JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

# Local Buckling of Plate Girder with Longitudinal Web Stiffener

A Thesis Submitted to School of Graduate Studies of Jimma University in Partial Fulfillment of the Requirement for the Degree of Masters of Science in Structural Engineering

By SOLOMON HAILE GETAHUN

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By SOLOMON HAILE GETAHUN

MAIN ADVISOR: Eng. Elmer C. Agon

CO-ADVISOR: Eng. Yehamleshet Menberu

November 2019

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# DECLARATION

I, SOLOMON HAILE, declare that all the work done in this study originates from my own work and that all secondary sources referred to in this work have been duly acknowledged.

Name: SOLOMON HAILE Signature: ___________________ Date: ________

| | | | |
|---|---|---|---|
| Advisor: | ________________________ | ___________________ | ____________ |
| | Name | Sign | Date |
| Co-Advisor: | _____________________ | ___________________ | ____________ |
| | Name | Sign | Date |

# ABSTRACT

*In the design of plate girders, the best approach is to put the material as far from the neutral axis as possible and keep the web area as small as possible. This approach maximized the section's efficiency for strength. As a result of this principle, Plate girders used in bridges are usually deep beams with relatively thin webs and, subsequently, web buckling becomes an important factor to consider during their design. When the limit state of web buckling governs the design, transverse and longitudinal stiffeners may be used to increase section strength. Therefore; an Optimum location of longitudinal stiffeners in plate girders under high shear low moment near support were investigated in this study.*

*This study explores simple span, plate girders with yield stresses of 240Mpa for flange and 280MPa of web and investigates optimal horizontal single stiffener location. Parametric studies are conducted for a range of slenderness ratio, aspect ratio and relative location of longitudinal stiffener with the web using ABAQUS, and sections are examined under where high shear-low moment conditions near supports. The girders studied were divided into three groups. Each group is composed of seven girders.*

*A model for each combination of parameters is built, and seven cases are analyzed. In the first group, the second group and the third group the percentage increased the value of Pcr at 0.5D location of the longitudinal stiffener is 173%, 162%, and 128% respectively of a plate girder without longitudinal stiffener. Recommendations are proposed for the location of a longitudinal stiffener for this case, 0.5D. Therefore, in high shear-low moment conditions, longitudinal stiffeners can be used to increase girder strength to resist shear by placing it at the mid-depth of the web. The contribution of a web longitudinal stiffener to increase the strength of the plate girder depends upon the aspect ratio. For a plate girder aspect ratio value of (a/D) 1 and 2 units, the contribution of longitudinal stiffener placed at the optimal location is 2.5 and 2 times a plate girder without longitudinal stiffener respectively.*

**Keywords***: Buckling load, local buckling, longitudinal web stiffener, plate girder, Abacus.*

# ACKNOWLEDGEMENT

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I cannot forget my friends Tekleweyni and Tsegay instructors of Adigrat University who never failed to help me in my computing needs. Without their assistance, my thesis, which was almost exclusively done with the use of computers, would not have been successful.

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# ACRONYM

ERA- Ethiopian Road Authority

FEA- Finite Element Analysis

AASHTO LRFD 2012- American Association of State Highway and Transportation Officials Load Resistance Factor Design 2012

# LIST OF SYMBOLS

| | |
|---|---|
| a | Length of a web panel |
| an | Deflection coefficient of plate |
| d | Stiffener distance from the bottom of compression edge |
| bs | Width of the stiffener |
| D | Depth of the web panel |
| E | Young’s modulus of elasticity |
| m | Number of half-waves in the longitudinal x-direction |
| n | Number of half-waves in the transverse y-direction |
| ts | Thickness of stiffener |
| tf | Thickness of the flange plate |
| tw | Thickness of the web plate |
| T | Total work done by external forces |
| U | Total strain energy |
| w | Deflection of plate |
| L | Span length |
| T | Total work done by external forces |
| U | Total strain energy |
| w | Deflection of plate |
| L | Span length |

# CHAPTER ONE

# INTRODUCTION

## 1.1 Background of the Study

Plate girders are frequently used in the design of steel and composite bridges due to their good flexural strength, which makes them useful for long-span bridges. In the design of plate girders, the tendency is to arrange the material as much as possible in the extreme fibers. By keeping the web area as small as possible, the lever arm of the internal forces is maximized and the carrying capacity. But the web buckles, and this is a clear limit to the tendency towards optimum utilization of the material.

**Figure 1-1 plate girder**

Previous studies conclude that when a slender structure is loaded in compression, for small loads it deforms with hardly any noticeable change in geometry and load-carrying ability. On reaching a critical load value, the structure suddenly experiences a large deformation and it may lose its ability to carry the load. At this stage, the structure is considered to have buckled. For example, when a rod is subjected to an axially compressive force, it first shortens slightly but at a critical load the rod bows out, and we say that the rod has buckled. In the case of a thin circular ring under radial pressure, the ring decreases in size slightly before buckling into a number of circumferential waves. For a cruciform column under axial compression, it shortens and then buckles in torsion.

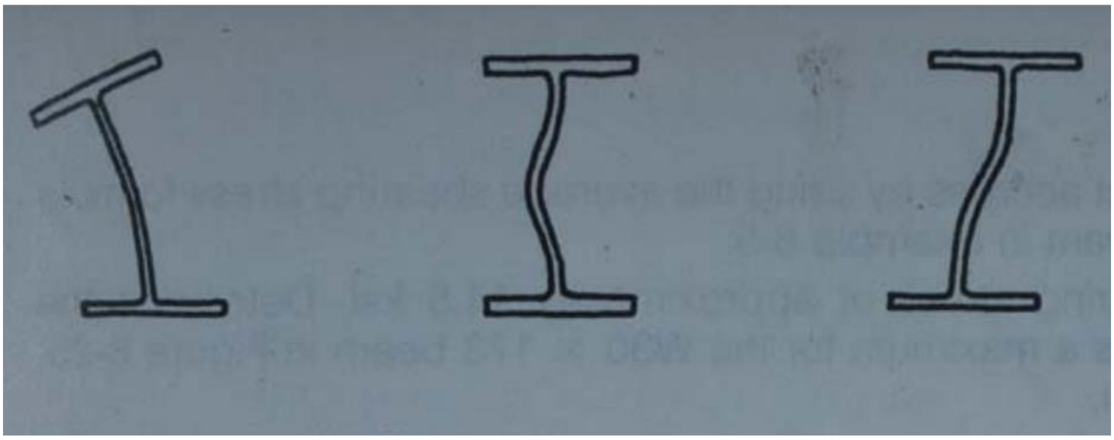


**Figure 1-2 Web buckling**

Several investigations have also been devoted to study the contribution of longitudinal stiffening on the ultimate strength of plate girders subjected to different loading(1).

In plate girders subjected to moving loads, the web plates between transverse stiffeners are prone to buckle locally due to direct compressive stresses, arising from in-plane bending moments and shear.

In most fabricated girders, webs are slender and tend to buckle locally prior to flexural–torsional, distortional buckling, or yielding. In most cases, elastic buckling does not represent a true strength limit state, since the webs exhibit significant post-buckling reserves of strength(2). In spite of this reserve, intermittent buckling under live loads, commonly known as plate breathing, gives way to fatigue cracks in regions where tension fields and folds are anchored on to flanges and transverse stiffeners; which in turn degrades the strength of plates and causes premature failure. Fatigue cracks are usually due to the secondary bending stresses induced by out-of-plane deflections of the web(3). This experience forces engineers to use longitudinal stiffeners in order to prevent web buckling. Longitudinal stiffeners are primarily added to obtain a higher local buckling capacity. A properly designed and efficient stiffener should remain intact and enforce a nodal line at the stiffener-plate junction(1). On the other hand, transverse stiffeners are meant to perform a similar task under shear loading(4).

**Figure 1-3 Failure of plate girder due to web buckling**

## 1.2 Statement of the Problem

The basic principle of plate girder design was well stated by Basler and Thürlimann in 1963(5) when they indicated that in the design of plate girders, the best approach is to put the material as far from the neutral axis as possible and keep the web area as small as possible. This approach maximized the section's efficiency for strength. As a result of this principle, webs are often slender, making web buckling a potential controlling limit state.

Web buckling is more specifically defined as plate buckling, an excessive out of plane deflection of the plate (3). When the limit state of web buckling occurs before the limit state of yielding, stiffeners used to increase member strength. The transverse stiffener is necessary to increase shear strength and post-buckling strength and to eliminate web folding, while the longitudinal stiffeners are used to increase the web's resistance to initial buckling(6). The optimal location of the longitudinal stiffener to resist web local buckling under high shear low moment condition near support to increase the capacity of plate girder for Buckling load have to be investigated for the best performance in design of plate girder.

The purpose of this paper was to study the most efficient location longitudinal stiffeners in plate girders to provide the greatest increase in Buckling load.

## 1.3 Research Question

1. What is the maximum buckling load that can perform the plate girder?
2. What is the optimal location of a longitudinal stiffener for maximum buckling load?
3. What is the increase in strength of plate girder as a result of providing longitudinal stiffener?
4. How is the contribution of a longitudinal stiffener on the buckling load of plate girder?

## 1.4 Objectives of the Study

### 1.4.1 General Objective

- The general objective was investigating local buckling of plate girder with longitudinal web stiffener.

### 1.4.2 Specific Objective

- To determine the maximum Buckling Load.
- To determine the optimal location of a single longitudinal web stiffener.
- To determine the greatest increase in buckling strength.
- To determine the contribution of longitudinal stiffener on the buckling load.

## 1.5 Significance of the Study

When plate girders are to be used for long-span bridges due to good in their flexural strength, the webs become slender and the bridge may fail due to the web's local buckling. But the buckling of the web can be treated by providing longitudinal stiffener so that the strength of the plate girder will increase. Therefore; this study will help engineers how could the longitudinal stiffeners will be provided for the maximum performance of the plate girder.

After conducting this research, the Ethiopian road authority will be the one who will be the beneficiary. Besides, this paper help university students and researchers for further study and as a reference.

## 1.6 Scope and Limitation of the Study

This thesis explores a plate girder with a longitudinal web stiffener and will investigate the optimum horizontal stiffener location to minimize the buckling effect of the web on the carrying capacity. Parametric studies were conducted for a range of relative stiffener location, aspect ratio and slenderness ratio using ABAQUS, and sections were also examined under where high shear and low moment condition near to the support. The results will be then compared to the other.

## 1.7 Report Organization

This thesis report is presented in five chapters together with the references and appendices. The first chapter contains a general introduction about the study; it includes the problem background, problem statement, research questions, research objectives, scope of the study and limitation of the study. The Second chapter is a literature review and it reviews the different ways of developing the web local buckling, the definitions on the study area, reviewing standards (AASHTO LRFD 2012) and previously done works on longitudinal stiffeners. In the third chapter, the research methodology is presented; it includes the tasks done to arrive at the final results like the inputs required for the software, the cases considered, and the software validation. The fourth chapter includes results and discussion. The conclusions and recommendations are included in the final section of this chapter. References are listed after the of fifth chapter ends, and subsequently the appendices and image for the results are included.

# CHAPTER TWO

# REVIEW OF RELATED LITERATURE

## 2.1 Buckling of Unstiffened Plates

Plate girders are an assembly of plates. Web buckling can be considered as a plate buckling phenomenon where the particular boundary conditions caused by the restraint of flanges and stiffeners that intersect the web plate determine member strength.

In a simply supported plate, subjected to bending moment in its own plane, as in thin webs of plate girders, it is supposed that the buckled form consists of m half-waves in the x-direction. Therefore, the deflection of the plate (w) can be represented by the series given in(7)

$$w = \sin\frac{m\pi x}{a}\sum_{n=1}^{\infty} a_n \sin\frac{n\pi y}{d} \quad \text{------------------------------------------------------------- (1)}$$

where n is the number of half-waves across the web depth and, an is the deflection coefficient. The corresponding strain energy, which does not involve any knowledge of the load distribution is(7)

$$U = \frac{\pi^4 Dad}{8}\sum_{n=1}^{\infty} a_n^2\left(\frac{m^2}{a^2}+\frac{n^2}{d^2}\right)^2 \quad \text{-------------------------------------------------------------- (2)}$$

D represents the flexural rigidity of the plate. Then, if σ0 is the maximum normal stress at the plate edges, the total work done by the linearly varying bending stresses (σb) would be

$$T = \frac{1}{2}\sigma_0 t_w \int_0^d \int_0^a \left(1-\frac{2y}{d}\right)\left(\frac{\partial w}{\partial x}\right)^2 dx\,dy \quad \text{------------------------------------------------ (3)}$$

Where tw is the thickness of the web plate and $\sigma_b = \sigma_0(1-2y/d)$.

