the girders were tested to the point of collapse. The test results for ultimate loads, load-deflection relationships, deformation behavior, and failure characteristics are reported. Overall, shear failure dominates all of the girders (Yatim, Azmi and Mukhlisin, 2020).

For the purpose of this study, SPG with vertical transverse stiffeners (90° inclination angle) is validated.

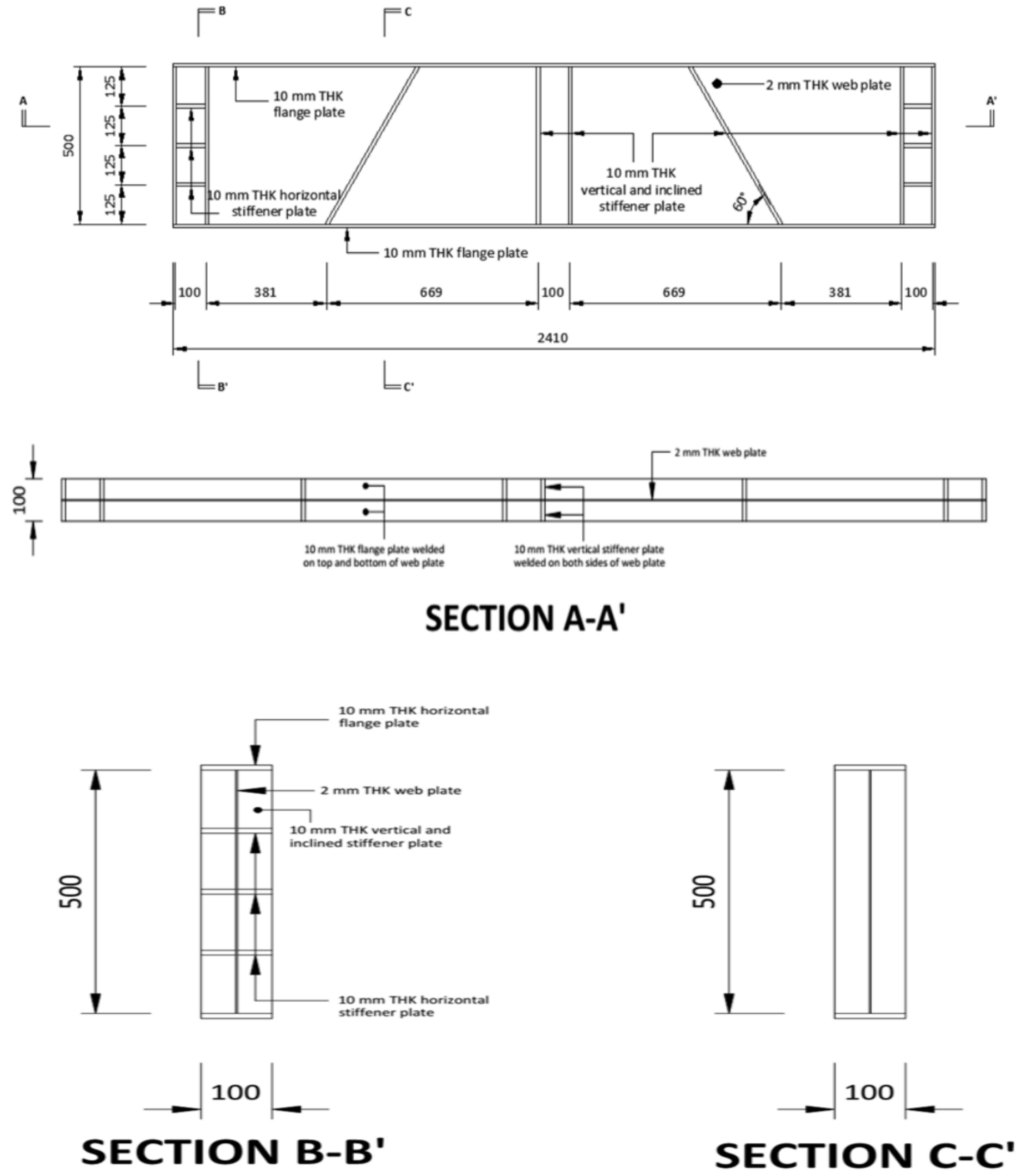


Figure 4-1 Typical details of experimental girder (Yatim, Azmi and Mukhlisin, 2020)

Figure 4-2 Typical test setup and Diagonal buckling formation in typical web panels (Yatim, Azmi and Mukhlisin, 2020)

4.1.2 Validation of Finite Element Modelling

Loading can be applied in the software using one of two methods: force or displacement control. The first method applies the force value to the structure and then the software gives its corresponding displacement value, whereas the second method applies the displacement value to the structure and the software presents the value of the force required for this displacement (Bahrami and Najarnasab, 2020).

The displacement method was used in this study at the midspan of the SPG, and the loading type is ascending in a linear pattern. By restricting the appropriate degrees of freedom, the boundary conditions of the tested specimen were accurately simulated.

The material modeling was completed as an important part of the finite element method. For the steel to adopt progressive hardening and softening effects, the bilinear kinematic hardening behavior has been considered.

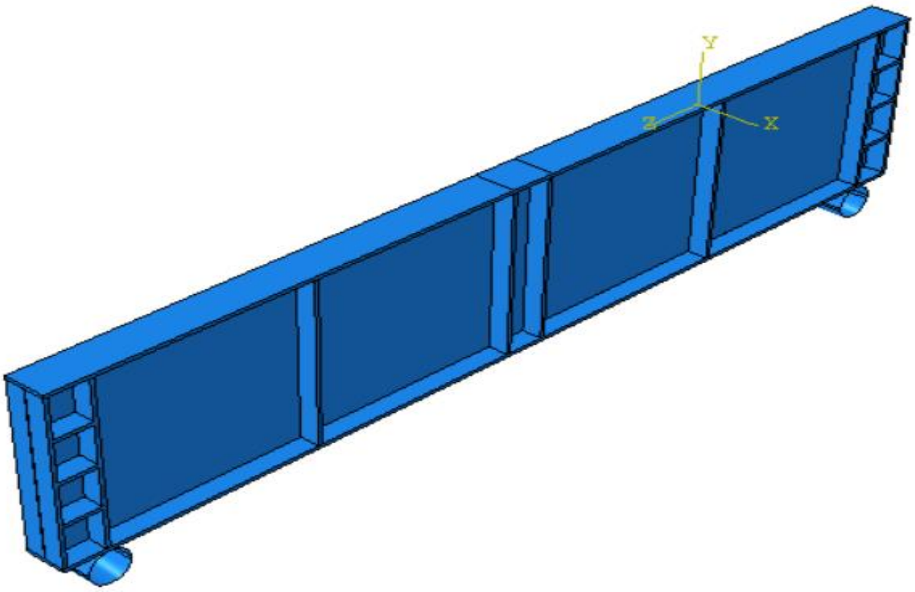


Figure 4-3 Simulated SPG for modelling verification

4.1.2.1 Type of Finite element

For modeling the flanges, web, and stiffeners, a three-dimensional eight-node solid linear brick element family with hourglass control and reduced integration (C3D8R) was used.

This first-order reduced integration element was chosen because of its low computational cost and ability to effectively capture stress concentration. Reduced integration is recommended to avoid the numerical issue associated with linear element shear locking.

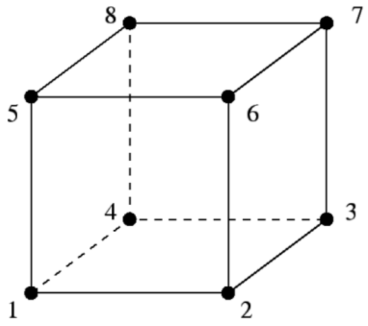


Figure 4-4 Eight node brick element (C3D8R)

R3D4, four nodded linear tetrahedral rigid elements were used to model the supports.

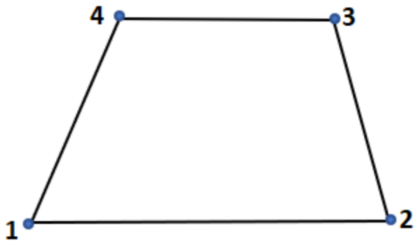


Figure 4-5 Four nodded linear tetrahedral rigid element (R3D4)

Table 4-1 Finite element types

Element types of the finite element (FE) model	
Flanges	8-Noded brick element with reduced integration (C3D8R)
Web	
Stiffeners	
Supports	4-Noded bilinear quadrilateral rigid elements (R3D4)

4.1.2.2 Mesh Sensitivity

The elements used and their mesh size are crucial in a numerical simulation. How the structure will react to various loading conditions is determined by the structural and material properties that are programmed into the mesh. A finer meshing produces a more accurate solution in finite element modeling. However, as the mesh is finer, the computation time increases. The element chosen is determined by the available computing power as well as many other parameters such as the geometry, support, and loading condition of the problem. It must be carefully tuned because complicated elements with a greater number of nodes will undoubtedly produce an accurate result, but will also make the computation tedious, time-consuming, and inefficient.

After examining various mesh sizes for the simulated SPG, the appropriate finite element mesh size achieving a more accurate result was chosen for the analysis as shown in the figure 4.6.

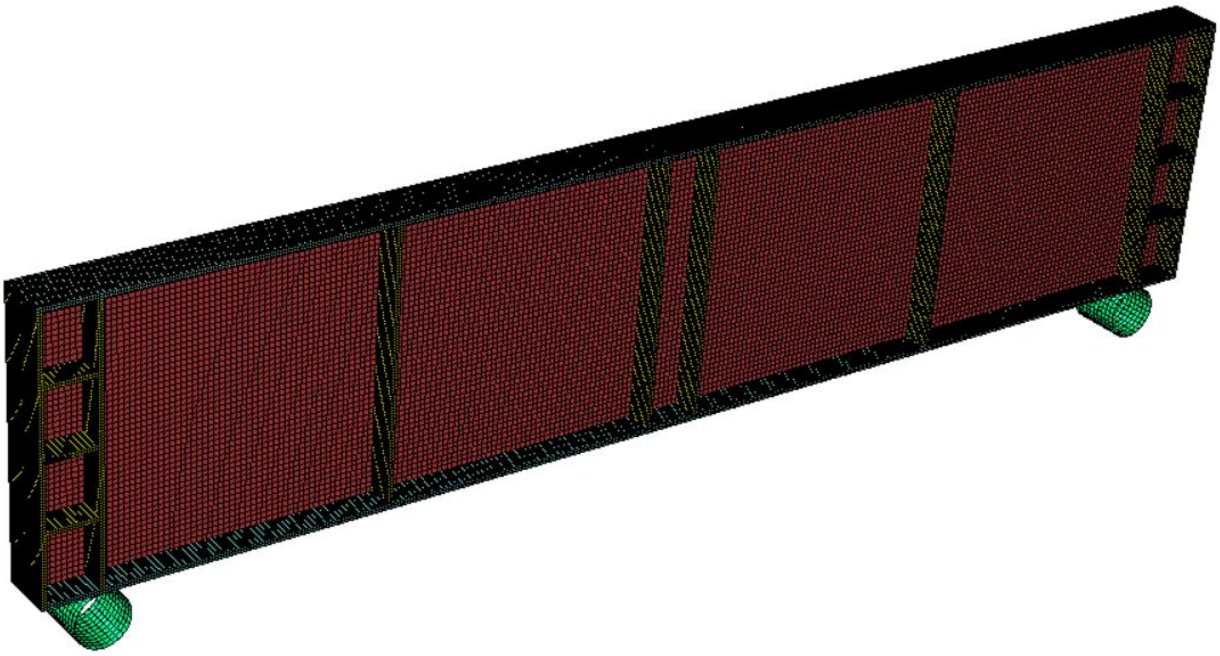


Figure 4-6 Meshed finite element model

4.1.2.3 Interaction between components

Tie constraint was used to connect the flanges to the web, the stiffeners to the web and flanges and the supports to the bottom flange. In Abaqus, a tie constraint is a type of boundary condition that connects two or more surfaces and prevents relative motion between them. It is frequently used to simulate the behavior of bolted or welded structural connections. The *Tie keyword in the input file can be used to define the tie constraint, which can be applied between surfaces of different elements or within a single element.

4.1.2.4 Boundary Conditions and Load Application

In order to simulate the supports, circular hollow rigid element was used restraining the appropriate degrees of freedoms.

The static, monotonic load was applied at the center of the top flange using Abaqus's Static general analysis, which solves for the equilibrium state of a structure under applied loads without taking into account any time-dependent effects. And ramp amplitude was used, which is a type of loading in which the applied load is gradually increased over time until it reaches a predetermined maximum value.

4.1.2.5 *Material Modelling*

The steel material has been modeled as von-Mises criteria with kinematic hardening which is suitable for most metals, including steel. Kinematic hardening is a type of material hardening that occurs when a material undergoes plastic deformation without changing shape. The same material properties that were used in the experimental work were used in this work.

A bi-linear representation of the stress-strain curve was used. The initial elastic modulus, E, was taken as 200kN/mm2 with a reduction to 2kN/mm2 (E/100) on reaching the yield point as shown in figure 4.7. This material curve is recommended in EN 1993-1-5 appendix C.6, which provides guidance on the use of F.E methods for plated structures.

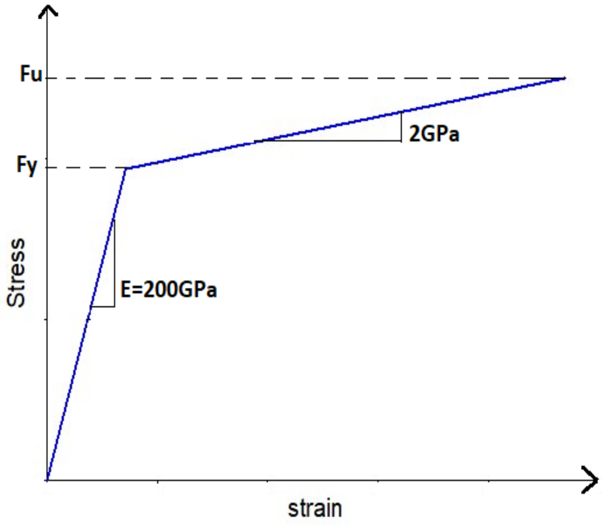


Figure 4-7 Steel adopted model: bilinear stress-strain curve

4.1.2.6 *Validation Results*

In most cases, the comparison of the finite element results and the experimental failure load shows reasonable agreement validating the accuracy of the proposed numerical models. The load-mid span deflection diagram has been depicted from the obtained modeling result with that of experiment, and modeling validation has been assessed, as shown in Figure-4.8. The FE analyses showed similar behavior in both the elastic and plastic regions. The graph shows that the maximum force carried by the simulated SPG is 204.78 kN, while the tested SPG carries 219 kN, representing a 6.49% difference. In some cases, the finite element method appears to underestimate load carrying capacity, while in others, it appears to overestimate.