Equating U = T yields to an expression for evaluating critical edge stress (σ0) cr. The result for the buckling coefficient kb (in Eq. (4)) is about 23.9; which is six times greater than the case of plates under pure compression(7):

$$(\sigma_0)_{cr} = k_b \frac{\pi^2 E}{12(1-\upsilon^2)} \left(\frac{t_w}{d}\right)^2$$

-----------------------------------------------------------(4)

## 2.2 Web Buckling Resistance

It is observed that girders with largely spaced transverse stiffeners present a considerably different response from girders with closely spaced transverse stiffeners(8). Rolando observed the phenomenon related that occurs when transversally and longitudinally stiffened steel plate girders are subjected to patch loading. The failure mechanism differs considerably for the particular structural case of girders with largely spaced transverse stiffeners, which have been studied thoroughly in the last decades. Steel plate girders with closely spaced stiffeners are occasionally found in bridge design and for such cases, the current EN1993-1-5 rules underestimate the strength of the webs to transverse forces. Research work on girders with closely spaced transverse stiffeners is available but for such cases, the web plates are longitudinally unstiffened(9).

In unstiffened panels or panels with largely spaced transverse stiffeners, the failure mode of steel plate girders subjected to patch loading is related to web folding under the concentrated load. The ultimate load capacity primarily depends on the web strength as well as on the flange stiffness. The mechanical behavior of longitudinally unstiffened panels with largely spaced transverse stiffeners is well known. Most predictions found in structural codes are reliable and based upon simple yet accurate formulations derived from mechanical models.

In girders with closely spaced transverse stiffeners, the failure mode is associated with an intertwined mechanism of web folding and flange yielding. The ultimate load capacity primarily depends on the web strength and flange strength. For the particular case of closely spaced transverse stiffeners, research has been active rather

recently and guidelines do not provide an explicit theoretical treatment to those cases. The definition of the distance between transverse stiffeners as large or close is particularly crucial for the definition of the patch loading resistance. Largely spaced transverse stiffeners do not contribute to the resistance to patch loading whereas closely spaced elements allow a considerable redistribution of stresses within the loaded panels at high load levels.

According to AASHTO LRFD 2012, The nominal bend-buckling resistance webs without Longitudinal Stiffeners shall be taken as:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \text{-----------------------------------------------------------------------------(5)}$$

but not to exceed the smaller of Rh*Fyc and Fyw /0.7

in which:

k = bend-buckling coefficient

$$= \frac{9}{(D_c/D)^2} \text{--------------------------------------------------------------------------------(6)}$$

where:

Dc = depth of the web in compression in the elastic range (in.).

For composite sections, Dc shall be determined as specified in Article D6.3.1.

Rh = hybrid factor specified in Article 6.10.1.10.1(10).

Graciano and Johansson present a design procedure for the determination of the ultimate resistance of longitudinally stiffened girder webs to concentrated loads. The influence from the longitudinal stiffener is considered in the slenderness parameter, through the buckling coefficient kf. This procedure is harmonized with other design procedures currently used for describing buckling problems in steel structures. An expression is developed for the buckling coefficient based on finite element analysis. The interaction between the web plate with flanges and a longitudinal stiffener was considered in the analysis. The ultimate strength according to the design procedure presented herein and the results are compared with available experimental results. The interaction with bending is also investigated(11).

Graciano also discussed on “Patch loading resistance of slender plate girders with longitudinal stiffeners” and observed failure mode of several analyzed girders using different stiffener locations. From the plots, it is observed that the largest increase in resistance is attained when the stiffener is located at b1/hw= 0.15 (12).

In lieu of alternative rational analysis, the nominal bend-buckling resistance webs with Longitudinal Stiffeners may be determined as specified in Eq. (5), with the bend-buckling coefficient taken as follows:

If $\frac{d_s}{D_c} \geq 0.4$, then:

$$k = \frac{5.17}{(d_s/D)^2} \geq \frac{9}{(D_c/D)^2}$$ ---------------------------------------------------------------------(7)

If $\frac{d_s}{D_c} < 0.4$, then:

$$k = \frac{11.64}{\left(\frac{D_c - d_s}{D}\right)^2}$$ ---------------------------------------------------------------------------(8)

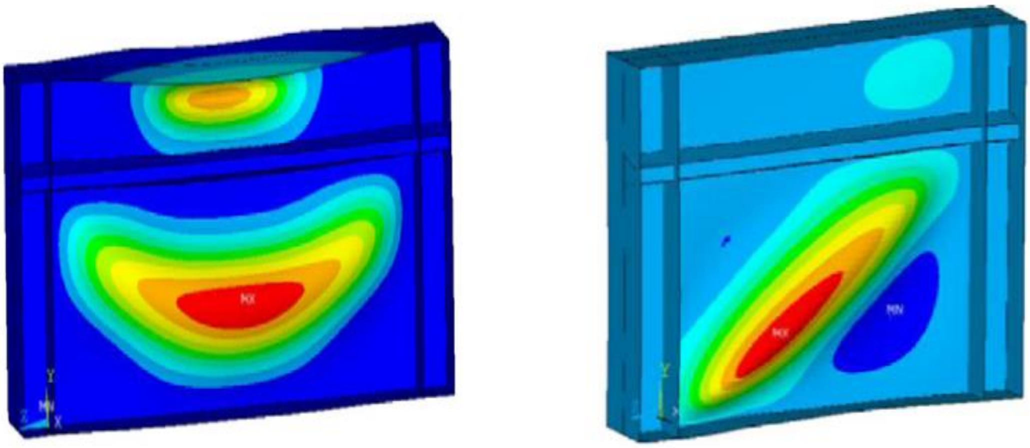
where:

ds = distance from the centerline of the closest plate longitudinal stiffener or from the gage line of the closest angle longitudinal stiffener to the inner surface or leg of the compression-flange element (in.) When both edges of the web are in compression, k shall be taken as 7.2(10).

In AASHTO 2012 LRFD, the web flexural strength is a function of k, a web bend buckling coefficient, k is defined by different equations for the two cases of unstiffened web or transversely and longitudinally stiffened web. But these equations allow the longitudinal stiffener to be at any location on the web(13). The AASHTO 2012 LRFD specifies that the location of the longitudinal stiffener should be where it enhances the performance of the girder.

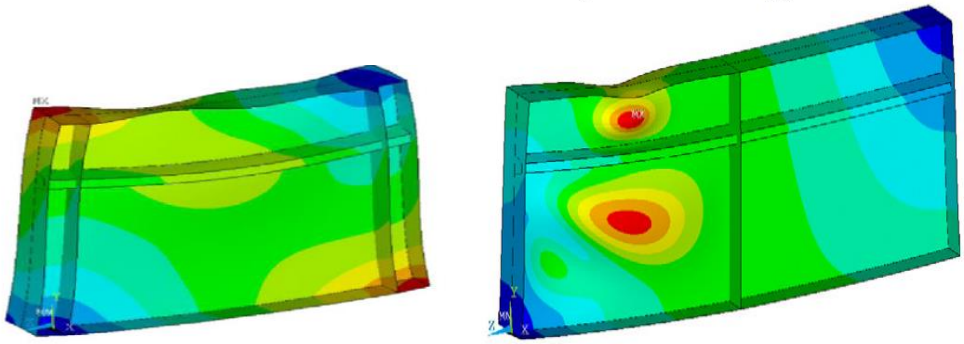
## 2.3 Web Buckling Failure Modes

Failure modes for web buckling are the following(14).



a) Patch loading failure mode

b) shear buckling failure mode



c) bending failure mode

d) combined failure mode

**Figure 2-1 Web buckling failure mode**

## 2.4 Stability of Longitudinally Stiffened Web Plates

The longitudinal stiffener should be placed in a judicious position so that it can influence the buckling mode and stresses to a considerable extent. The local buckling and post-local buckling performance of web plates in bending can be improved by the provision of a longitudinal stiffener parallel to the direction of the longitudinal stresses(14).

The shear strength of girder webs can be conservatively estimated by neglecting the contribution of longitudinal stiffeners if the location of the longitudinal stiffener is near the compression flange, and is not much helpful in increasing the shear strength. Longitudinal stiffeners increase the patch loading resistance and ductility of plate girder webs during the incremental launching of bridge girders(15).

The resistance of longitudinal stiffened steel I-girders subjected to patch loading, considering the influence of the slenderness ratio of the directly loaded panel (b1/tw). Patch loading resistance can be significantly enhanced by placing a longitudinal stiffener close to the loaded flange, especially for thin webs. A stiffener placed at this optimal location is effective to increase patch loading resistance for thin webs. Thicker webs without reinforcement exhibit a better performance for patch loading(16).

It is also developed a parametric investigation on plates with a longitudinal stiffener subjected to axial force, in-plane bending moment and shear loading with the aim of giving some new practical insights for the estimation of the buckling coefficient, taking into account (a) dimensions and shape (square and rectangular) of the plate, (b) dimensions and shape of the stiffener having the same area of the cross-section, Regarding the analysis of panels subjected to axial force, in-plane bending moment and shear a stiffener with a closed-section (rectangular, triangular, trapezoidal), due to its torsional rigidity, has a better performance than a stiffener with an open section and the same area of the cross-section(17). Regarding both panels subjected to axial force and in-plane bending moment and shear, stiffeners with small flexural rigidity are not able to allow the entire plate to behave as two separate subpanels in relation to local buckling since the stiffener with small flexural rigidity follows the out-of-plane deformation of the entire panel. On the other hand, it is also performed a numerical investigation for these stiffened girder sections and showed that the application of a longitudinal stiffener to the web would not increase the lateral-torsional buckling (LTB) strength(18). M.M. Alinia discussed Longitudinal stiffeners in web panels utilized to delay buckling. The results showed that it is actually dependent upon the flexural rigidity of stiffeners and panel aspect ratios(1). Graciano was observed that the critical buckling load increases considerably due to the presence of a longitudinal stiffener (19).

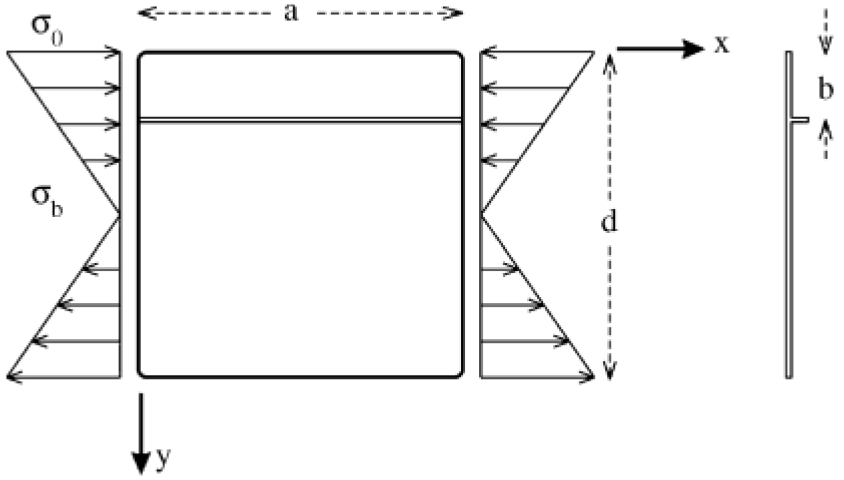


**Figure 2-2 Longitudinally stiffened plate**

## 2.5 Slenderness of Compression Flange

The AASHTO 2012 LRFD gives some indication on the dimensions of the compression flange of a plate girder(10):

bf/tf ≤ 12

bf >D/6

tf >=1.1tw

## 2.6 Slenderness of Unstiffened Web

For an unstiffened web, the AASHTO 2012 LRFD Specifications give the following web slenderness limits(10):

D/tw ≤ 150

where D = web height between flanges and tw = web thickness

## 2.7 Slenderness of Stiffened Web

For stiffened webs, the AASHTO 2012 LRFD Specifications limits the slenderness of the web to(10):

D/tw ≤ 300

## 2.8 Strength of Stiffeners

In order to have longitudinal stiffener maintaining a line of near-zero deflection in the web needed to prevent local web buckling, the 2012 LRFD Specifications limit the slenderness of the longitudinal stiffener. It is also stated that for the stiffeners to perform adequately, their yield stress should be the same as the flange's yield stress(10).

## 2.9 Projecting Width of Longitudinal Stiffener

The projecting width, b, of the stiffener shall satisfy:

$$b_\ell \leq 0.48 t_s \sqrt{\frac{E}{F_{ys}}}$$

where:

ts = thickness of the stiffener (in.)

## 2.10 Finite Element Analysis

The elements used in FE buckling analyses might comprise shells, beams or a mixture, on the understanding that plate local buckling effects cannot be identified using beam elements. The choice of elements should consider the shape function and mesh refinement should be checked. Elastic critical buckling loads may be obtained from Eigenvalue buckling analyses. The elastic critical forces, moments or stresses may be used together with codified buckling curves, in the determination of member resistances. Importantly, buckling modes are identified and may be visualized, potentially resulting in a better understanding of structural behavior than when calculations proceed "blind". Initial imperfections need to be included in any nonlinear buckling analysis.

Elastic critical buckling loads can also be used to investigate the global stability of a girder system and its susceptibility to second-order effects. While simple rules exist for girder systems meeting particular criteria, an FE Eigenvalue buckling analysis allows the consideration of non-identical, non-prismatic, non-symmetric girders, and those with skew and/ or plan curvature(20).

Extensive work has been carried out to determine expressions for critical buckling loads of flat unstiffened plates under shear, compression, bending and a combination of different loadings. Existing solutions are based on constant stress levels throughout the plate, and no theoretical solution or design rule exists for more complex situations. Therefore, finite element analysis is often used to solve more complicated cases(21).

# CHAPTER THREE

# RESEARCH METHODOLOGY

## 3.1 Research Design

The study of this research was devoted to local buckling of plate girder with a longitudinal web stiffener that determines the optimal location for a longitudinal stiffener. In order to study the influence of longitudinal stiffener, selected specimens of plate girders were analyzed by simulation analysis technique using computer software Abacus.

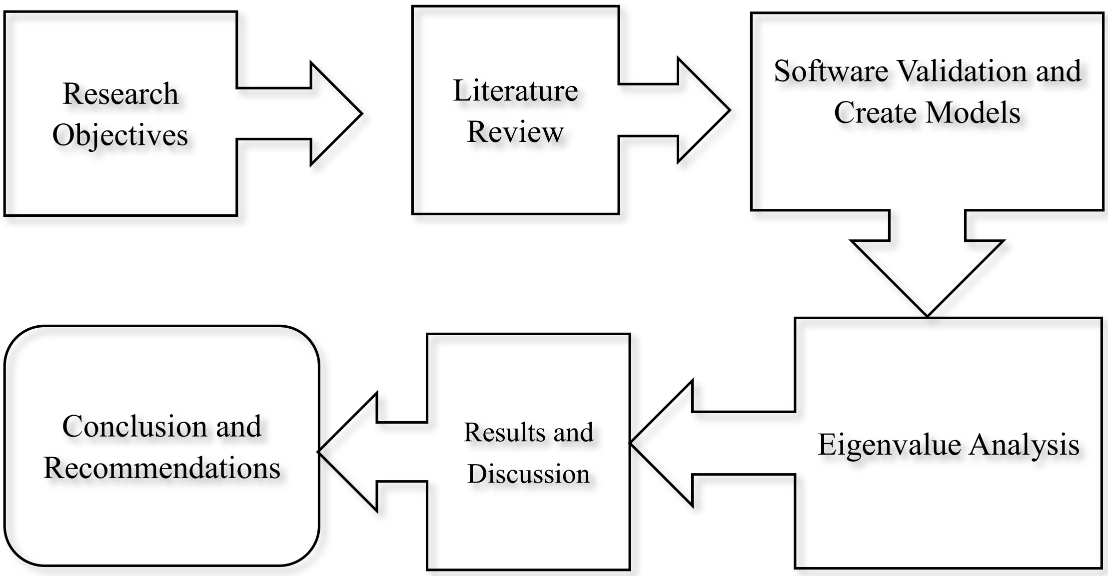


**Figure 3-1 chart showing the order of tasks of the methodology**

## 3.2 Study Variables

### 3.2.1 Dependent Variable

The dependent variable is: -

- Buckling load

### 3.2.2 Independent Variable

The independent variables are the following: -

- Location of longitudinal stiffener or relative stiffener location(d/D).
- Web slenderness ratio (D/tw)
- Aspect ratio (a/D)

## 3.3 Data Presentation and Analysis

This paper was done by using finite element analysis considering a model for plate girder with longitudinal web stiffener. For analytical studies of the behavior of structural elements, Finite Element Analysis (FEA) has given satisfactory results in predicting the behavior of the elements under various loads(22). There have been a number of FEA based studies of buckling girders and their results were comparable to those of tests and experiments. The most commonly used software for the analytical study of plate girder is ABAQUS(23,24). ABAQUS enables a wide range of Eigenvalue analysis simulations to be carried out efficiently, accurately and reliably. Moreover, linear buckling is simulated accurately in ABAQUS(25).

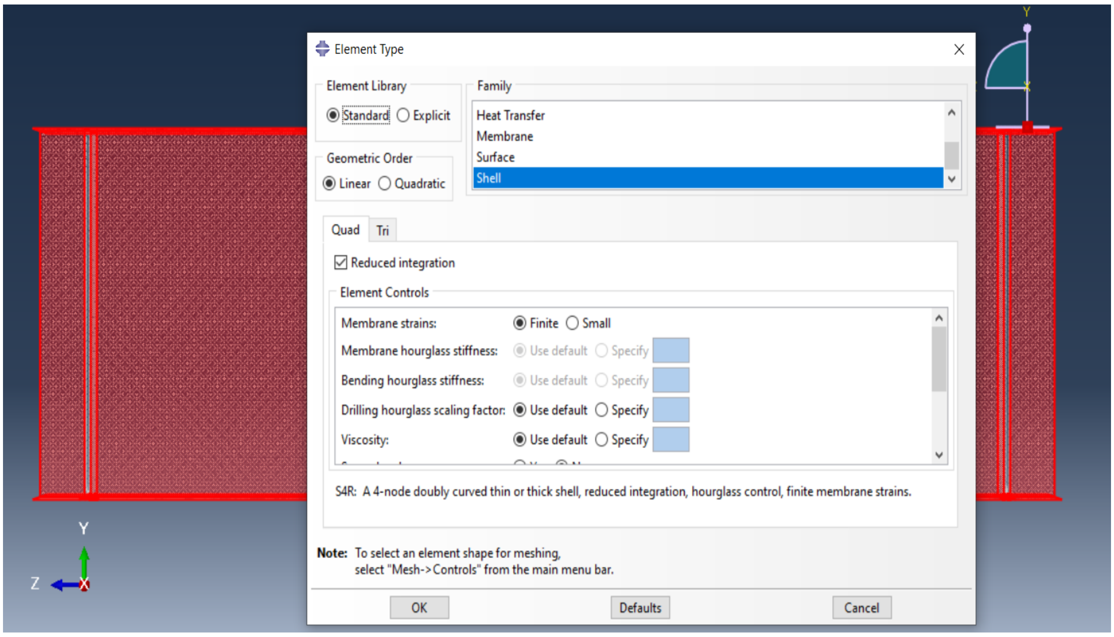
The general-purpose shell element type S4R was used to model all of the plate components of the girders (i.e. the web, the top and bottom flanges, and the stiffeners)(24).

### 3.4 Finite Element Model

The finite element program ABAQUS (v.6.14) is used for all the computational work. Recent research successfully used ABAQUS to investigate plate girders. The general-purpose shell element, S4R, is used to model plate girder components (i.e. top and bottom flanges and web) and the stiffeners (transverse and longitudinal). The S4R is a large strain shell element based on an exact geometric description of large rotation kinematics. The shell element assumptions are:

- The Plate is initially flat,
- Material is homogeneous and isotropic,
- Plane sections remain plane,
- Straight lines normal to the mid-surface remain straight after deformation,
- Straight lines normal to the mid-surface remain normal to the mid-surface after deformation,
- Thickness of the plate does not change during deformation.

The shell element uses five integration points and Simpson integration in the thickness direction, with one integration point per element. A similar shell element, S4, uses four 23 integration nodes per element and could have been selected for the current study. However, for web buckling it has been shown that the S4R element gives accurate results; hence the use of this more efficient reduced integration element is appropriate. This study investigates the following longitudinal stiffener locations: 0.1D, 0.2D, 0.3D, 0.4D, 0.5D, 0.6D. see Figure 3-1.


Element Type

Element Library: ● Standard ○ Explicit

Geometric Order: ● Linear ○ Quadratic

Family: Heat Transfer, Membrane, Surface, Shell (selected)

Quad | Tri

☑ Reduced integration

Element Controls

| | |
|---|---|
| Membrane strains: | ● Finite ○ Small |
| Membrane hourglass stiffness: | ● Use default ○ Specify |
| Bending hourglass stiffness: | ● Use default ○ Specify |
| Drilling hourglass scaling factor: | ● Use default ○ Specify |
| Viscosity: | ● Use default ○ Specify |

S4R: A 4-node doubly curved thin or thick shell, reduced integration, hourglass control, finite membrane strains.

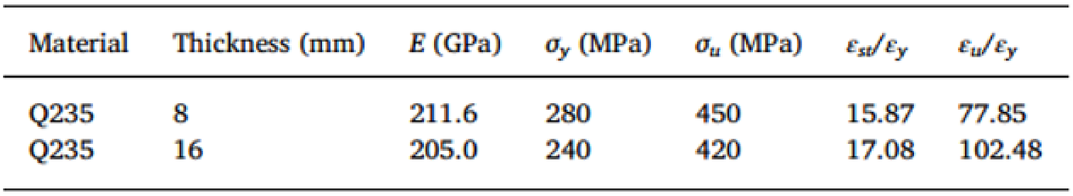
**Note:** To select an element shape for meshing, select "Mesh->Controls" from the main menu bar.

**Figure 3-2 Element Type**

## 3.5 Steel Constitutive Relationship

An elastic-plastic constitutive relationship is used to model the steel (see Figure 3-2). A tri-linear model is used to represent the steel. Young’s modulus for flanges and stiffeners is equal to 205 GPa, for the web is equal to 211.6 GPa and Poisson’s ratio is equal to 0.3. The yield strength and ultimate stress are also as the following table.