A steady rate of concentrated load was applied to the top flange of the girder. During the earliest stages of stress, the girder underwent elastic bending with no appreciable vertical displacement, particularly at the mid-span as shown in figure 4.8. The panels began to buckle along the diagonal after reaching the critical buckling load, suggesting the establishment of a web tension field, as seen in figures 4.2 and 4.9.

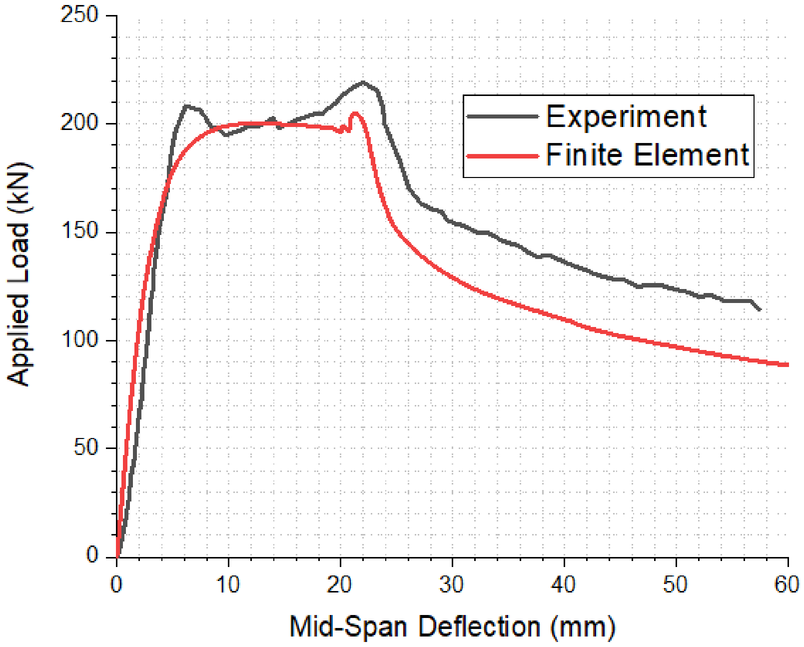


Figure 4-8 Load vs mid-span deflection comparison

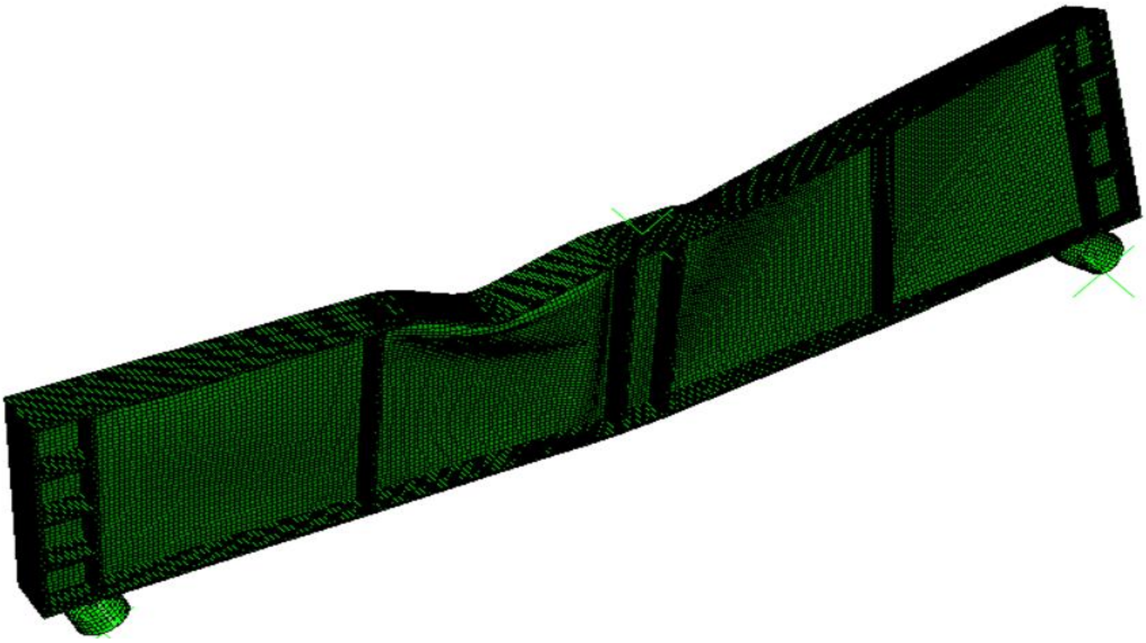


Figure 4-9 Deformed shape of validated SPG

Further compressive stress application in the post-buckling phase was resisted by tensile membrane action, resulting in increased out-of-plane deformation of the web. At this point, applying any increment of stress resulted in a substantial increase in the corresponding vertical deflection when compared to the elastic phase, as illustrated in figure 4.8.

Such behavior is caused by a gradual reduction of flexural stiffness in the girder. After reaching the ultimate point, the applied force began to gradually decrease, showing web plate yielding and thus loss of shear resistance in the girder. Beyond the maximum load, girders swayed to one side in most cases, as can be seen from figure 4.9 and 4.2, and web deformation was visible due to plastic hinges in the top and bottom flanges. The vertical component of the pulling force induced by the tension field mechanism caused these hinges. The von misses stress distribution for the validated SPG, as shown in fig 4.10, lends support to the previously described concept. Overall, the experimentally tested and numerically evaluated girders collapsed due to shear.

From the above discussion and the figures, it can be understood that the finite element modeling of the tested SPG was done correctly. As a result, the proposed modeling is capable of accurately predicting the failure load.

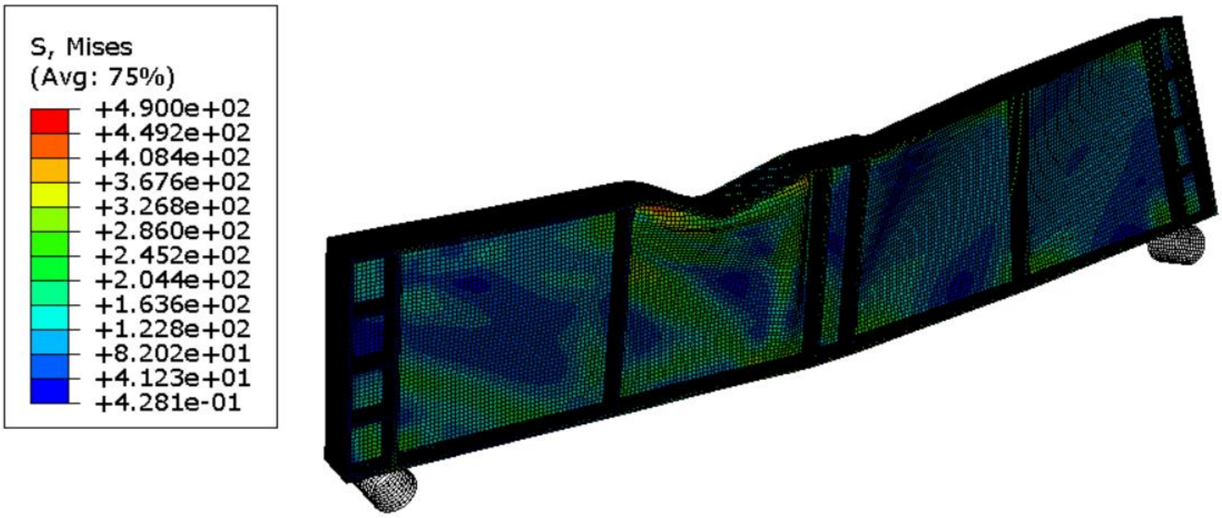


Figure 4-10 Von Mises stress distribution for the validated SPG

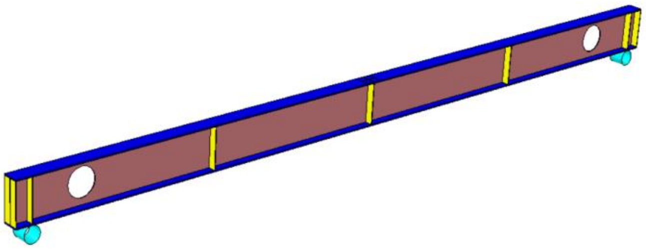
4.2 Results and Discussions

4.2.1 Parametric study

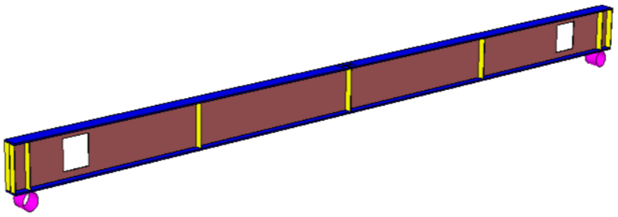
A parametric study of plate girders with webs that contain circular and square holes was carried out using the suggested finite element method. The same material properties were applied to each model. The parametric analysis considered the span length, opening type, opening depth, opening location, and number of openings on the SPG web.

The effect of the aforementioned parameters on plate girder ultimate shear capacity was explored by examining 45 girders with spans of 15m, 20m, and 25m with square and circular web openings. According to the validated finite element modelling, the proposed modelling has been used for the thorough evaluations of the SPGs. Under a concentrated load applied at the mid-span, the girders were tested to the point of collapse. The test results for ultimate loads, load-deflection relationships, deformation behavior, and failure characteristics are shown here.

It's interesting to note that plastic hinges are always produced near to the corners of web openings with asymmetrical yield patterns around the opening center, that is, yield patterns that are similar but face diagonally. The table 4.2 summarizes analysis results and sample models are displayed in figure 4.11.



(a) SPG with circular opening



(b) SPG with square opening

Figure 4-11 Sample models

Table 4-2 Analysis Result summary

No.	Span (mm)	SPG Label	Opening depth (mm)	Opening location from support (mm)	Number of openings	Ultimate load (kN)	Displacement at ultimate load (mm)
1	15000	WO	0	750	2	228.714	115.045
2	15000	C-OP-1	150	750	2	227.102	113.055
3	15000	C-OP-2	375	750	2	226.852	118.401
4	15000	C-OP-3	525	750	2	226.299	141.351
5	15000	S-OP-1	150	750	2	225.713	111.79
6	15000	S-OP-2	375	750	2	226.879	120.684
7	15000	S-OP-3	525	750	2	200.596	170.414
8	20000	WO	0	1000	2	421.365	160.639
9	20000	C-OP-1	200	1000	2	418.129	170.162
10	20000	C-OP-2	500	1000	2	417.081	164.301
11	20000	C-OP-3	700	1000	2	414.434	187.893
12	20000	S-OP-1	200	1000	2	414.75	171.844
13	20000	S-OP-2	500	1000	2	413.768	168.632
14	20000	S-OP-3	700	1000	2	395.769	233.066
15	25000	WO	0	1250	2	607.462	203.745
16	25000	C-OP-1	250	1250	2	602.2	206.606
17	25000	C-OP-2	625	1250	2	609.5	212.362
18	25000	C-OP-3	875	1250	2	608.9	240
19	25000	S-OP-1	250	1250	2	604.9	207.39
20	25000	S-OP-2	625	1250	2	607.5	216.831
21	25000	S-OP-3	875	1250	2	557.507	260
22	15000	C-OP-2	375	150	2	228.19	117.557
23	15000	C-OP-2	375	375	2	228.036	117.866
24	15000	C-OP-2	375	1125	2	229.178	119.357
25	15000	C-OP-2	375	1500	2	229.2	119.357
26	15000	S-OP-3	525	150	2	209.43	110.446
27	15000	S-OP-3	525	375	2	200.6	170.414
28	15000	S-OP-3	525	1125	2	208.876	202.134
29	15000	S-OP-3	525	1500	2	203.973	217.854
30	15000	S-OP-3	525	4100	2	186.594	96.647
31	20000	S-OP-2	500	1000	4	414.7	184.584
32	20000	S-OP-2	500	1000	6	412.927	192.931

33	20000	S-OP-2	500	1000	8	394.314	158.3
34	20000	S-OP-2	500	1000	10	389.97	199.97
35	15000	S-OP-2	375	750	4	225.772	130.333
36	15000	S-OP-2	375	750	6	225.987	136.344
37	15000	S-OP-2	375	750	8	225.267	131.887
38	15000	S-OP-2	375	750	10	211.747	150.902
39	25000	S-OP-2	625	1250	4	608.955	230
40	25000	S-OP-2	625	1250	6	608.645	247.102
41	25000	S-OP-2	625	1250	8	580	220
42	25000	S-OP-2	625	1250	10	560	260

4.2.2 Span vs Web opening

Three span lengths with the same L/d and d/t_w ratio having web openings of the same proportions and shapes were analyzed. It is difficult to make a direct comparison of all the specimens as they have proportional sizes of web and flange with respect to span.

Since all of them showed similar properties for web opening sizes up to 0.5d, SPGs with larger opening sizes with both square and circular opening shapes were compared among them.

The table below tries to compare the percentage reduction in ultimate shear capacity for the three spans. They responded the same for circular openings having almost the same ultimate shear capacity as the respective solid SPG but with an increased mid-span deflection of nearly equal amount, as shown in table 4.3. When the opening shape is changed to square, they also show the same response regarding ultimate shear capacity but with a variable proportion of mid-span deflection, 25m span deflecting less relative to a solid SPG.

Table 4-3 Span vs web opening

	C-OP-3		**S-OP-3**	
Span (m)	Ultimate shear capacity relative to solid SPG	Additional mid span deflection relative to solid SPG	Ultimate shear capacity relative to solid SPG	Additional mid span deflection relative to solid SPG
15	99%	22.86%	88%	48%
20	98%	16.97%	94%	45%
25	100%	17.79%	92%	28%

4.2.3 Effect of size and shape of openings on ultimate shear capacity

Figure 4.11 illustrates load versus mid span deflection curves for 15m span with square and circular openings of size 0.2d, 0.5d and 0.7d. From the figure, it can be noticed that the depth of openings up to 50% of the depth of web has no considerable effect on the ultimate capacity of the plate girder, so they behave the same as the solid girder. On the other hand, when the depth of opening is increased to 70%, the plate girder with a square opening loses around 12% of its ultimate shear capacity, and SPG with a circular opening deflects 18.6% more than the solid SPG.

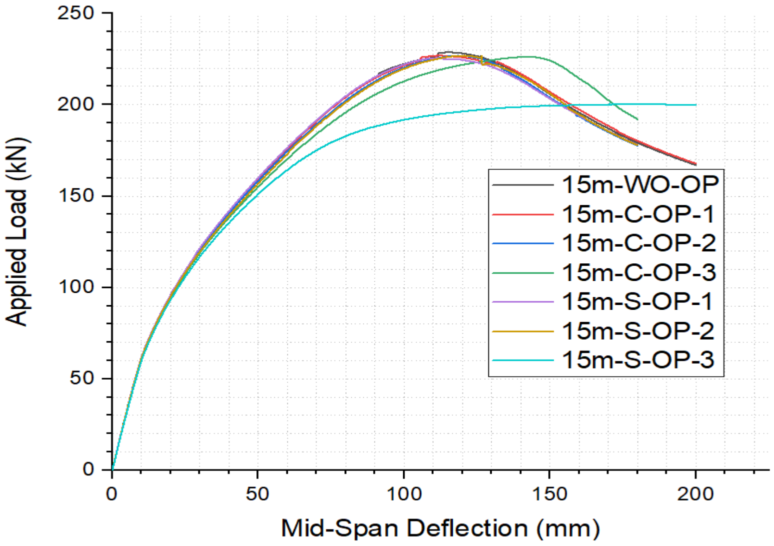


Figure 4-12 Load vs Mid-Span deflection of 15m SPG

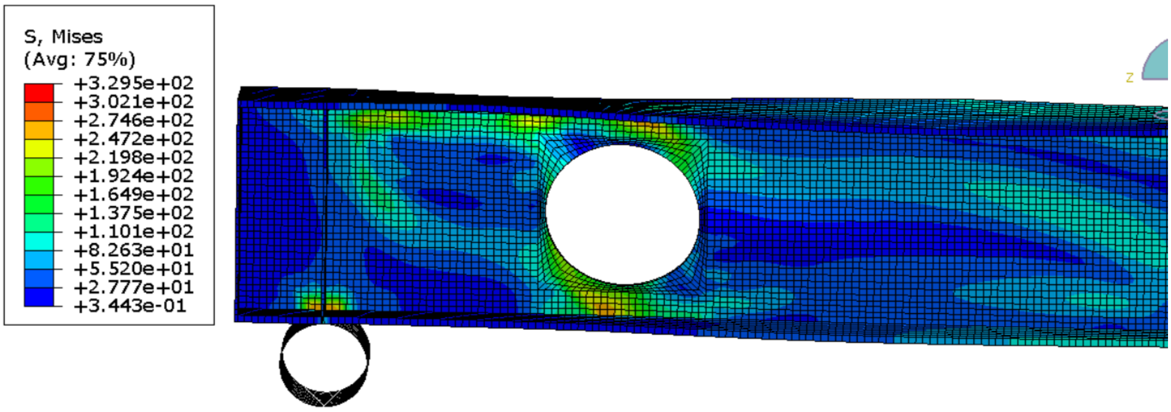


Figure 4-13 Von Mises stress for 15m-C-OP-3

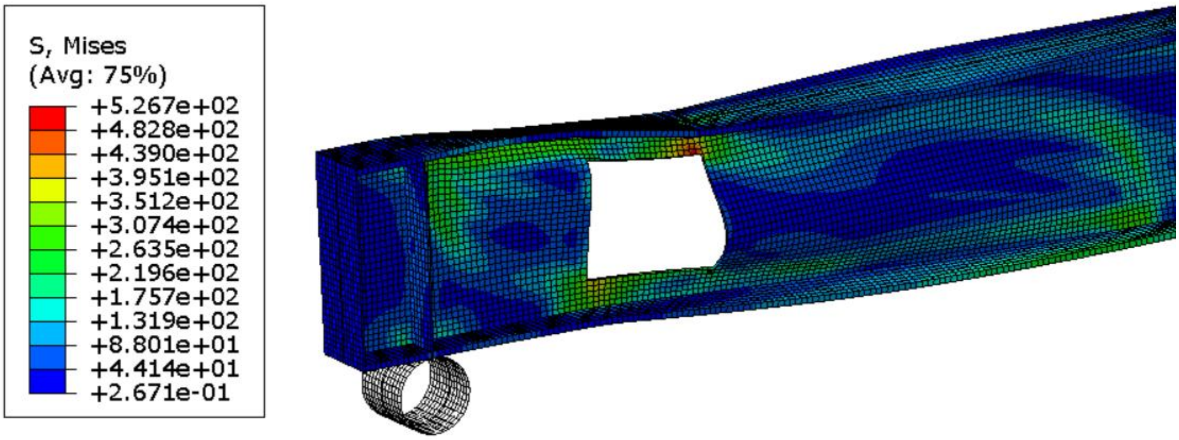
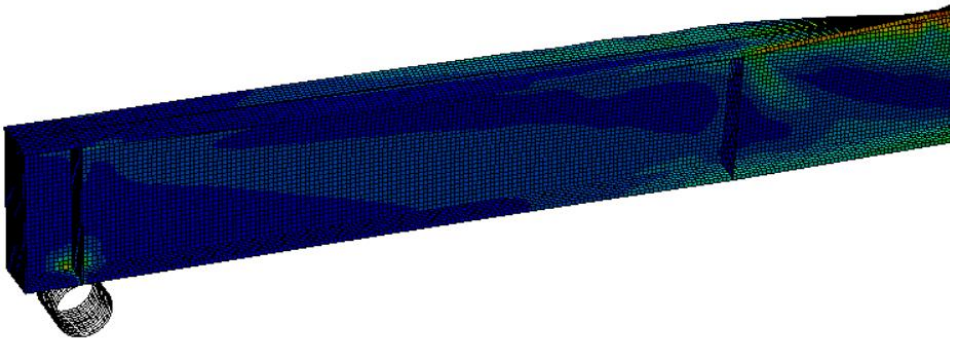
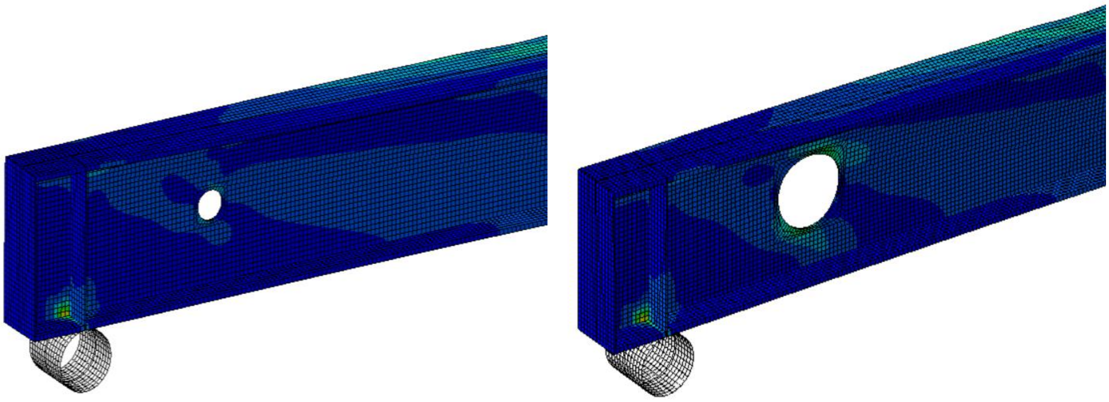


Figure 4-14 Von Mises stress for 15m-S-OP-3

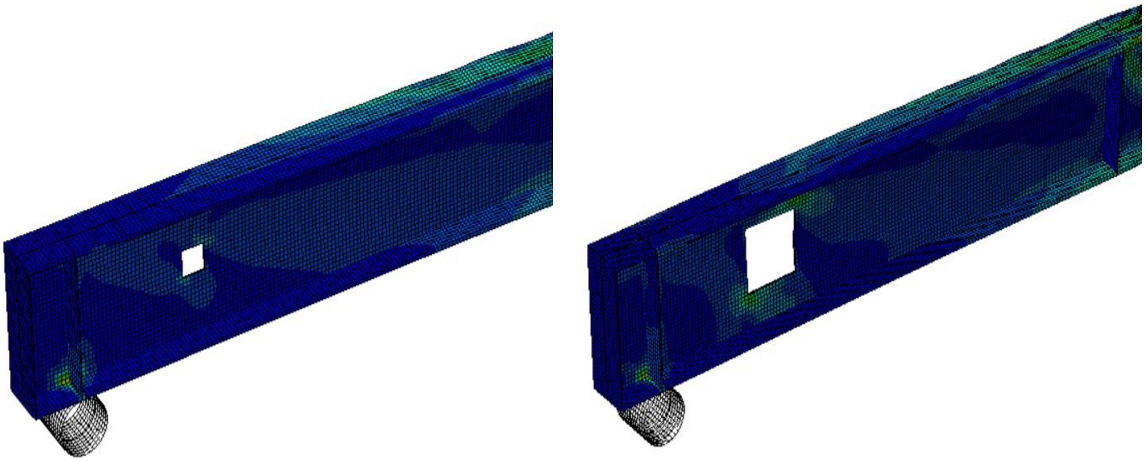


(a) 15m-WO-OP



(b) 15m-C-OP-1

(c) **15m-C-OP-2**



(d) 15m-S-OP-1 **(e) 15m-S-OP-2**

Figure 4-15 Von Mises Stress distribution for15m span (0.2d and 0.5d opening depth)

It can also be understood that the shape of the opening has a significant effect on the ultimate capacity of a girder when the depth of the opening is greater than 50% of depth of web. Therefore, plate girders with circular or square web openings behave the same until the depth of the opening reaches 0.5d.

When the depth of the opening increases, plate girders with circular web openings show better performance than those with square openings.

A plate girder with a circular opening of depth 0.7d failed at relatively the same load as the solid SPG but with additional 18.6% mid span deflection. On the other hand, a plate girder with a square opening of the same size as the circle failed at 87.7% of the failure load of the solid plate girder and with additional 32.5% mid span deflection. Therefore, SPG with a circular opening showed better performance than a square opening of the same depth for a depth of opening above 0.5d.

When the opening depth is greater than 0.5d, the circular opening is preferable to the square one because the stress in square openings is concentrated in their corners, as opposed to circular ones. The possibility of the perforated SPGs' web buckling increases as a result of these concentrated loads. On the other hand, compared to square openings with four angles, the shear stress distribution on the web of SPGs with circular openings is more uniform, as can be seen in the above figures 4.13 and 4.14. The SPGs with circular openings have better stress distribution on their web, which results in higher ultimate capacities than the SPGs with square openings. Figure 4.16 shows the stress concentrations around square web openings. As a result, while designing SPGs with web openings, the geometry of the opening is crucial.

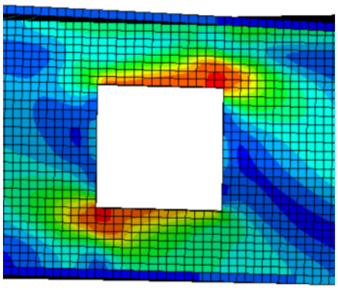


Figure 4-16 Stress concentrations around corners of square web opening

4.2.4 Effect of Location of openings on ultimate shear capacity

Ultimate capacity of plate girder with variable locations of openings from the support, 0.2d, 0.5d, d, 1.5d, 2d, 5.5d were analyzed, where d= depth of web.

Figure 4.16 illustrates that for plate girders with longer spans, having two circular web openings each at both sides of plate girder, and for locations up to 2d from the supports has no significant impact on the shear capacity of the girder.

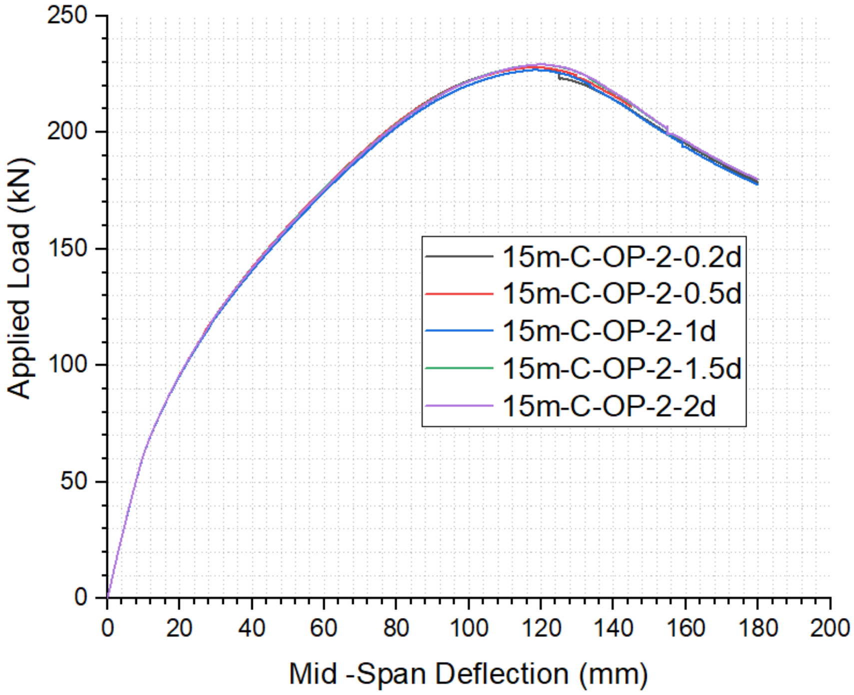


Figure 4-17 Load vs Mid span deflection for 15m -C-OP-2 varying opening location

From figure 4.16 it can be understood that when we go away from the support towards the load application point and cut the web, the shear capacity reduces greatly in comparison to SPGs having web openings up to a distance 2d from the support.