**Table 3-1 Primary Mechanical properties of the plates**



| Material | Thickness (mm) | *E* (GPa) | $\sigma_y$ (MPa) | $\sigma_u$ (MPa) | $\varepsilon_{st}/\varepsilon_y$ | $\varepsilon_u/\varepsilon_y$ |
|---|---|---|---|---|---|---|
| Q235 | 8 | 211.6 | 280 | 450 | 15.87 | 77.85 |
| Q235 | 16 | 205.0 | 240 | 420 | 17.08 | 102.48 |

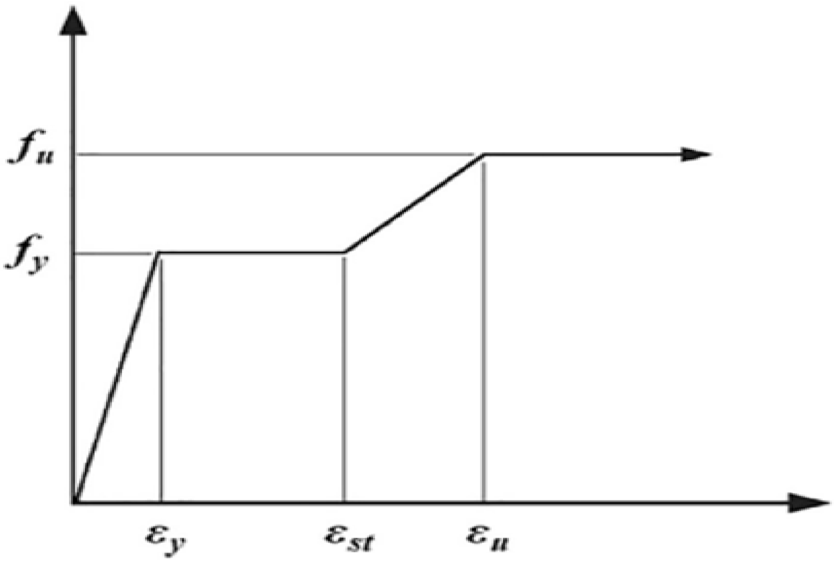
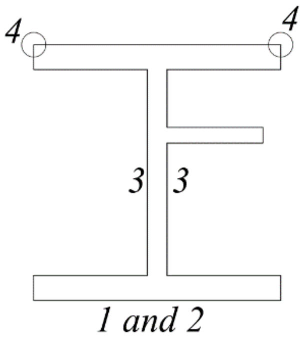


**Figure 3-3 Simplified constitutive model of material**

### 3.6 Boundary Conditions

This research addresses simply supported beams. Supports are idealized as follows (see Figure below):

**Table 3-2 boundary conditions**



| | Ux | Uy | Uz | URx | URy | URz |
|---|---|---|---|---|---|---|
| 1 | 0 | 0 | 0 | 1 | 1 | 0 |
| 2 | 0 | 0 | 0 | 1 | 1 | 0 |
| 3 | 0 | 1 | 1 | 1 | 1 | 0 |
| 4 | 1 | 1 | 1 | 1 | 1 | 1 |

0 – restrained, 1- Free

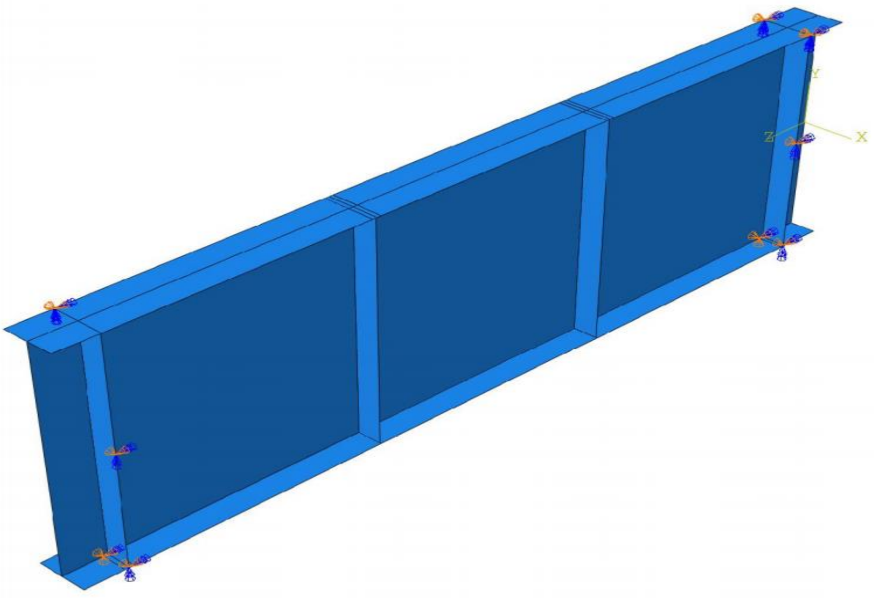
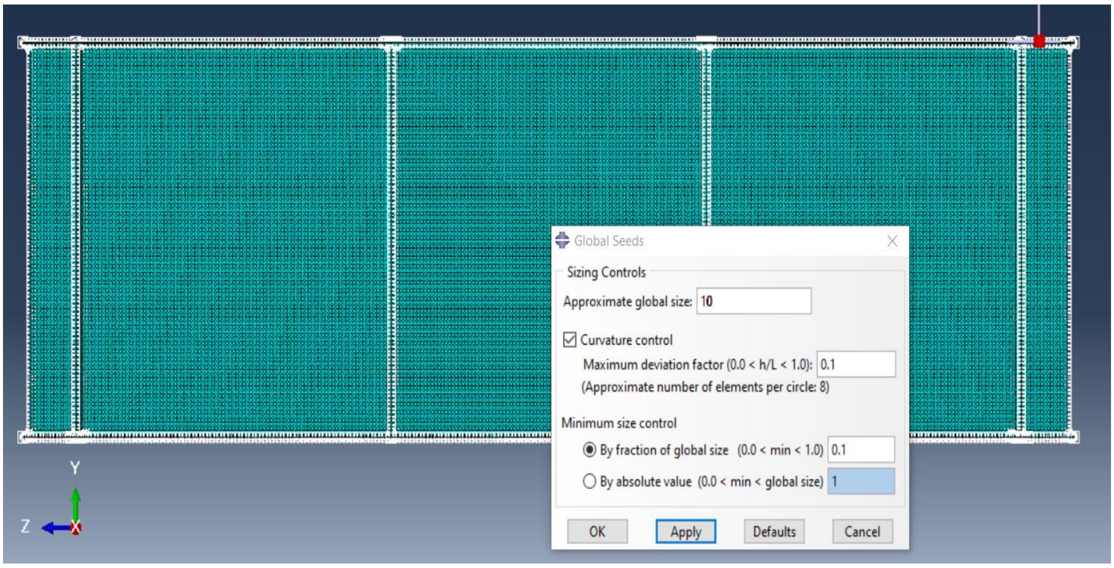


**Figure 3-4 Boundary conditions**

## 3.7 Convergence Study

To select a suitable mesh element size, a convergence test was carried out. The studied plate girder is G3 as shown in Table 3.3.



Global Seeds

Sizing Controls

Approximate global size: 10

☑ Curvature control

Maximum deviation factor (0.0 < h/L < 1.0): 0.1

(Approximate number of elements per circle: 8)

Minimum size control

● By fraction of global size (0.0 < min < 1.0) 0.1

○ By absolute value (0.0 < min < global size) 1

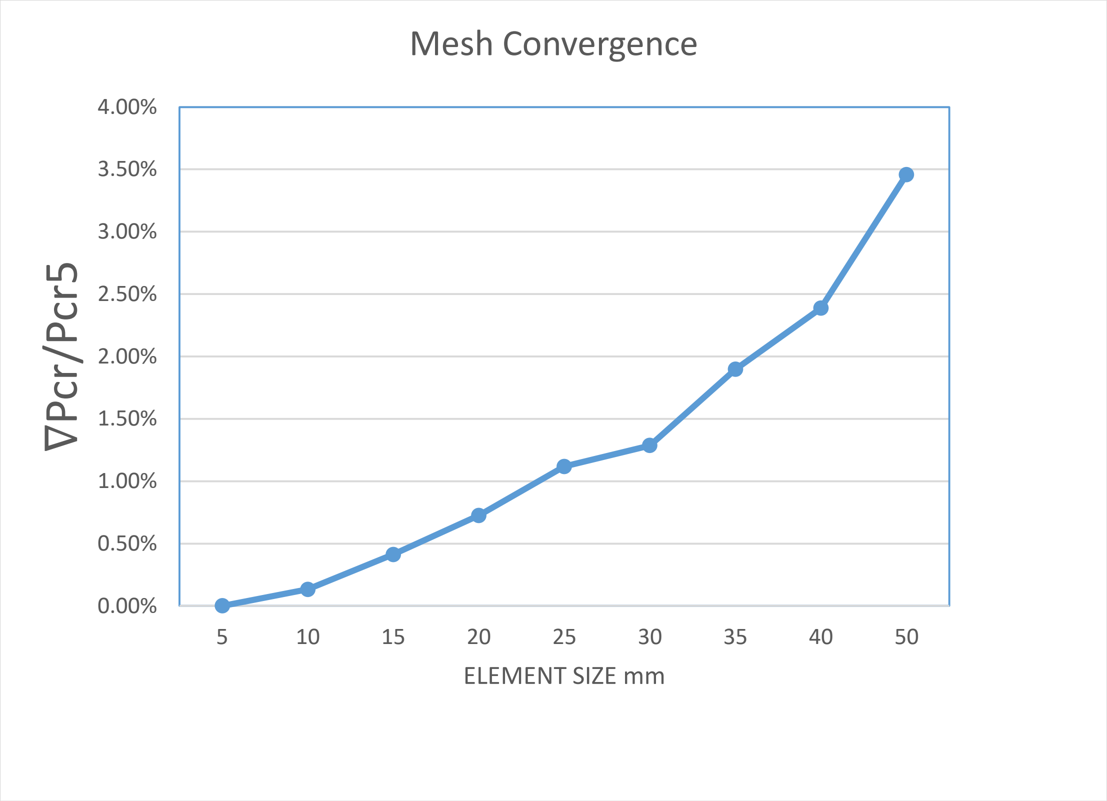


**Figure 3-5 Mesh Convergence**

Based on the fact that the shape of the shell element S4R is a square, the length of the square is selected to quantify the size of the shell element. Different mesh element sizes are considered, including 5 mm, 10 mm, 20 mm, 25mm, 30mm, 35mm, 40 mm, 45 mm and 50 mm. The FEA results (Eigenvalue) for the different mesh sizes are considered, where ∇Pcr=Pcri−Pcr 5 and i=5, 10, 20, 25, 30, 35, 40, 45 and 50. Based on Fig 3.4, the error for the element with a size of 10 mm was less than 0.13%, and was suitable. See Figure 3.5.

## 3.8 Model Validation

### 3.8.1 Overview

Experimental tests were conducted for local buckling classification plate girders(26). Test results from this research are used to validate the finite element models in this study.

### 3.8.2 Experiments Set-up

Specimens were designed and tested as a simply supported beam with two concentrated loads at 1/3 and 2/3 points of the span, and bearing stiffeners were located directly under the point of loading, as shown in Fig. 3.5. Each specimen is symmetric about mid-span with a test panel on one side. The plate girders were placed on pin and roller support, under static loads with different cross-section dimensions and length of the plate girder. From those tests, where a flange of compact section and non-compact web section (G3) was selected for validation. Torsional and lateral constraints were provided at the end supports. In the test, the bottom flange displacement at the mid-span was investigated. Dimensions of the girders of this experiment are given in Table 3-3 (all dimensions are in mm). D is the clear web depth, a is web panels length under investigation, tw and tf thickness of the web and the flanges respectively, and L the span length of girder.

**Table 3-3 Design properties of specimens.**

| Label | Section | Span-Length | bf/2tf | hw/tw | Lb/h | a/h | Section class |
|---|---|---|---|---|---|---|---|
| G1 | A | A | 5.625 | 71 | 4.5 | 1.5 | 2 |
| G2 | A | B | 5.625 | 71 | 6 | 2 | 2 |
| G3 | B | A | 5.625 | 103.5 | 3.14 | 1.05 | 3 |
| G4 | B | B | 5.625 | 103.5 | 4.19 | 1.40 | 3 |

### 3.8.3 Analysis and Results

A linear perturbation, buckling mode analysis available in ABAQUS was used to determine the buckling load. The buckling load, or the critical load, of a system can be obtained from an eigenvalue analysis. The first positive eigenvalue is compared to maximum loads reported from the experiments. Two failure mode buckling shape figures from the experiments were available. Figure 3-5 and Figure 3-6 show the failure mode observed during the experiments and the finite element analysis. It is clear that the failure mode predicted by the FEA is the same one that occurred during the experiments. The results indicate that the ABAQUS models can accurately predict the buckling load when a linear perturbation approach in ABAQUS is used.