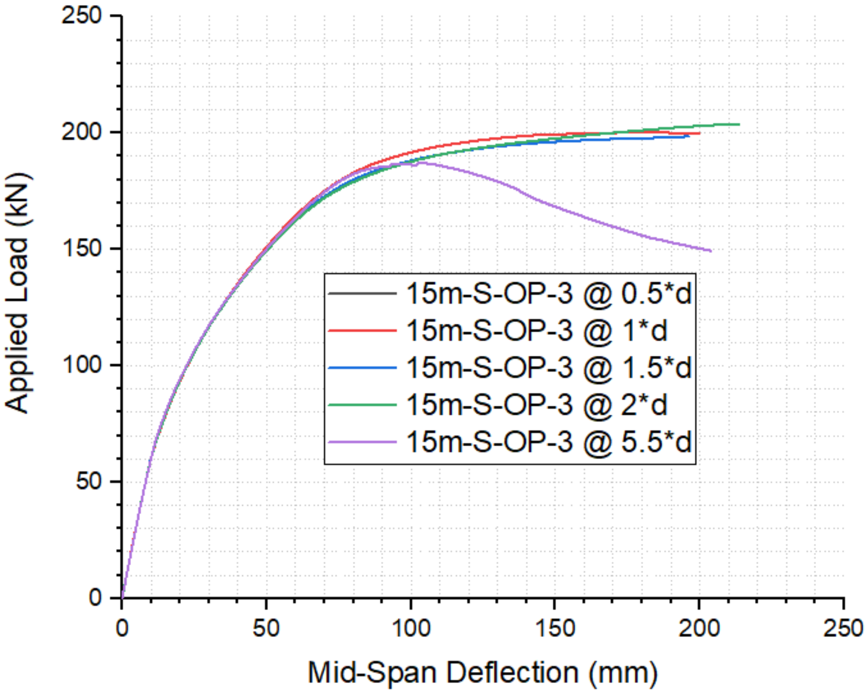
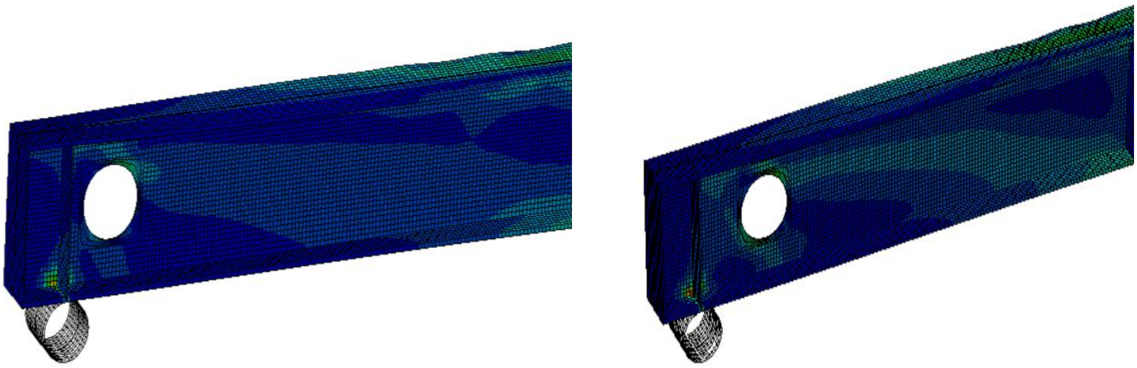
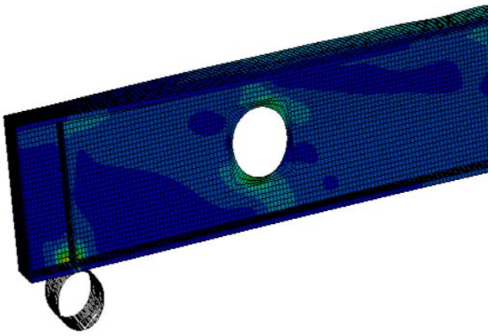


Figure 4-18 Load vs Mid span deflection for 15m-S-OP-3 varying opening location

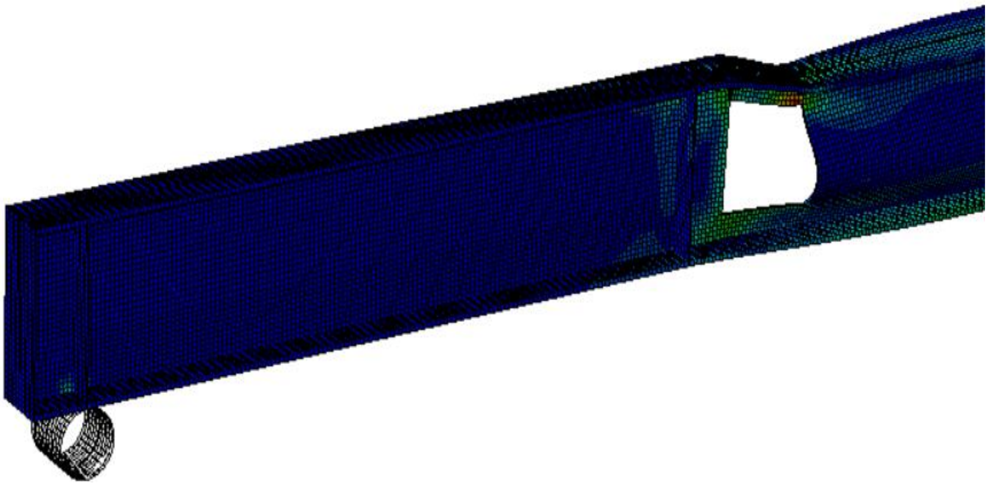
Sample model outputs for location of opening as a variable are shown in the figure 4.18.



(a) 15m-C-OP-2 at 0.2d distance **(b), 15m-C-OP-2pening at 0.5d distance**



(c) 15m-C-OP-2 opening at d distance

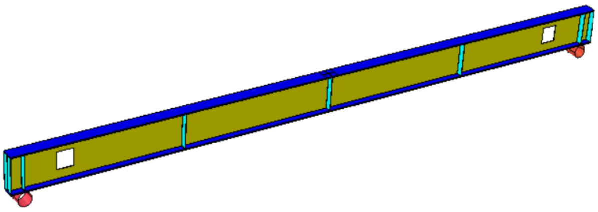


d, 15m-S-OP-3 at 5.5d distance

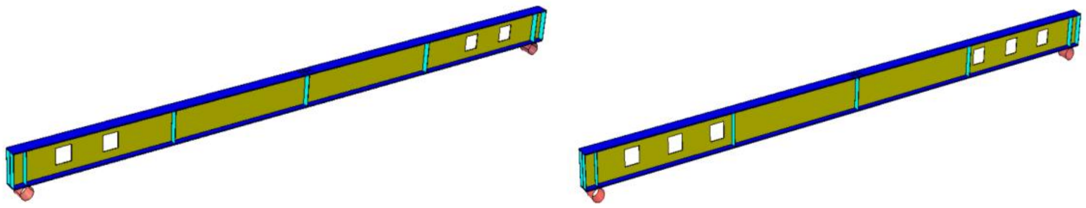
Figure 4-19 Von Mises stress for SPGs with openings located at different locations

4.2.5 Effect of number of openings on ultimate shear capacity

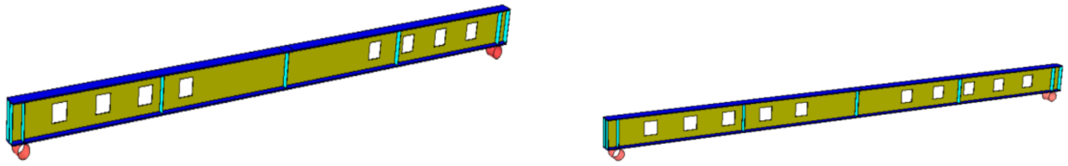
For the three considered spans with a square opening of depth 0.5d, the effect of increasing the number of openings was analyzed. The spacing between the additional and previous openings was equal to the depth of the web. From the figures 4.21, 4.22, and 4.23, we can understand that increasing the number of openings from two to ten, significantly reduces the ultimate load capacity up to 8% and promotes around 32% additional mid-span when the number of openings is increased.



(a) SPG with 2 openings



(b) SPG with 4 openings **(c) SPG with 6 openings**



(d) SPG with 8 openings **(e) SPG with 10 openings**

Figure 4-20 Plate girders with variable number of openings

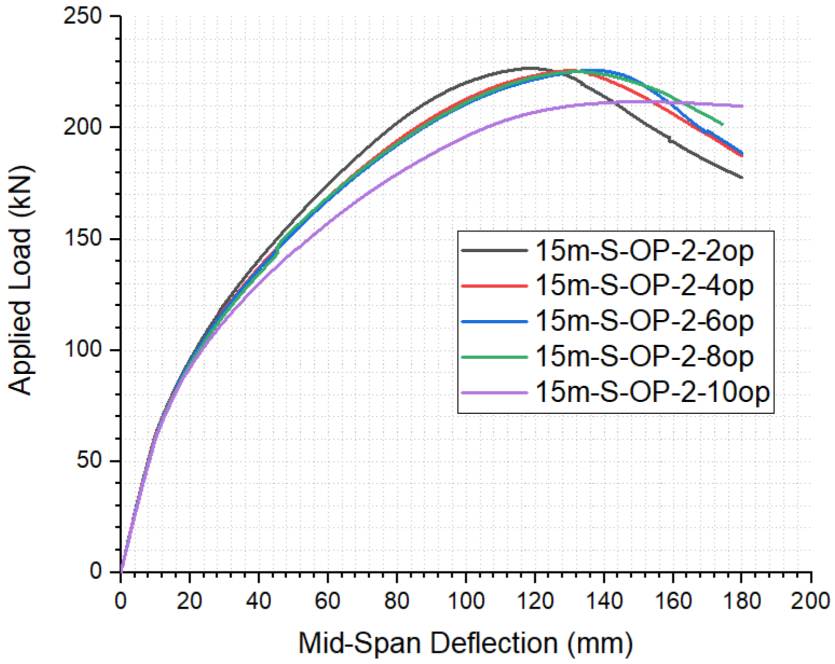


Figure 4-21 Load vs Mid span deflection curve for 15m-S-OP-2 with openings ranging from two to ten

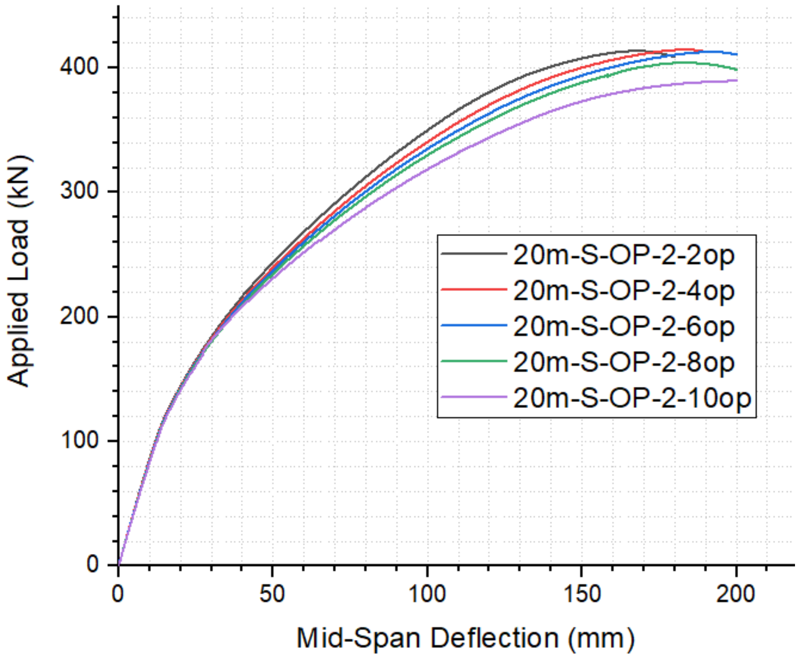


Figure 4-22 Load vs Mid span deflection curve for 20m-S-OP-2 with openings ranging from two to ten

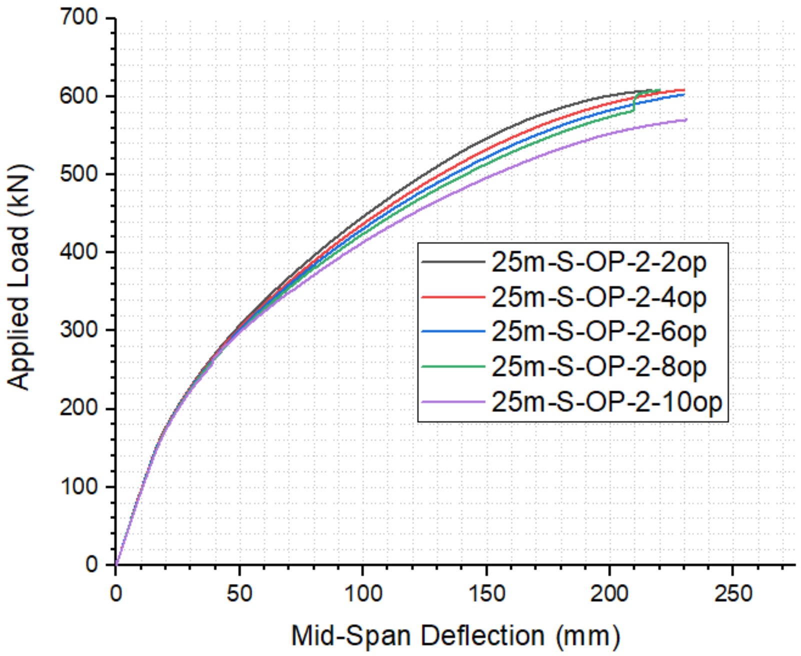


Figure 4-23 Load vs Mid span deflection curve for 25m-S-OP-2 with openings ranging from two to ten

4.2.6 Comparison of results with different researchers

Though earlier studies with exactly the same parameters, spans, loading, and support conditions are not available, common findings were investigated between the current study and past research. The common findings are listed below:

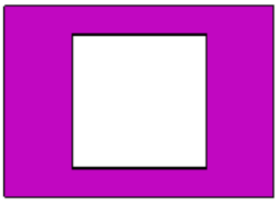
- The shape of the holes has a considerable impact on the ultimate shear capacity of plate girders, with circular openings being preferable.
- The use of steel stiffeners around web openings enhances the ultimate shear capacity of the perforated plate girder.
- The total mid-span deflection of the plate girder is affected by the presence of openings.
- Increasing the number of openings greatly affects mid span deflection rather than the shear capacity of the plate girder.