**Figure 3-6 Experiment set up**

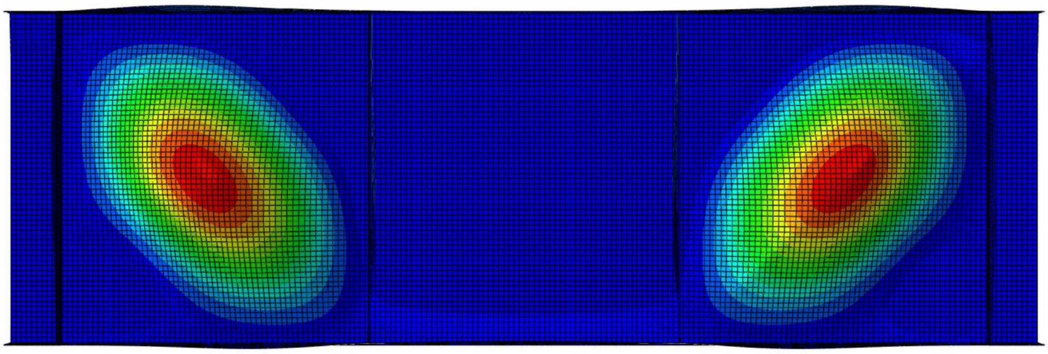


**Figure 3-7 Failure mode from FEA**

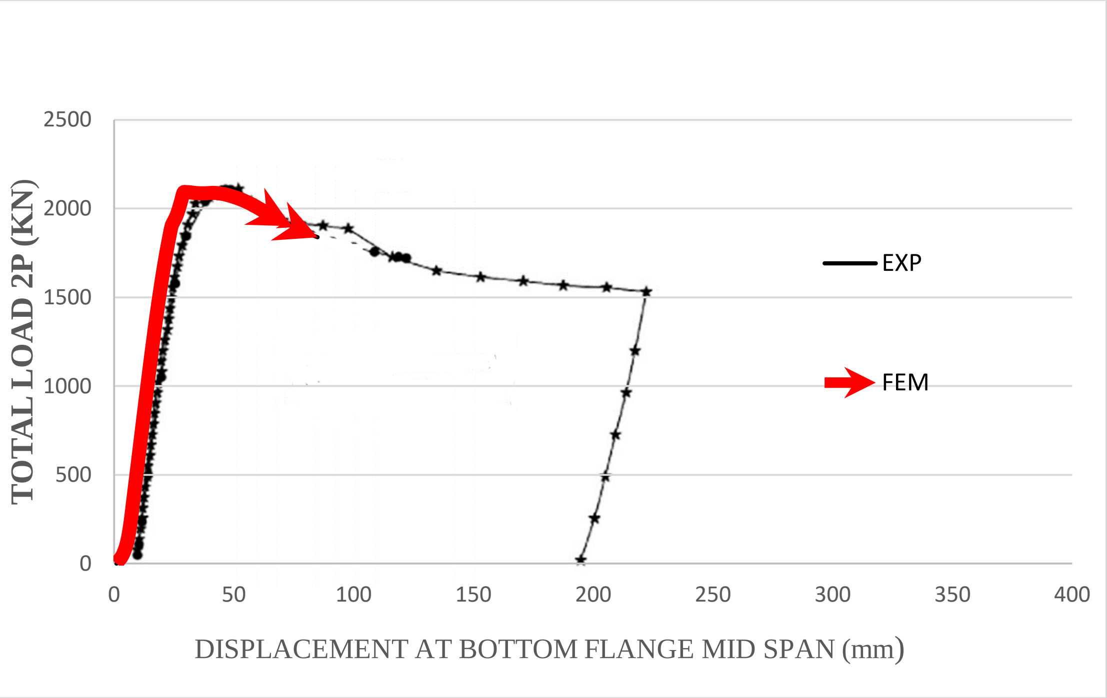


**Figure 3-8 Load Vs Displacement**

# CHAPTER FOUR

# RESULTS AND DISCUSSION

## 4.1 Parametric Studies

### 4.1.1 Parameters

Prior to initiating an experimental design matrix, relevant parameters that completely describe simply supported, single-span plate girder can be presented as follows:

- ✓ bft / bfb: Ratio of the width of the top flange to that of the bottom flange.
- ✓ tw: thickness of the web (or slenderness of the web: D/tw).
- ✓ tf: thickness of the compression flange (or slenderness of the flange: bf/tf).
- ✓ a: distance between 2 consecutive transverse stiffeners (or the panel aspect ratio a/D).
- ✓ depth of longitudinal stiffener with respect to depth of the web.

As stated previously this study is limited to:

- ✓ Doubly symmetric I-shaped, plate girders: bft = bfb, and

Therefore, the parameters that can vary and were the focal point of the studies were limited to a/D, D/tw, and d/D. All relevant parameters are shown on a representative plate girder section and elevation in Figure below.

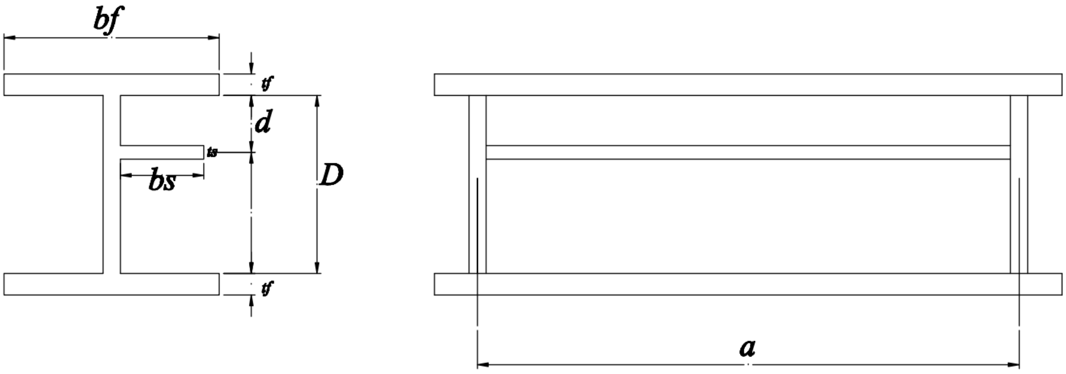


**Figure 4-1 A typical plate girder section and elevation**

## 4.2 Combination of Parameter

The parameters are selected based on the classification of sections on local buckling. For the case of local buckling, the slenderness is based on the width/thickness ratios of the slender plate elements. The member cross-sections are then classified by which of the ranges their slenderest element falls in. The flanges are compact and the webs are slender so that we can compare the influence of longitudinal stiffener on the strength of plate girder. Therefore; the values of the parameters are selected by fulfilling the requirements of making the girder fail through local buckling. Having defined ranges for all parameters, the resulting general matrix for all possible combinations is shown in Table 4-1.

The girders are divided into two sets based on web proportioning, and each set is divided into groups based on panel aspect ratio. In the first set the web slenderness, D/tw, is 150 and in the second it is 300, where D is the clear height of the web between the flanges, and tw is the web thickness. In all groups, this study used different longitudinal stiffener location to web depth ratios d/D, 0.1, 0.2, 0.3, 0.4, 0.5 and 0.6. The span length is also varied to consider if it has an effect. The same system is followed in the second set the web slenderness is 150. Finally, this gives a total of 21 specimens.

**Table 4-1 Combination of parameters**

| *Mod* | *D/tw* | *a/D* | *d/D* | *a* | *D* | *tw* | *bf* | *tf* | *d* | *bs* | *ts* | *L* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 150 | 1 | 0.00 | 1200 | 1200 | 8 | 240 | 16 | 0 | 120.00 | 16 | 3,600 |
| 2 | | | 0.10 | 1200 | 1200 | 8 | 240 | 16 | 120 | 120.00 | 16 | |
| 3 | | | 0.20 | 1200 | 1200 | 8 | 240 | 16 | 240 | 120.00 | 16 | |
| 4 | | | 0.30 | 1200 | 1200 | 8 | 240 | 16 | 360 | 120.00 | 16 | |
| 5 | | | 0.40 | 1200 | 1200 | 8 | 240 | 16 | 480 | 120.00 | 16 | |
| 6 | | | 0.50 | 1200 | 1200 | 8 | 240 | 16 | 600 | 120.00 | 16 | |
| 7 | | | 0.60 | 1200 | 1200 | 8 | 240 | 16 | 720 | 120.00 | 16 | |
| 8 | | 2 | 0.00 | 2400 | 1200 | 8 | 240 | 16 | 0 | 120.00 | 16 | 7,200 |
| 9 | | | 0.10 | 2400 | 1200 | 8 | 240 | 16 | 120 | 120.00 | 16 | |
| 10 | | | 0.20 | 2400 | 1200 | 8 | 240 | 16 | 240 | 120.00 | 16 | |
| 11 | | | 0.30 | 2400 | 1200 | 8 | 240 | 16 | 360 | 120.00 | 16 | |
| 12 | | | 0.40 | 2400 | 1200 | 8 | 240 | 16 | 480 | 120.00 | 16 | |
| 13 | | | 0.50 | 2400 | 1200 | 8 | 240 | 16 | 600 | 120.00 | 16 | |
| 14 | | | 0.60 | 2400 | 1200 | 8 | 240 | 16 | 720 | 120.00 | 16 | |
| 15 | 300 | 1 | 0.00 | 1920 | 1920 | 8 | 340 | 16 | 0 | 170.00 | 16 | 5,760 |
| 16 | | | 0.10 | 1920 | 1920 | 8 | 340 | 16 | 192 | 170.00 | 16 | |
| 17 | | | 0.20 | 1920 | 1920 | 8 | 340 | 16 | 384 | 170.00 | 16 | |
| 18 | | | 0.30 | 1920 | 1920 | 8 | 340 | 16 | 576 | 170.00 | 16 | |
| 19 | | | 0.40 | 1920 | 1920 | 8 | 340 | 16 | 768 | 170.00 | 16 | |
| 20 | | | 0.50 | 1920 | 1920 | 8 | 340 | 16 | 960 | 170.00 | 16 | |
| 21 | | | 0.60 | 1920 | 1920 | 8 | 340 | 16 | 1152 | 170.00 | 16 | |

## 4.3 Loading Pattern

The loading configuration discussed herein considers high shear -low moment near support. The loading configuration is shown in Figure 4-1. The girder is simply supported at location 1L and 1R. It is loaded with 2 equal concentrated loads, P, at third points of the span (locations 2L and 2R), thus dividing the girder into 3 equal arc lengths. The girder is braced to prevent lateral displacements of the top and bottom flanges at the end. The critical panel is the panel near support.

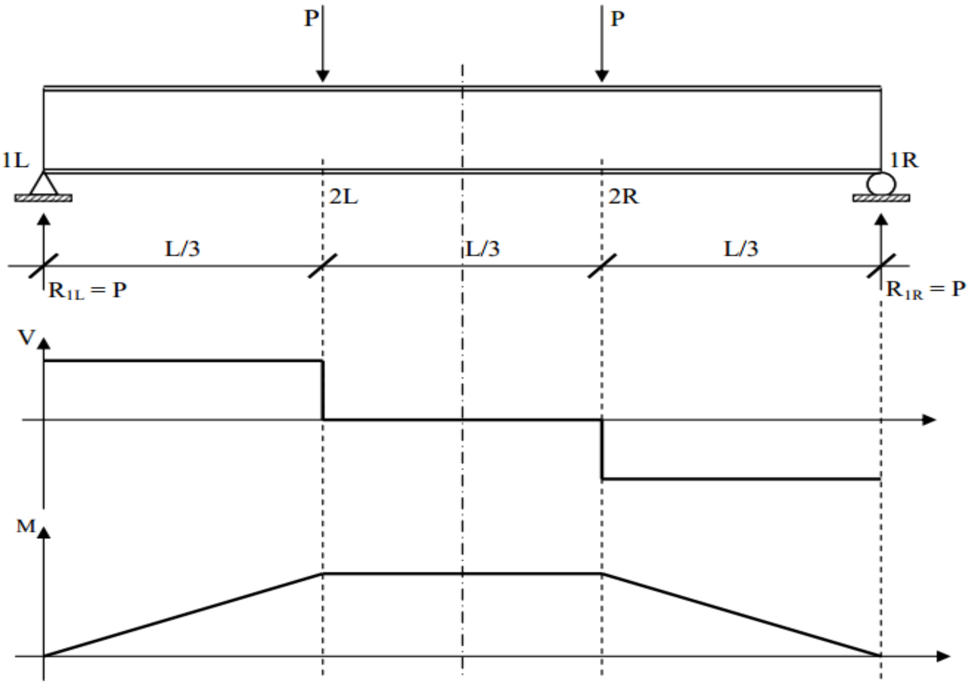


**Figure 4-2 loading pattern**

## 4.4 Results and Discussion

The girders studied were divided into three groups. Each group is composed of seven girders. The parameter that changes for each girder within the group is the distance of longitudinal stiffener with respect to depth of the web (d/D).

A model for each combination of parameters is built, and seven cases are analyzed with ABAQUS: the first run is without longitudinal stiffeners, the second is with the longitudinal stiffener located at 0.1D from the bottom of the compression flange, the top flange, the third run is with the longitudinal stiffener located at 0.2D, the fourth is with the longitudinal stiffener at 0.3D, the fifth at 0.4D, the sixth at 0.5D and seventh at 0.6D. Material properties from the experiment were used for the analysis with S4R element type and 10mm mesh size.

After the runs, all the positive eigenvalues are examined since only positive eigenvalues mean that compression is in the upper part of the web and buckling occurred in the compression zone. The optimal location of the longitudinal stiffener for any given girder yields the highest critical load, Pcr, for all six longitudinal stiffener cases. For a given location of the longitudinal stiffener, the web does not buckle in the critical panel for any eigenvalue or increase the buckling load capacity of the section for maximum, this location could be an optimal one for the girder under investigation. The percentage of increase of carrying load capacity for each location of the longitudinal stiffener is also to be determined.

After all, discussions for maximum buckling load determination, optimal longitudinal stiffener location, and percentage of increment were made.

**Web slenderness, D/tw = 150: Group 1 and 2**

**For aspect ratio, a/D= 1, Group 1: Girders G01, 02, 03, 04, 05, 06, 07**

In the first group, the eigenvalue (buckling load /critical load), called Pcr for each girder is given in Table 4-1 below. Values of Pcr are given for every location of the longitudinal stiffener. The zero value for the stiffener location denotes the case without a longitudinal stiffener. For the result, details see on appendix A.

**Table 4-2 Pcr for varying longitudinal stiffener locations: Group 1**

| GROUP 1 | D/tw = | 150 |
|---|---|---|
| | a/D= | 1 |
| d/D | | Pcr (kN) |
| - | | 658.90 |
| 0.10 | | 793.76 |
| 0.20 | | 911.00 |
| 0.30 | | 1,094.00 |
| 0.40 | | 1,383.00 |
| 0.50 | | 1,800.00 |
| 0.60 | | 1,374.00 |

This table shows the value of buckling load, for each location of the longitudinal stiffener for Group 1 (D/tw=150, a/D=1), with the other variables constant.

Here in the above Table 4-2, it can be seen that as the depth of longitudinal stiffener (d/D) increases from the bottom of top flange, the corresponding critical load/maximum buckling load that can carry the plate girder increases until the location of longitudinal stiffener will be, d/D=0.5 with maximum result of buckling load =1800. But the buckling load Pcr starts to decreases when the longitudinal stiffener is placed far away from the compression side i.e d/D > 0.5. Therefore; the optimal location of longitudinal stiffener for maximum buckling load for group 1 models is at 0.5D. (i.e. at mid-depth of the web).

A graphical representation for this group can be seen in Figure 4-3 below.

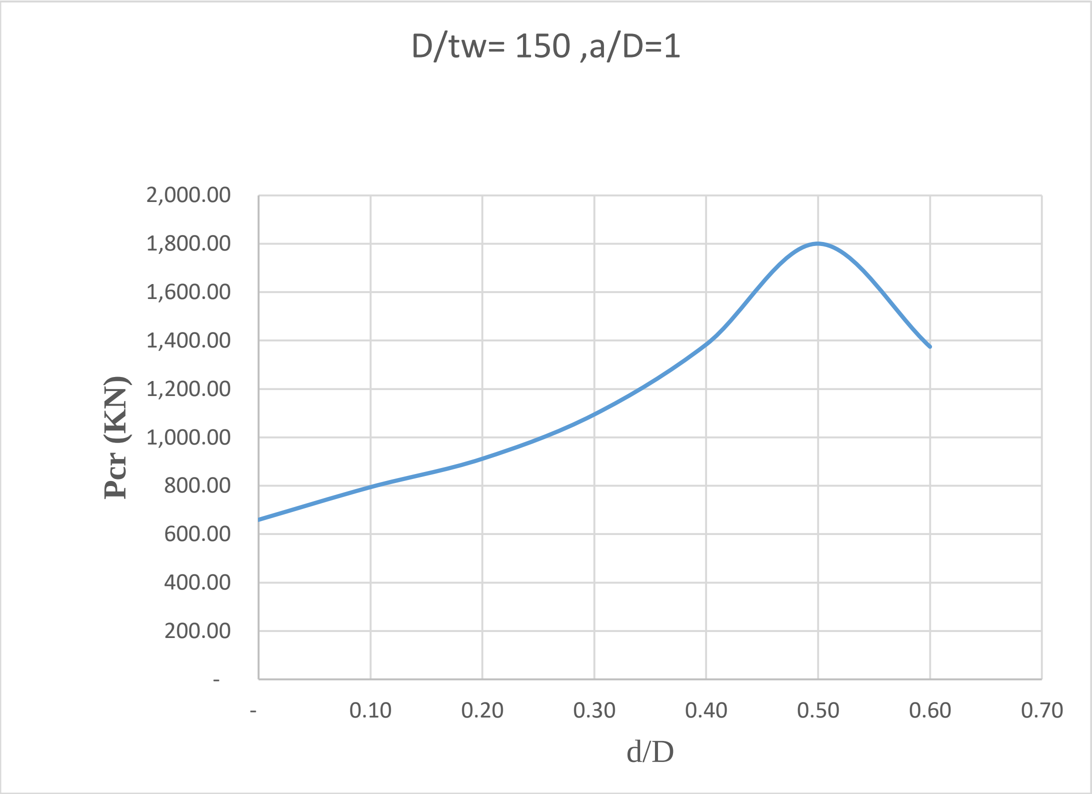


**Figure 4-3 Pcr (kN) value for Group 1**

The plots are grouped for the values of d/D. In the graph, the horizontal axis represents the location of the longitudinal stiffener measured from the bottom of the top flange, d/D and the vertical axis the critical load, Pcr, where d represent the longitudinal stiffener location.

The line shows the comparison of the buckling load results with corresponding longitudinal stiffener locations. From the results we can see at d=0.5D, the buckling load is maximum, Pcr=1800. This means that at 0.5D the buckling effect is minimized to maximize the carrying capacity of the plate girder.

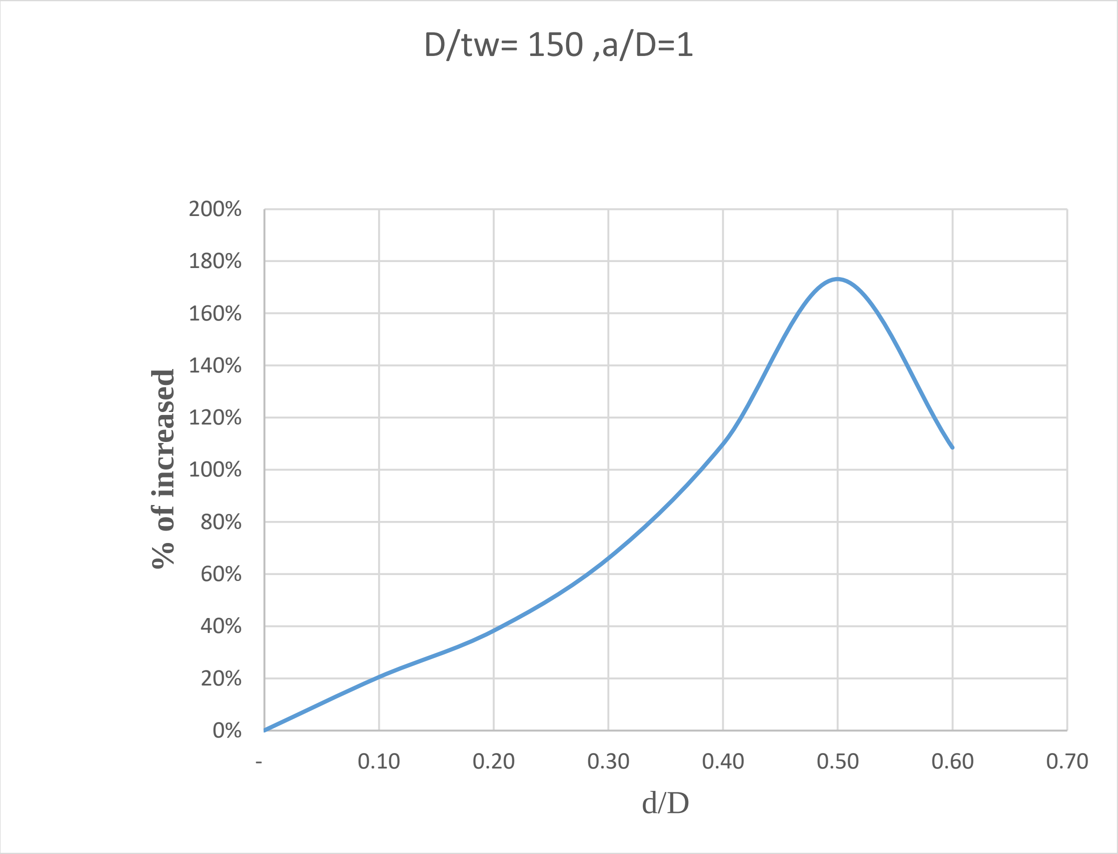


**Figure 4-4 percentage of increments**

The plots are for group 1 above and are grouped for the values of d/D. In the graph, the horizontal axis represents the location of the longitudinal stiffener measured from the bottom of the top flange, d/D and the vertical axis the percentage of increases as a result of providing longitudinal stiffener, where d represents the longitudinal stiffener location. Here the figure illustrates, the increased percentage of buckling load for each location of longitudinal stiffener. Therefore; the maximum increased percentage buckling load is 173% at d = 0.5D.

Overall, in this group, there is a maximum increasing buckling load of 1800kN and 173% is increased when the longitudinal stiffener is placed at 0.5D.

**For aspect ratio, a/D= 2, Group 2: Girders G08, 09, 10, 11, 12, 13 and 14**

In this group, the aspect ratio is 2 and the eigenvalue, called Pcr, for every case for each girder is given in Table 4-2. For each girder, values of Pcr are given for every location of the longitudinal stiffener. The zero value for the stiffener location denotes the case without a longitudinal stiffener.

**Table 4-3 Pcr for varying longitudinal stiffener locations: Group 2.**

|  | D/tw = | 150 |
|---|---|---|
| GROUP 2 | a/D= | 2 |
| d/D |  | Pcr (kN) |
| - |  | 385.74 |
| 0.10 |  | 503.55 |
| 0.20 |  | 607.57 |
| 0.30 |  | 736.07 |
| 0.40 |  | 849.44 |
| 0.50 |  | 879.99 |
| 0.60 |  | 830.03 |

This table shows the value of buckling load, for each location of longitudinal stiffener for Group 1 (D/tw=150, a/D=2), with the other variables constant. Here in the above Table 4-3, it can be seen that as the depth of longitudinal stiffener (d/D) increases from the bottom of top flange, the corresponding critical load/maximum buckling load that can carry the plate girder increases until the location of longitudinal stiffener will be, d/D=0.5 with maximum result of buckling load =879.9. But the buckling load Pcr starts to decreases when the longitudinal stiffener is placed far away from the compression side i.e d/D > 0.5. Therefore; the optimal location of longitudinal stiffener for maximum buckling load for group 2 models is at 0.5D. (i.e. at mid-depth of the web).

A graphical representation of Pcr for this group can be seen in Figure 4-2 as follow.