4.2.7 Proposed Remedial measures for the weak steel plate girders

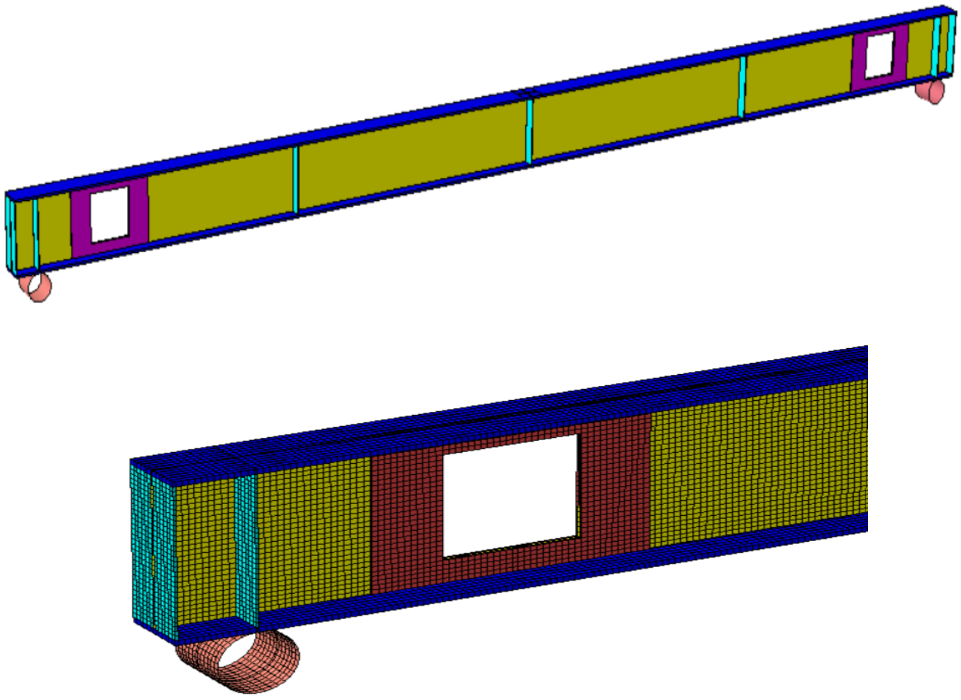
It is clear from the discussion above that the shear capacity of plate girders with square opening heights equal to 0.7d was seriously compromised. Despite this, there are some circumstances where using these openings is unavoidable. The following three corrective actions are suggested for these situations to strengthen SPGs with openings deeper than 0.5d.

*4.2.7.1 **Providing a plate around the opening (R1)***

It is well known that the shear capacity of SPG with web openings is proportional to the thickness of the web. Rather than thickening the entire length of the web and increasing its weight, we can weld a plate around the opening with the same area as the web's lost area on both sides, as indicated in the picture below.



(a) Plate to be provided

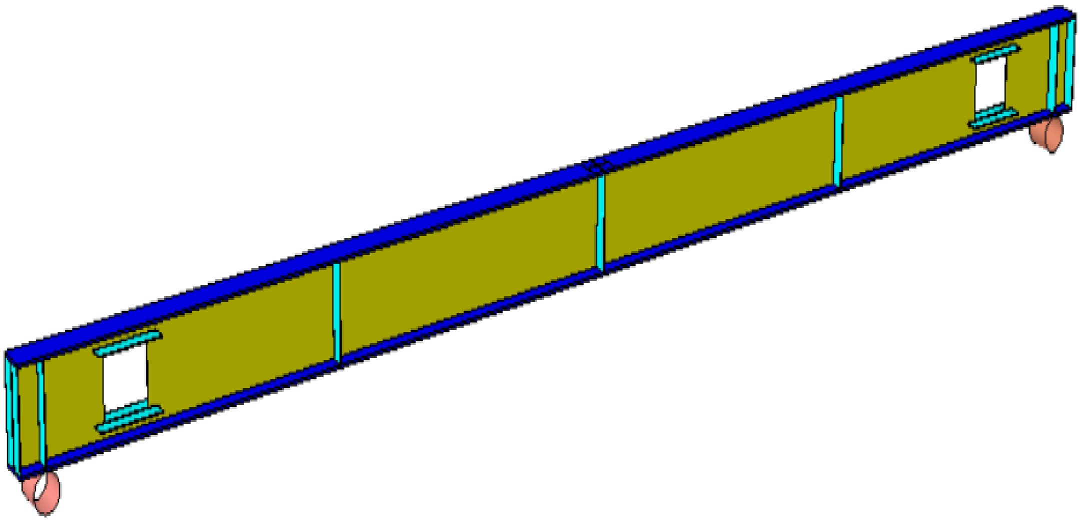


(b) SPG with plate around the opening

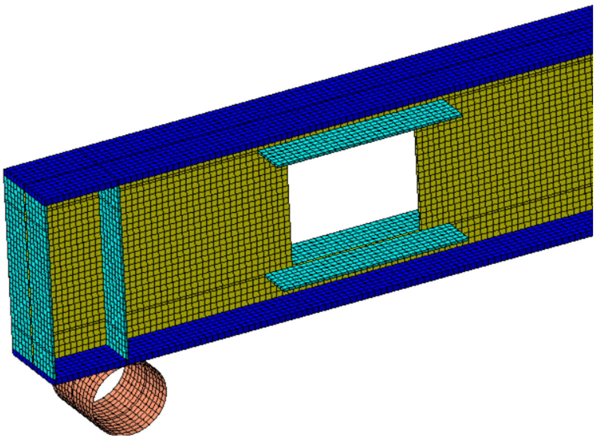
Figure 4-24 15m-S-OP-3-plate welded around the opening

4.2.7.2 ***Providing longitudinal stiffener above and below the opening (R2)***

To strengthen the Vierendeel bending resistance, welded horizontal reinforcements (stiffeners) are sometimes required above and below the openings (Chung and Lawson, 2001) and longitudinal stiffener increases the web's buckling resistance (Garg, 2017) .



(a)

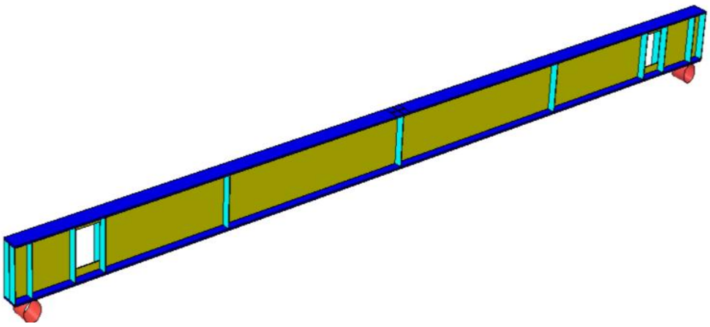


(b)

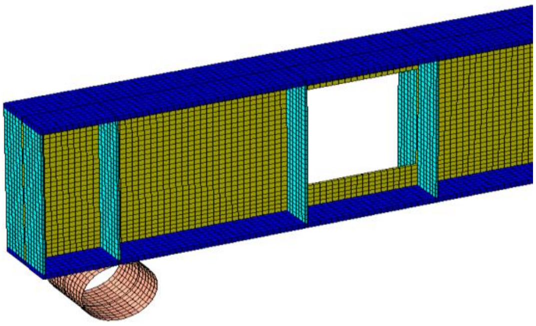
Figure 4-25 15m-S-OP-3 SPG with longitudinal stiffener around web opening

4.2.7.3 *Utilizing Vertical stiffener at the left and right side of the opening (R3)*

Adding transverse stiffeners is one approach to increase a thin web plate's ability to support loads, primarily improving the shear buckling strength of the web (Subramanian, 2010).



(a)



(b)

Figure 4-26 15m-S-OP-3 SPG with longitudinal stiffener around web opening

Table 4-4- Comparison of 15m-S-OP-3 and remedial measures with solid plate girder

		Ultimate shear capacity	Mid span deflection
15m-S-OP-3	without stiffening	87.7%	148%
	R1	105.4%	91%
	R2	99.6%	111%
	R3	102.2%	101%

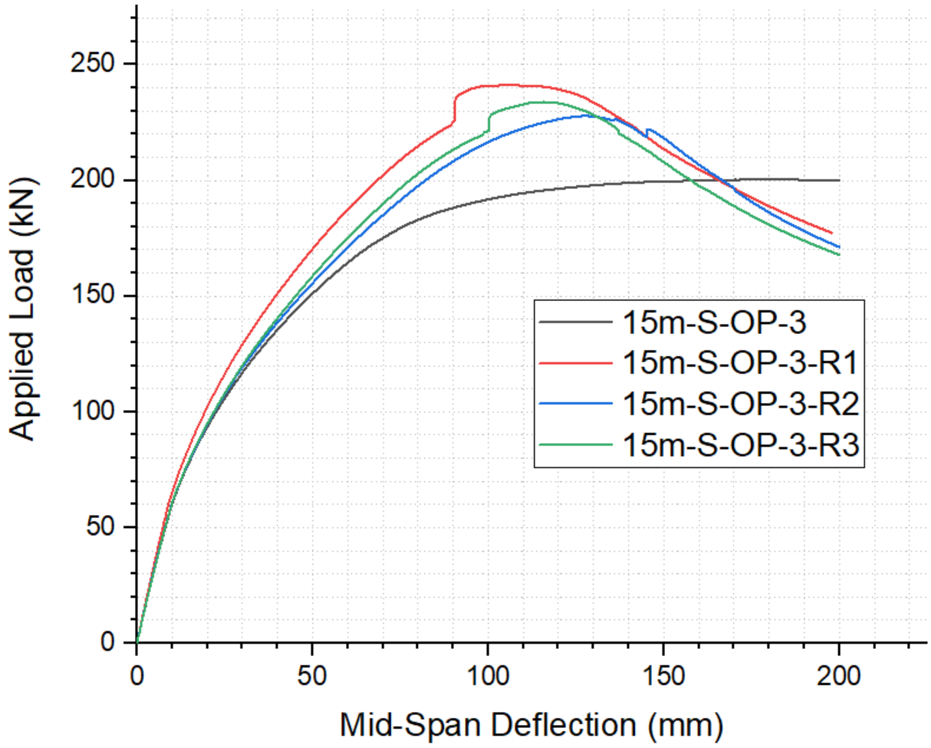


Figure 4-27 Comparison of shear capacity among proposed remedial measures

15m span SPG with 0.7d opening depth is analyzed being provided the remedial measures listed above. From the figure 4.24 and table 4-4 we can understand that providing a plate having an equivalent area to the lost web area performs better than providing stiffeners around the opening with higher ultimate shear capacity and lower mid span deflection. On the other hand, SPG with vertical stiffener at the left and right side of the opening performed better than SPG with longitudinal stiffener at the top and bottom of the opening.

4.2.8 Sensitivity Analysis

The study of how variation (uncertainty) in the output of a statistical model can be attributed to different variations in the model's inputs is known as sensitivity analysis. It is used to determine the importance of random variables. The sensitivity of each random variable is represented by the squared value of the partial coefficient of correlation (r_p^2). Inputs for the sensitivity analysis are taken from table 4.2. For uncertainty analysis, the sensitivity factor α_i based on the first-order approximation second-moment method is used. to determine effects of random variables (Tsubaki et al., 1992). The sensitivity factor is obtained as follows.

$$\alpha_i = \frac{\partial F}{\partial x_i} \frac{\overline{x}_i}{\overline{F}} \qquad (i = 1, 2, 3,. .., n)$$

where,

- α_i: sensitivity factor of random variable *i*
- *F*: function with statistical variations
- $\overline{F}$: mean of *F*

The sensitivity factor α_i is a kind of index to estimate the contribution of the uncertainty of x_i to the uncertainty of *F*. The contributions of the uncertainty of each random variable are obtained by multiplying the sensitivity factor by its coefficient of variation.

$$U_{x.F\,i} = \alpha_i (COV_i)$$

where,

- $U_{x.Fi}$: contributions of the uncertainty of random variable *i*
- COV_i: coefficient of variation for random variable *i*
- α_i: sensitivity factor of random variable

Using regression analysis, the general function expressing the shear capacity of the section is given as:

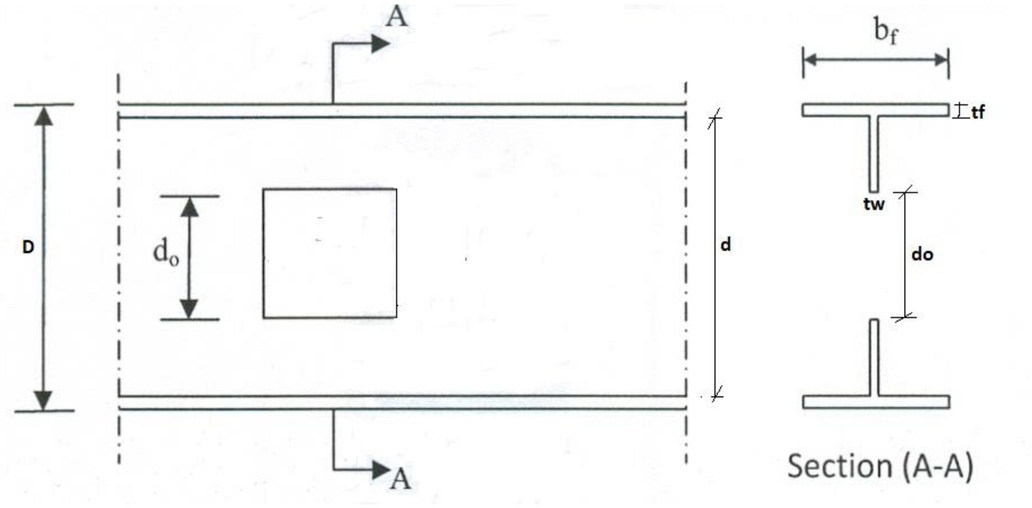
$$V_{pl,Rd,o} = V_{pl,Rd} * \eta$$

Where:

$V_{pl,Rd,o}$ is the shear strength of steel plates with holes

$V_{pl,Rd}$ is the shear strength of steel plates without holes

η is the reduction factor considering web opening



$$V_{pl,Rd} = (d - d_o\,)t_w * \frac{fy}{\sqrt{3} * \gamma_{mo}}$$

$$\eta = \left(\frac{d}{t_w}\right)^{-0.592} * A_f{}^{-0.372} * d^{0.683} * \left(\frac{1}{Lo}\right)^{0.003}$$

where

- d - depth of web
- t_w- thickness of web
- A_f – cross sectional area of flange $=b_f*t_f$ (included so as to account for the contribution of the flange to shear capacity)
- d_o – diameter of hole for circular opening / size of hole for square opening
- L_o=Location of opening from center of support

The sensitivity and uncertainty factors for the variables are computed and shown in table 4.5.

Table 4-5 Sensitivity and uncertainty factors

Random variables (i)	**Sensitivity factor (α_i)**	**COV (%)**	**Uncertainty factor (Ui)**
d/t_w	-0.32	5.00	-1.6
A_f	-0.069	44.70	-0.03
d_o	274.69	47.10	129.39
Lo	-0.018	55.93	-0.01

The negative sign in table 4.5 implies that the effect of the parameter is inverse to the strength. The most influential parameter is the diameter of holes and it is in good agreement with the parametric study done via finite element analysis, in which the depth of opening is the sensitive parameter of all the other parameters used in the study.

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CHAPTER 5 CONCLUSIONS AND RECCOMENDATIONS

5.1 Conclusions

This paper examined SPGs with circular and square web openings. The SPGs were analyzed using the finite element software ABAQUS. The modeling's accuracy was validated by comparing the experimental test results to the modeling results. The SPGs were then created. For the SPG analyses, a variety of variables were considered. The variables were span length, opening depth, opening location, opening shape, and number of openings. The effects of these variables on the SPGs' ultimate shear capacity were investigated.

According to the study's findings for the three spans under consideration, beams with openings deeper than 50% of the web depth would fail by the Vierendeel mechanism and exhibit a reduction in ultimate shear capacity as opposed to those with narrower opening depths, which displayed nearly comparable shear capacity to the solid SPG. The circular-opening beam provided better structural performance.

Increasing number of openings resulted in lower ultimate capacity and additional mid span deflections to 32%.

Additionally, decreasing the distance between the web opening and the support provides better resistance to shear capacity than placing the opening close to the concentrated load. The presence of openings appeared to have almost no effect on the shear capacity up to a distance of 2d from the openings.

The ultimate shear capacity of SPGs with openings deeper than 0.5d was increased more effectively by welding a plate around the openings than by welding stiffeners around the openings. All of the spans that were taken into consideration responded to the parameters essentially in the same way. The depth of the opening was also found to be the most sensitive parameter in the parametric study.

5.2 Recommendations

- The study is limited to static concentrated load at the plate girder's mid-span, but more studies involving cyclic load should be done for a broader application.
- It is also advised to investigate span lengths longer than 25 meters with an increased number of holes.
- As the study employed constant stiffener thickness and d/t_w ratios, altering those constants and examining their influence on ultimate shear capacity requires additional research.

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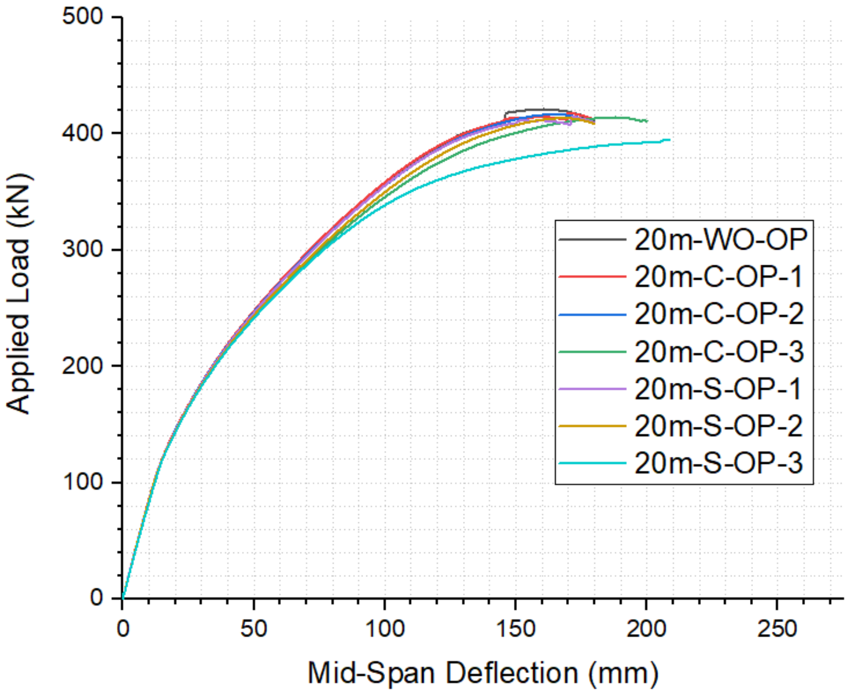
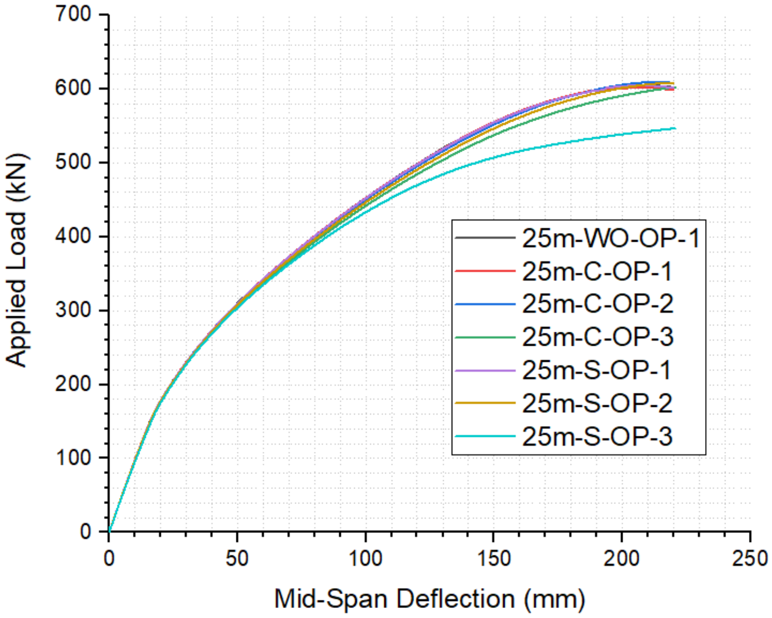
APPENDIX

APPENDIX-A

Load vs Mid span deflection for 20m and 25m span plate girders



Load vs Mid span deflection for 20m span with circular and square web openings

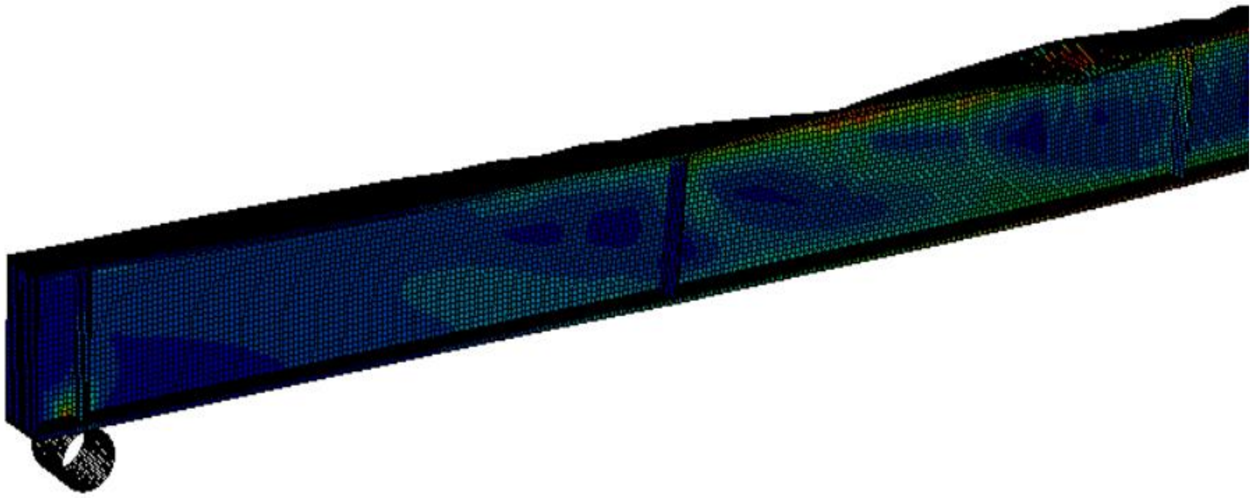


Load vs Mid span deflection for 20m span with circular and square web openings

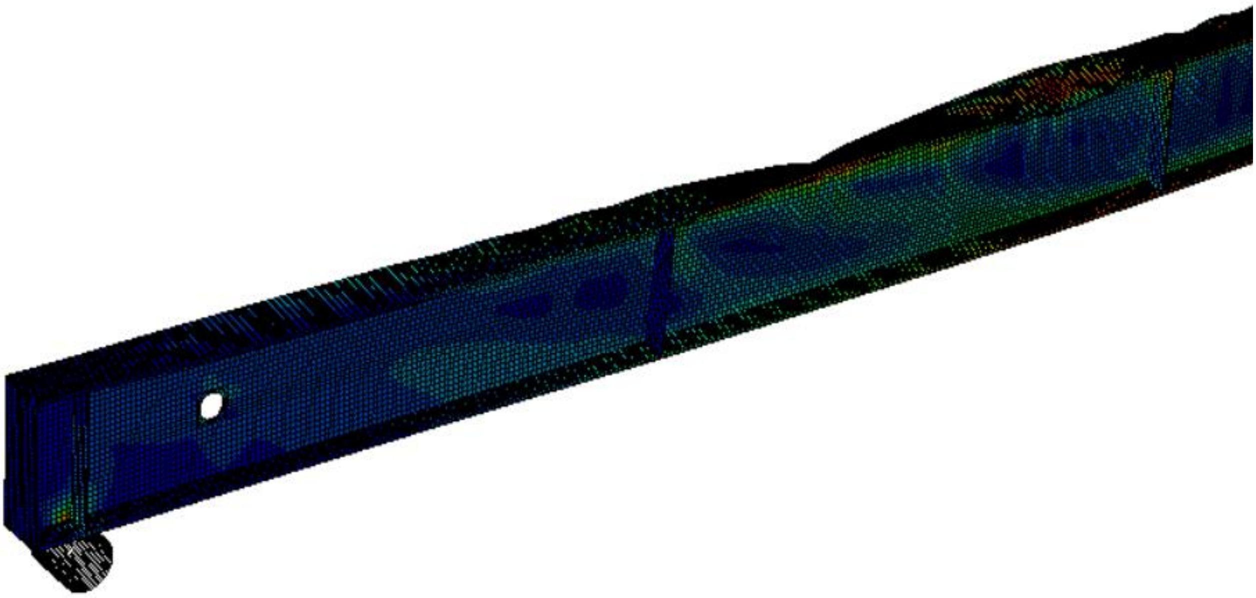
APPENDIX-B

Von Mises stress distribution for 20m span plate girders with different shapes and depths of openings (since the beams are symmetrical, half of spans are displayed here):

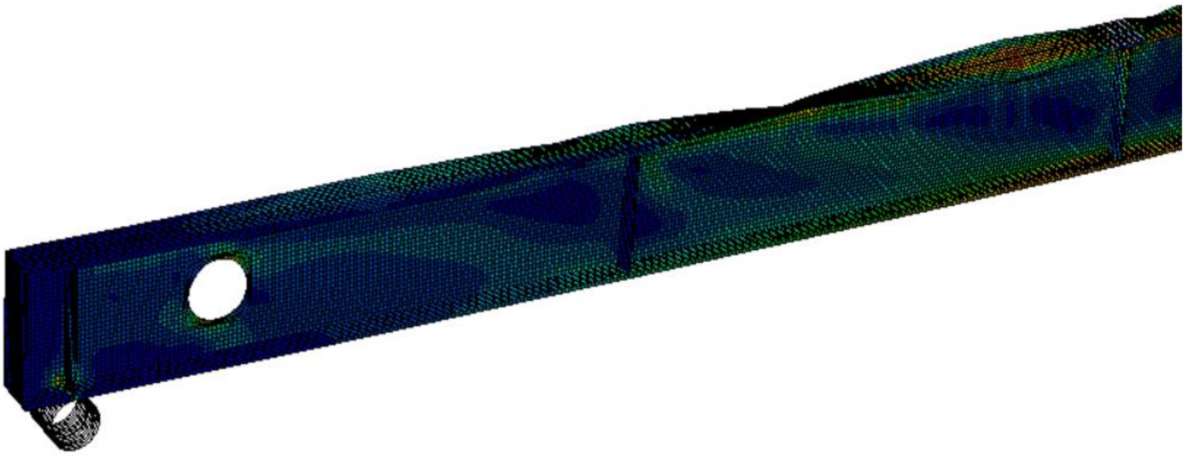
Areas with red color show the highest stress, whereas those in blue indicates regions of low stress. Color values for intermediate intensities are interpolated from cyan, green and yellow.