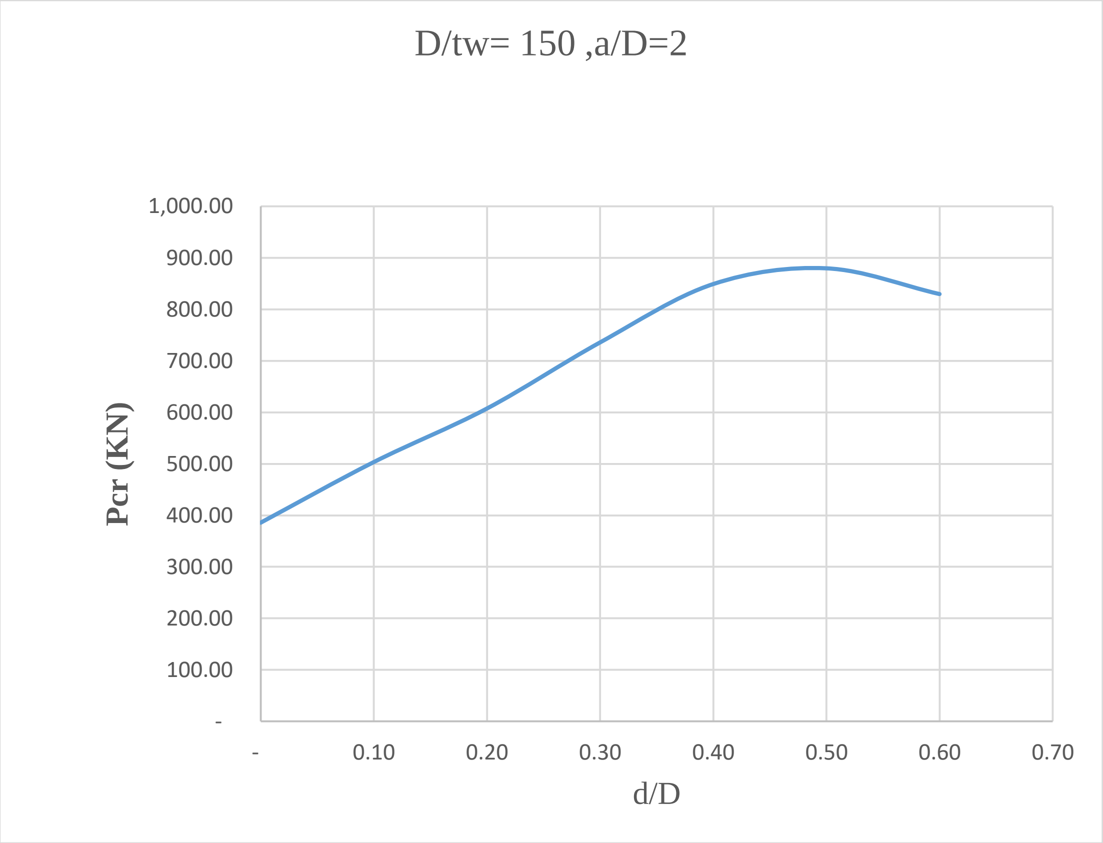


**Figure 4-5 Pcr (kN) value for Group 2**

The plots are grouped for the values of d/D. In the graph, the horizontal axis represents the location of the longitudinal stiffener measured from the bottom of the top flange, d/D and the vertical axis the critical load, Pcr, where d represent the longitudinal stiffener location.

The line shows the comparison of the buckling load results with corresponding longitudinal stiffener locations. From the results we can see at d=0.5D, the buckling load is maximum, Pcr=879.9. This means that at 0.5D the buckling effect is minimized to maximize the carrying capacity of the plate girder.

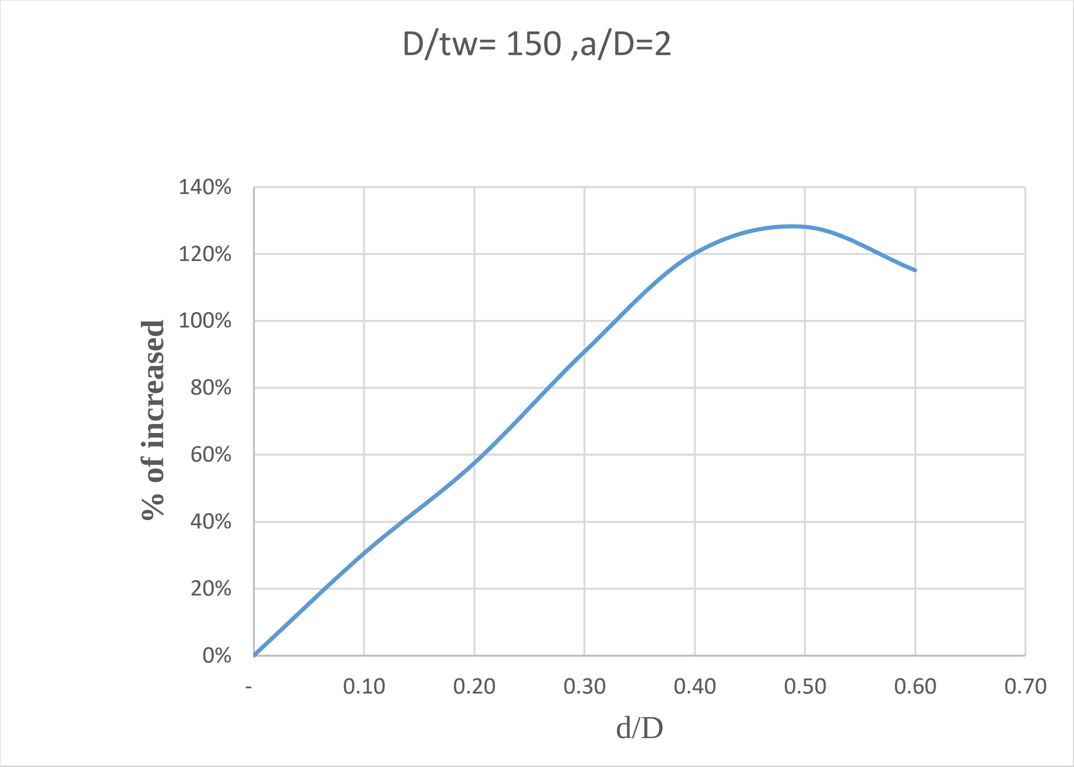


**Figure 4-6 percentage of increments**

The plots are for group 2 above and are grouped for the values of d/D. In the graph, the horizontal axis represents the location of the longitudinal stiffener measured from the bottom of the top flange, d/D and the vertical axis the percentage of increases as a result of providing longitudinal stiffener, where d represents the longitudinal stiffener location. Here the figure illustrates, the increased percentage of buckling load for each location of longitudinal stiffener. Therefore; the maximum increased percentage buckling load is 128% at d = 0.5D.

Overall, in this group, there is a maximum increasing buckling load of 879.9kN and 128% is increased when the longitudinal stiffener is placed at 0.5D.

**Web slenderness, D/tw = 300: Group 3**

**For aspect ratio, a/D= 1, Group 3:15, 16, 17, 18, 19, 20 and 21**

In this group, the eigenvalue, called Pcr, for every case for each girder is given in Table 4-4. Values of Pcr are given for every location of the longitudinal stiffener. The zero value for the stiffener location denotes the case without a longitudinal stiffener. For the result details see appendix A.

**Table 4-4 Pcr for varying longitudinal stiffener locations: Group 3**

| GROUP 3 | D/tw = | 300 |
|---|---|---|
| | a/D= | 1 |
| d/D | | Pcr (kN) |
| - | | 294.50 |
| 0.10 | | 341.70 |
| 0.20 | | 392.32 |
| 0.30 | | 471.18 |
| 0.40 | | 594.93 |
| 0.50 | | 771.20 |
| 0.60 | | 586.37 |

This table shows the value of buckling load, for each location of longitudinal stiffener for Group 3 (D/tw=300, a/D=1), with the other variables constant.

Here in the above Table 4-4, it can be seen that as the depth of longitudinal stiffener (d/D) increases from the bottom of top flange, the corresponding critical load/maximum buckling load that can carry the plate girder increases until the location of longitudinal stiffener will be, d/D=0.5 with maximum result of buckling load =771.2. But the buckling load Pcr starts to decreases when the longitudinal stiffener is placed far away from the compression side i.e d/D > 0.5. Therefore; the optimal location of longitudinal stiffener for maximum buckling load for group 3 models is at 0.5D. (i.e. at mid-depth of the web).

A graphical representation of Pcr for this group can be seen in Figure 4-3 as follow.

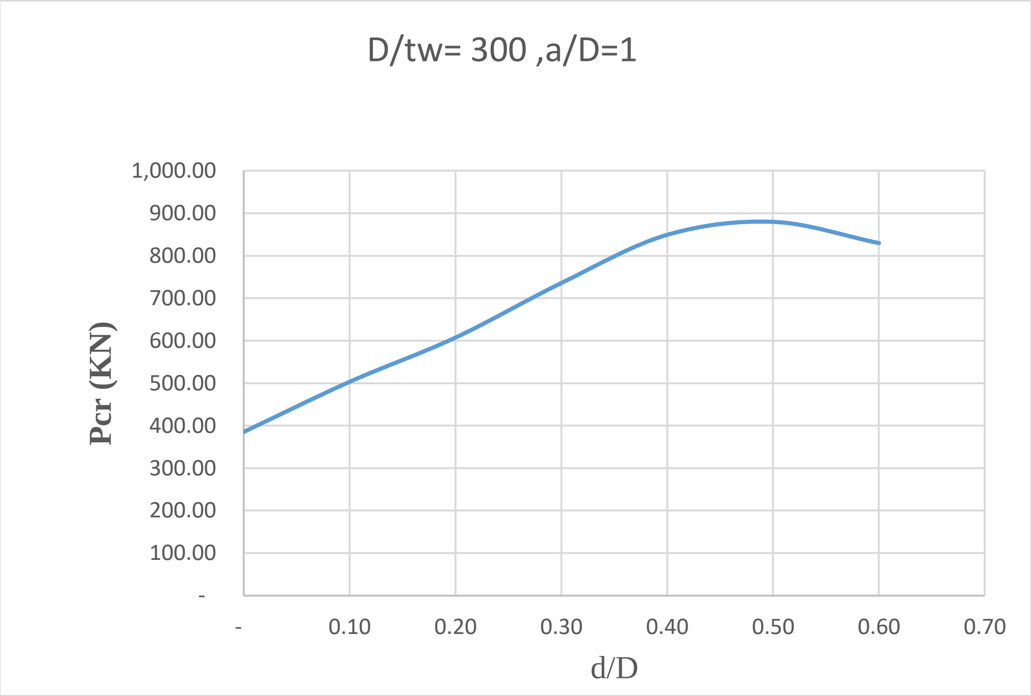


**Figure 4-7 Pcr (kN) value for Group 3**

The plots are grouped for the values of d/D. In the graph, the horizontal axis represents the location of the longitudinal stiffener measured from the bottom of the top flange, d/D and the vertical axis the critical load, Pcr, where d represents the longitudinal stiffener location.

The line shows the comparison of the buckling load results with corresponding longitudinal stiffener locations. From the results we can see at d=0.5D, the buckling load is maximum, Pcr=771.2. This means that at 0.5D the buckling effect is minimized to maximize the carrying capacity of the plate girder.

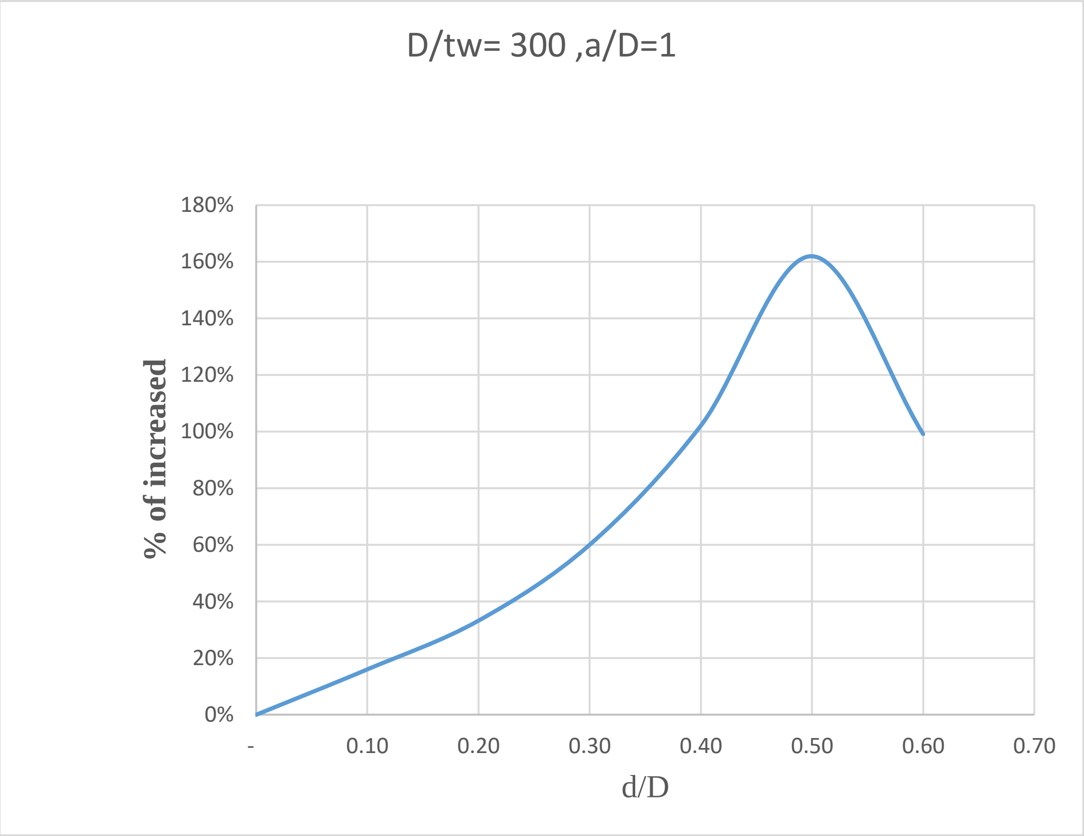


**Figure 4-8 percentage of increments**

The plots are for group 3 above and are grouped for the values of d/D. In the graph, the horizontal axis represents the location of the longitudinal stiffener measured from the bottom of the top flange, d/D and the vertical axis the percentage of increases as a result of providing longitudinal stiffener, where d represents the longitudinal stiffener location. Here the figure illustrates, the increased percentage of buckling load for each location of longitudinal stiffener. Therefore; the maximum increased percentage buckling load is 162% at d = 0.5D.