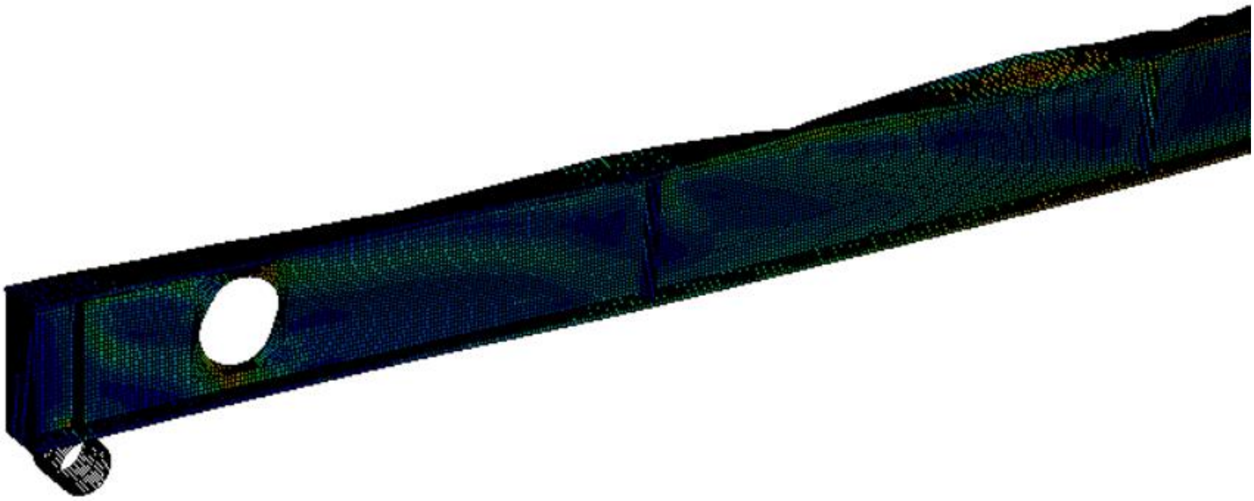
20m-WO-OP



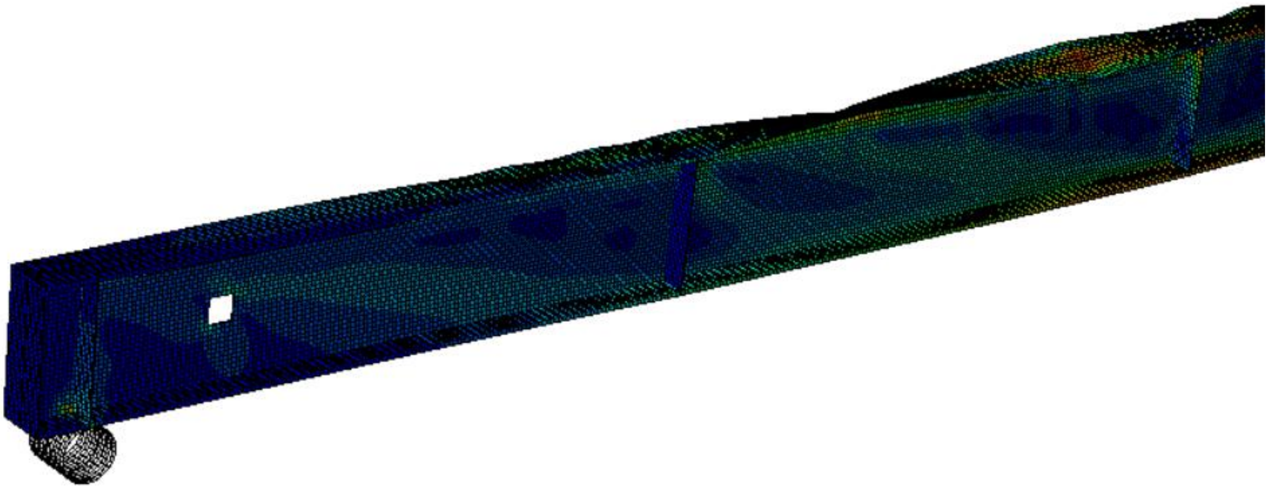
20m-C-OP-1



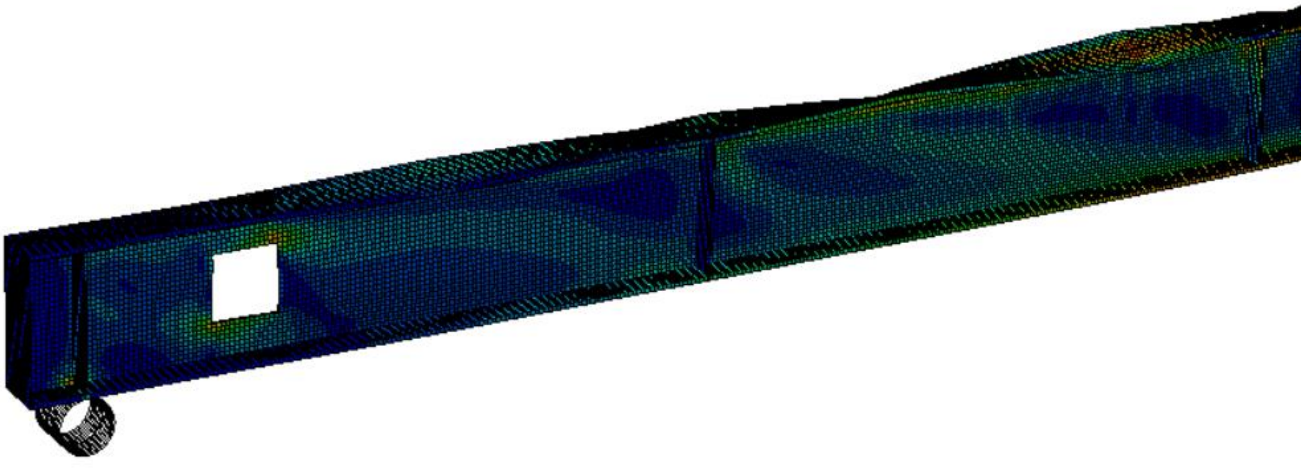
20m-C-OP-2



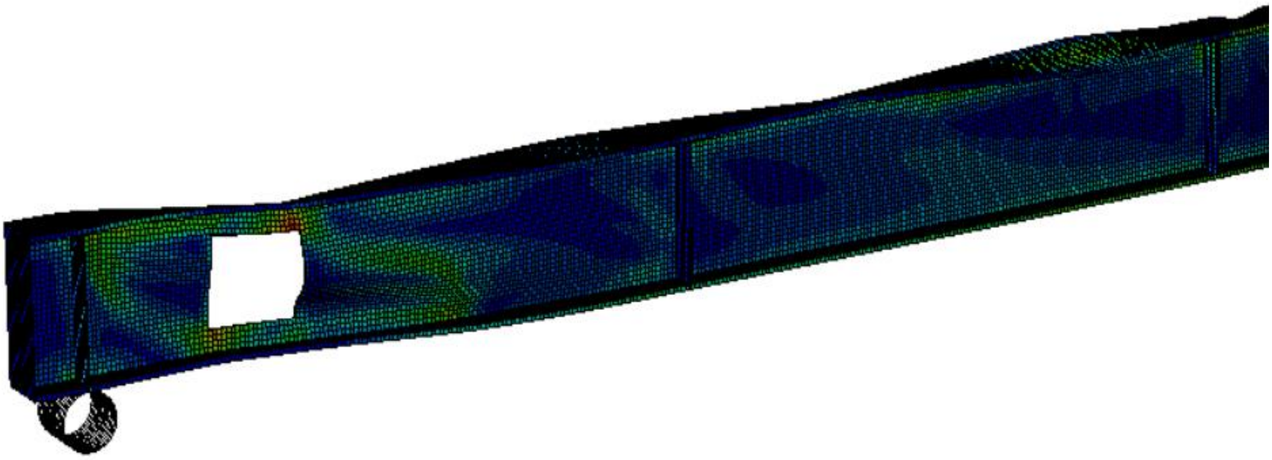
20m-C-OP-3



20m-S-OP-1



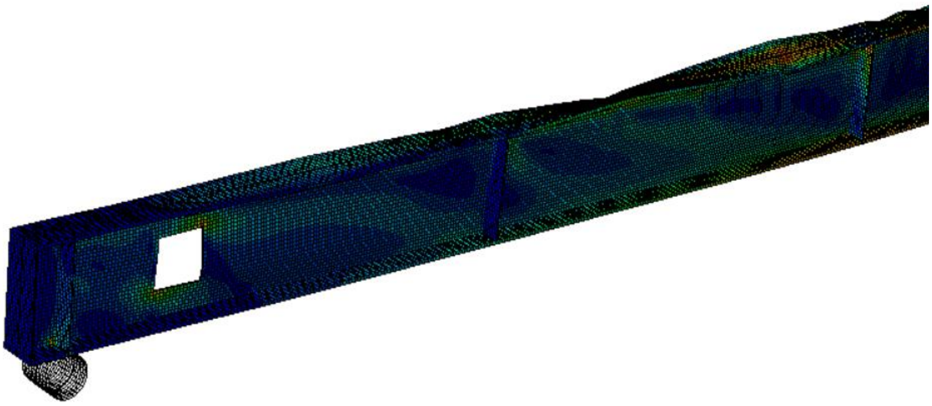
20m-S-OP-2



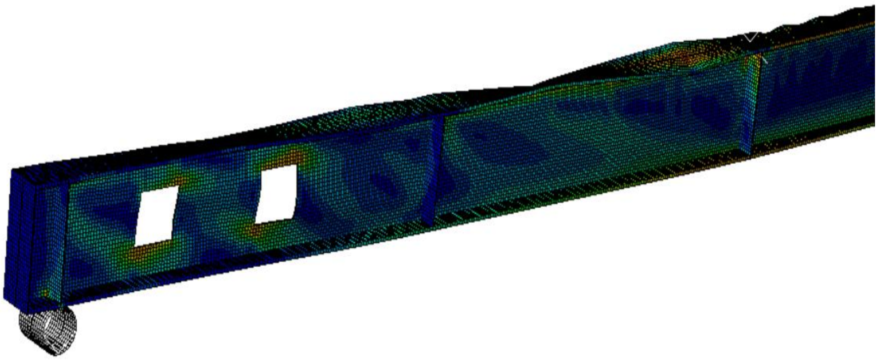
20m-S-OP-3

APPENDIX-C

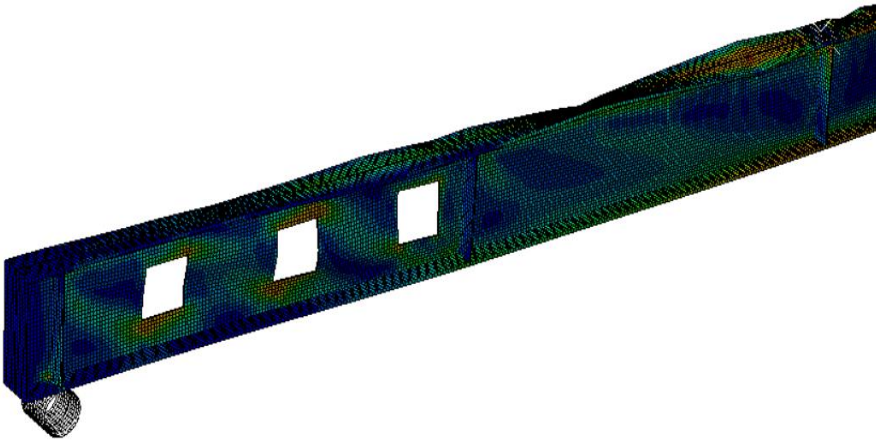
Von Mises stress distribution for 20m span plate girder with number of openings as a variable



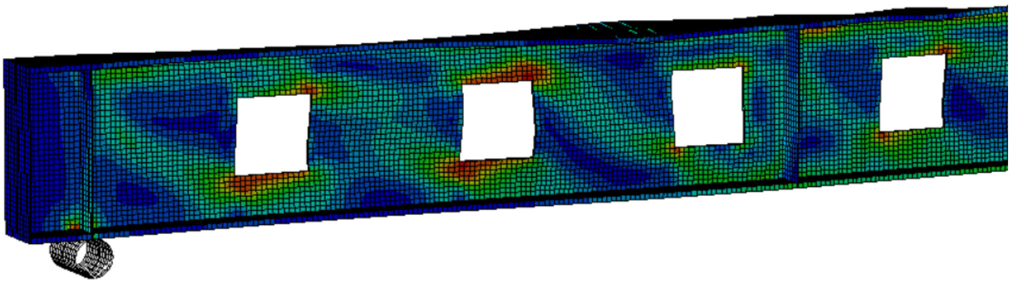
20m span SPG with two openings with depth of 0.5d



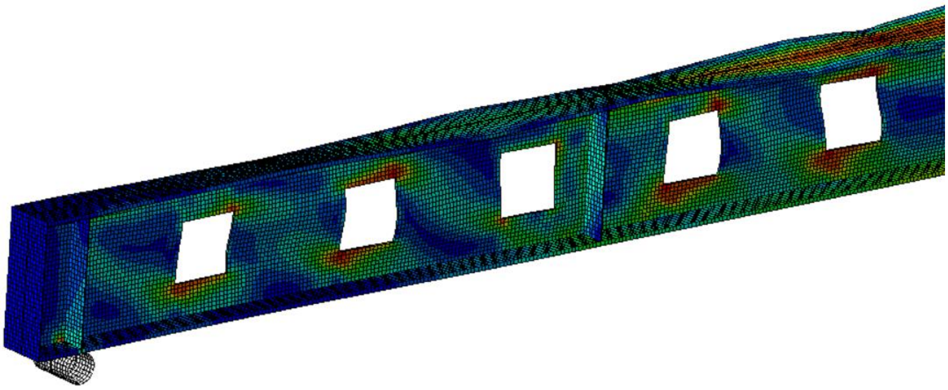
20m span SPG with four openings with depth of 0.5d



20m span SPG with six openings with depth of 0.5d



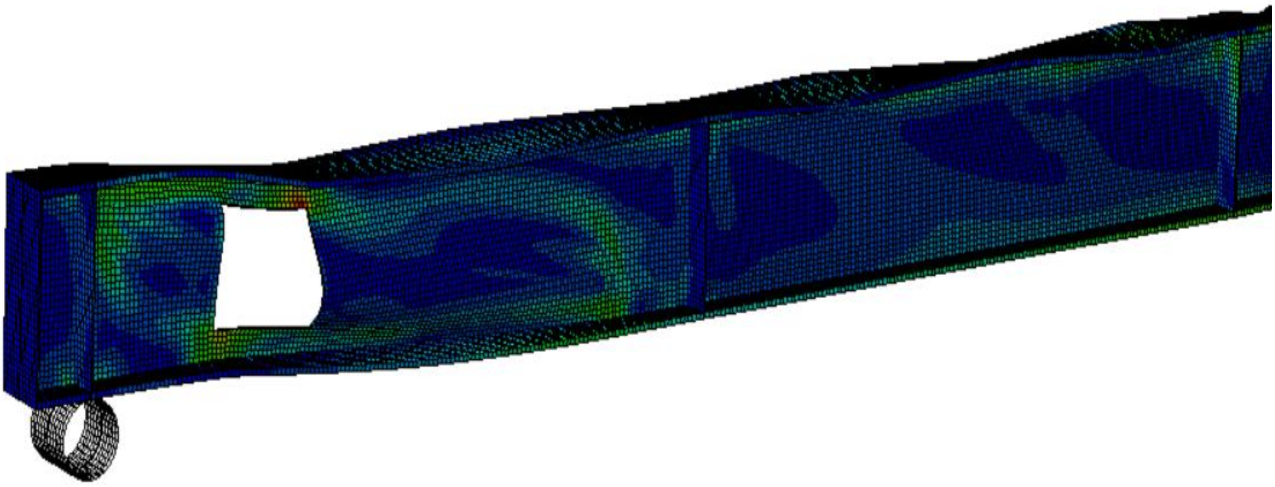
20m span SPG with eight openings with depth of 0.5d



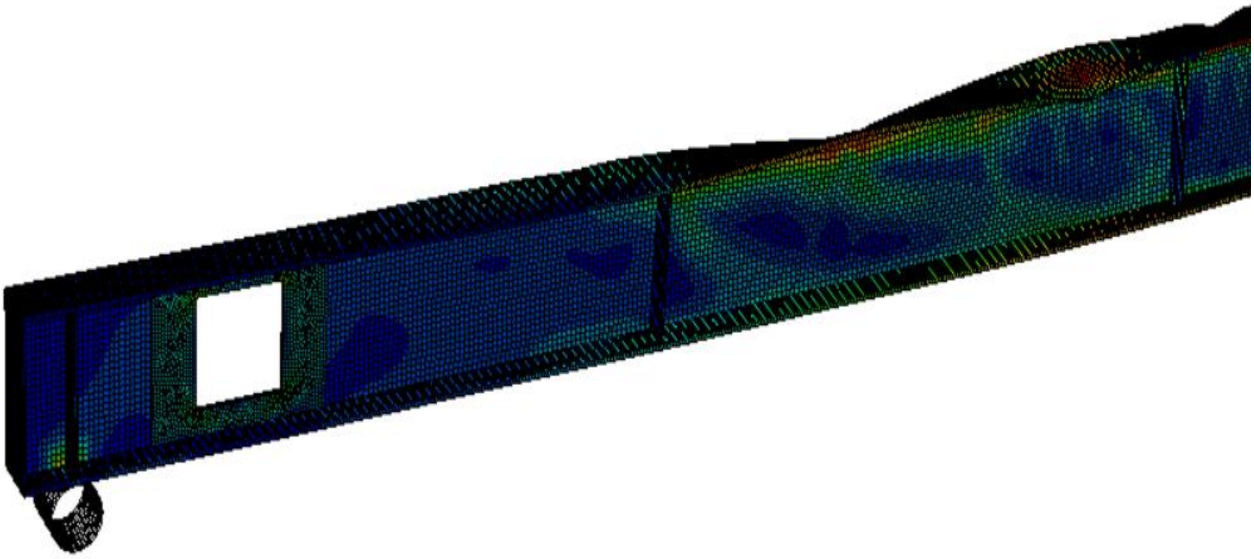
20m span SPG with ten openings with depth of 0.5d

APPENDIX-D

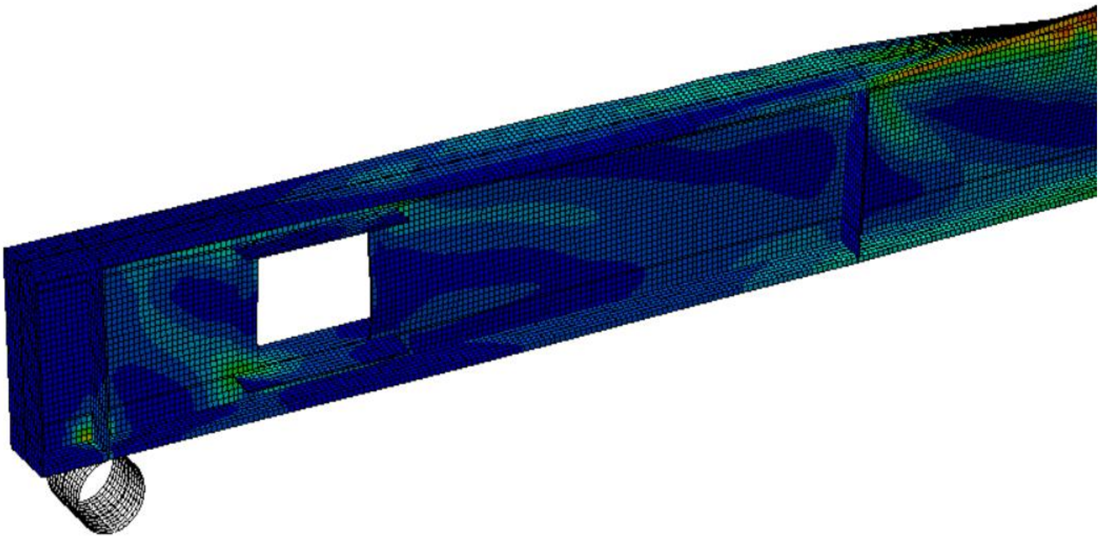
Von Mises stress distribution for the proposes remedial measures



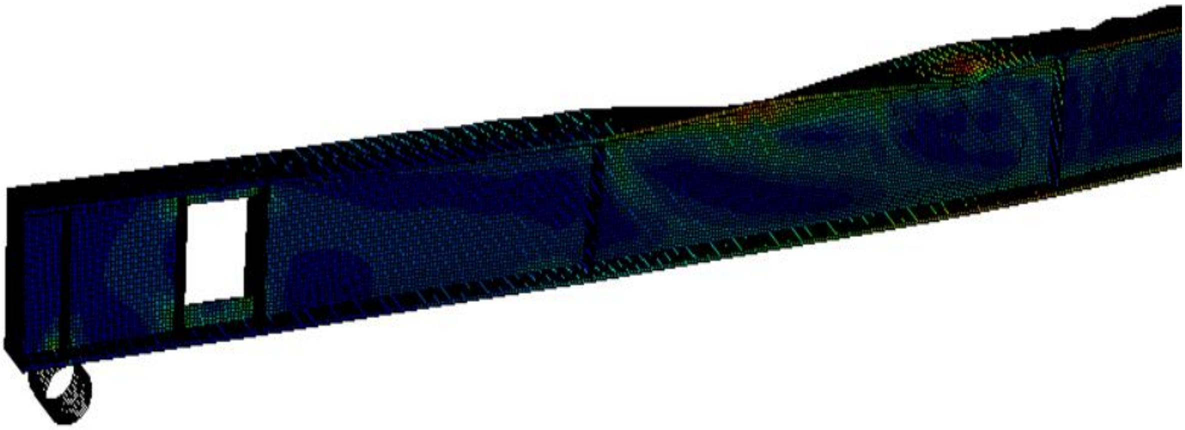
15m-S-OP-3- without any measure



15m-S-OP-3-provided with plate around the opening



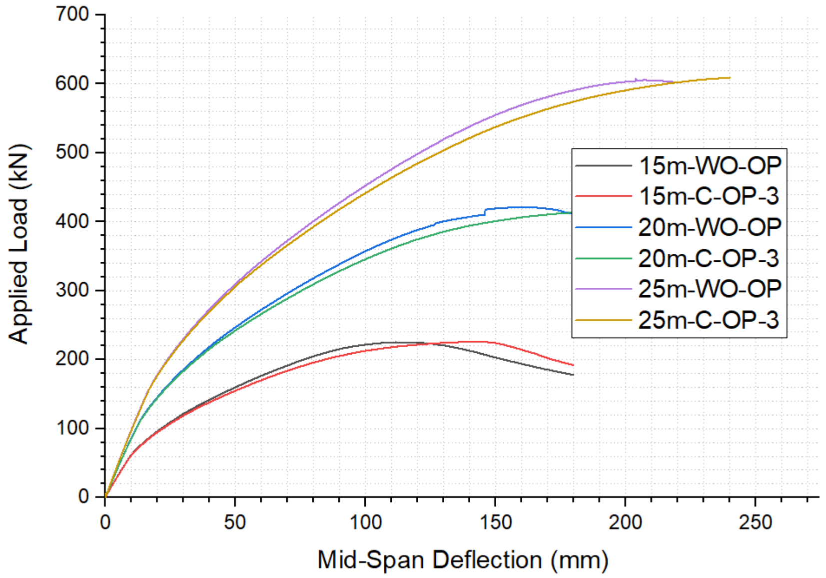
15m-S-OP-3-with horizontal stiffener around the opening



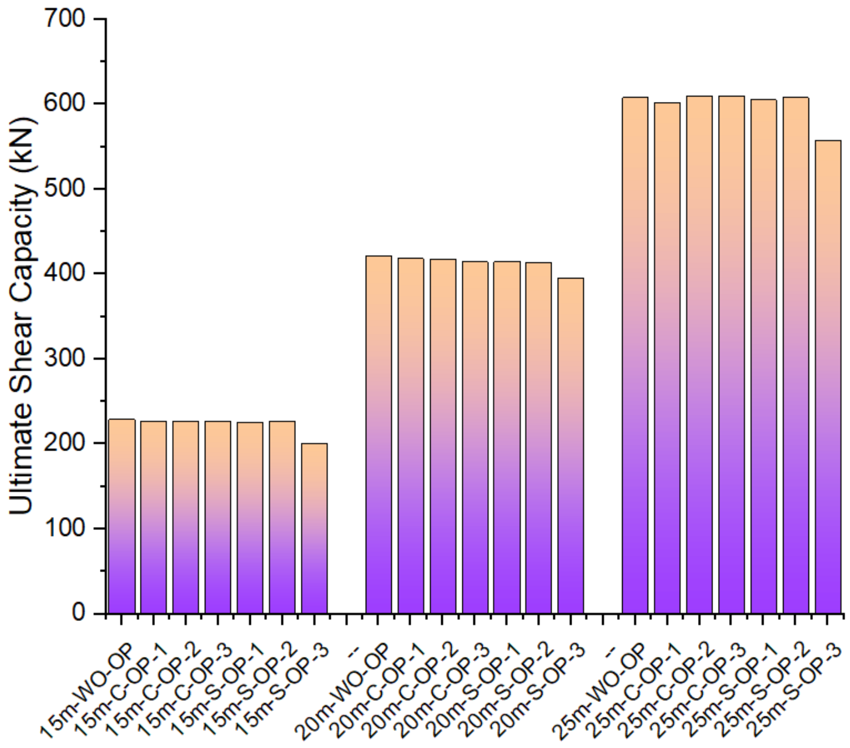
15m-S-OP-3-with vertical stiffener around the opening

APPENDIX-E

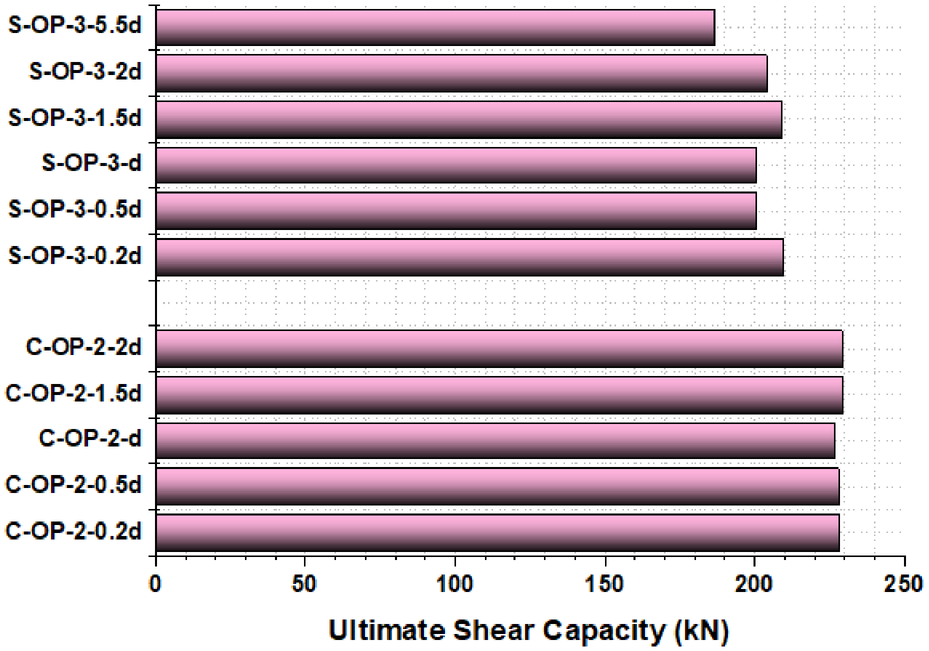
Comparisons of ultimate shear capacity using parameters studied above



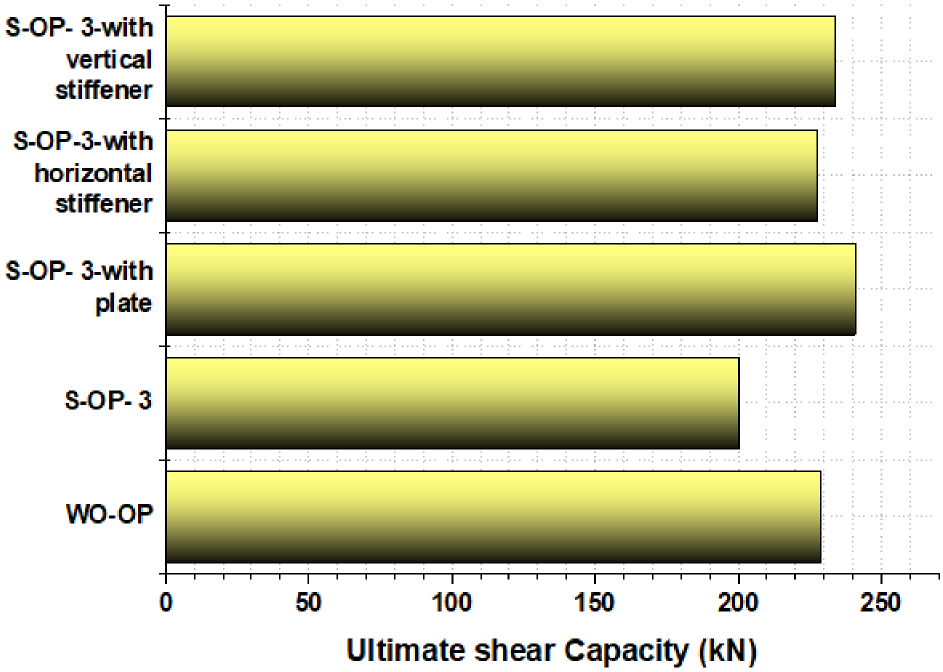
Applied load vs mid span deflection curve showing how each span respond to circular opening of depth 0.7d



Comparison of Ultimate Shear Capacity for all spans, shapes and depths of openings considered



Ultimate shear capacity vs Location of openings



Comparison of the proposed remedial measures for weak SPGs with the solid SPG