Overall, in this group, there is a maximum increasing buckling load of 771.2kN and 162% is increased when the longitudinal stiffener is placed at 0.5D.

### 4.4.1 Optimal Longitudinal Stiffener Location

The parametric studies indicated that the web in the critical panel reduced buckle as the longitudinal stiffener is placed far down from the bottom of the top flange until mid-depth of the web for a high shear low moment near support. Table 4-5 summarizes the results of the buckling load for all 21 girders.

**Table 4-5 Optimal location of the longitudinal stiffener**

| *Model* | *D/tw (slenderness limit)* | *a/D (Aspect Ratio)* | *d/D* | *Pcr (kN)* |
|---|---|---|---|---|
| 1 | 150 | 1 | 0.00 | 658.90 |
| 2 | | | 0.10 | 793.76 |
| 3 | | | 0.20 | 911.00 |
| 4 | | | 0.30 | 1,094.00 |
| 5 | | | 0.40 | 1,383.00 |
| 6 | | | 0.50 | 1,800.00 |
| 7 | | | 0.60 | 1,374.00 |
| 8 | | 2 | 0.00 | 385.74 |
| 9 | | | 0.10 | 503.55 |
| 10 | | | 0.20 | 607.57 |
| 11 | | | 0.30 | 736.07 |
| 12 | | | 0.40 | 849.44 |
| 13 | | | 0.50 | 879.99 |
| 14 | | | 0.60 | 830.03 |
| 15 | 300 | 1 | 0.00 | 294.50 |
| 16 | | | 0.10 | 341.70 |
| 17 | | | 0.20 | 392.32 |
| 18 | | | 0.30 | 471.18 |
| 19 | | | 0.40 | 594.93 |
| 20 | | | 0.50 | 771.20 |
| 21 | | | 0.60 | 586.37 |

From the above-summarized table, the maximum buckling load for Group 1 with aspect ratio (a/D) of 1 and slenderness ration (d/tw) of 150 is 1800KN, Group 2 with aspect ratio (a/D) of 2 and slenderness ration (d/tw) of 150 is 879.99KN, and Group 3 with aspect ratio (a/D) of 1 and slenderness ratio (d/tw) of 300 is 771.2KN, when the longitudinal stiffener is placed at d = 0.5D. For all Groups, the web is slenderer and it is seen from the results that the web in the critical panel buckles in almost all cases and when it reduced buckle, the longitudinal stiffener is at 0.5D. Therefore, 0.5D can be taken as an optimal location of the longitudinal stiffener.

### 4.4.2 Percentage of Increase in Buckling load

A graphical representation for all groups of plate girders is also shown below in figure 4.9.

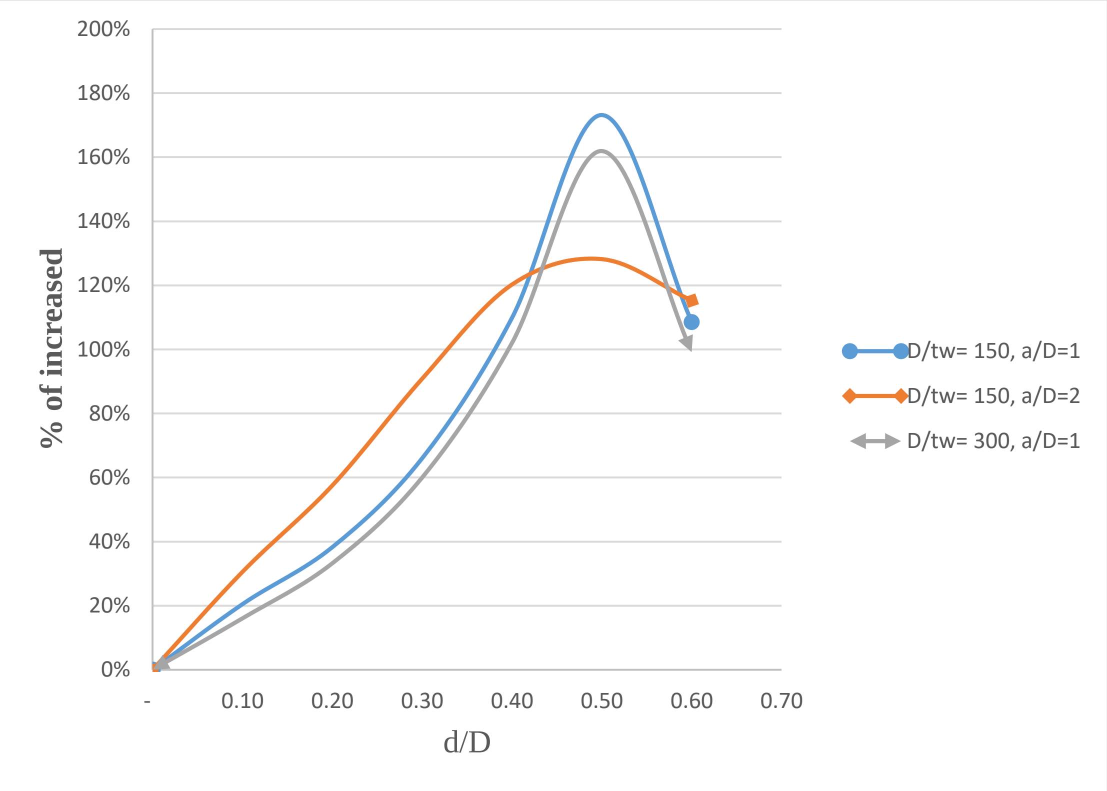


**Figure 4-9 percentage of increments**

The plots above are grouped together for the values of d/D and the percentage of increments. In the graph, the horizontal axis represents the location of the longitudinal stiffener measured from the bottom of the top flange, d/D and the vertical axis the percentage of increases as a result of providing longitudinal stiffener, where d represents the longitudinal stiffener location.

Here the figure illustrates, the increased percentage of buckling load for each group and location of the longitudinal stiffener. From the graph, as the aspect ratio (a/D) increases the influence of longitudinal stiffener on buckling load decreases, see for group 2. For group 1 and group 3 the increasing percentage of buckling load from the controller is high (173% and 162%).

For Group 1: D/tw=150 and a/D=1

Contribution of longitudinal stiffener = Maximum buckling load/controller buckling load

Contribution of longitudinal stiffener = 1800 kN / 658.9kN

Contribution of longitudinal stiffener = 2.72 times

For Group 2: D/tw=150 and a/D=2

Contribution of longitudinal stiffener = Maximum buckling load/controller buckling load

Contribution of longitudinal stiffener = 879.9 kN / 385.74kN

Contribution of longitudinal stiffener = 2.28 times

For Group 3: D/tw=300 and a/D=1

Contribution of longitudinal stiffener = Maximum buckling load/controller buckling load

Contribution of longitudinal stiffener = 771.2 kN / 294.5kN

Contribution of longitudinal stiffener = 2.62 times

The slenderness ratio is different for groups 1 and 3, and the contribution of longitudinal stiffener to increasing capacity of web plate girder is 2.72 and 2.62 times respectively. Therefore; the contribution of longitudinal stiffener 2.5 times the initial carrying capacity of plate girder for aspect ratio (a/D) of 1. Besides, the contribution of longitudinal stiffener to increase the strength for plate girders in group 2 with an aspect ratio of 2 is 2.28. From this, as the aspect ratio increases, the contribution of longitudinal stiffener decreases.

Therefore; the contribution of a longitudinal stiffener to increase the strength of the plate girder depends upon the aspect ratio. For aspect ratio, a/D =1, the contribution of longitudinal stiffener 2.5 times. Whereas for aspect ratio a/D=2, the contribution of the longitudinal stiffener is 2.

# CHAPTER FIVE

# CONCLUSION AND RECOMMENDATION

## 5.1 CONCLUSION

Advancement in materials and technology, increase in highway's network and bridges made the use of longer plate girders more frequent. So, the depth of girders is increasing making local web stability a concern, thus, the use of transverse and longitudinal stiffeners becomes important. An investigation was conducted to determine the optimal location of longitudinal stiffeners and the contribution of longitudinal stiffener in doubly symmetric, homogenous plate girder to increase the Buckling strength. To accomplish this task, ABAQUS (v.6.14.1), a commercial finite element software package was used to complete the study, and 21 models were analyzed. The research was completed with finite element models that were initially validated using results from experiments conducted for the classification of local buckling(26). The model parameters are web slenderness, D/tw, the aspect ratio of the panel between two consecutive transverse stiffeners, a/D, and the stiffener location, d/D. Models were constructed using shell elements, S4R, for all of the model's parts, i.e. web, flange, transverse and longitudinal stiffeners. Each girder in the general matrix is tested in the buckling module available in ABAQUS. The first positive eigenvalue for which the web in the panel under investigation buckles is taken as the critical load, Pcr. Web slenderness, D/tw, web panel aspect ratio, a/D, and d/D are varied throughout the study. For every girder and each location of the longitudinal stiffener, ABAQUS provided a critical load, Pcr. The highest value of Pcr, which represented the highest web buckling load for each longitudinal stiffener position in a given girder indicates the optimal stiffener location for that girder under the specified loading condition.

The girders studied were divided into three groups. Each group is composed of seven girders. A model for each combination of parameters is built, and seven cases are analyzed. In the first group, the maximum percentage increased value of Pcr at 0.5D location of the longitudinal stiffener is 173% of the controller. In the second group, the maximum percentage increased value of Pcr at 0.5D location of the longitudinal stiffener is 162% of the controller. In the third group, the maximum percentage increased value of Pcr at 0.5D location of the longitudinal stiffener is 128% of the plate girder without longitudinal web stiffener. From the results, the following conclusions are made for low moment high shear near at support condition.

1. The optimal location for longitudinal stiffeners under high shear low moment near at support condition is at d= 0.5D (at mid-depth of the web).
2. The contribution of a longitudinal web stiffener to increase the strength of the plate girder depends upon the aspect ratio value.
3. For a plate girder aspect ratio value of (a/D) 1 unit, the contribution of a longitudinal stiffener placed at the optimal location is 2.5 times the plate girder without longitudinal stiffener.
4. For a plate girder aspect ratio value of (a/D) 2 unit, the contribution of a longitudinal stiffener placed at the optimal location is 2 times the plate girder without longitudinal stiffener.

## 5.2 RECOMMENDATIONS

The following recommendations were made based upon the results from this study, which was limited to homogeneous, doubly-symmetric, single-span, simply-supported girders: This study investigated only two values of web slenderness, 150 and 300. Results show that the effective use of a longitudinal stiffener in I-girder depends on web slenderness for a high shear low moment load case near the support. Future work should be directed at:

1. Determining what value of web slenderness, the use of a longitudinal stiffener will begin to have a positive influence on the girders' strength to resist bending, or bending and shear.
2. The influence of moment to the shear ratio on optimal longitudinal stiffener location should also be investigated.
3. Finally, since composite girders with a concrete deck are commonly used, the use of a longitudinal stiffener and its location for composite girders should be investigated.

# REFERENCES

1. Alinia MM, Moosavi SH. A parametric study on the longitudinal stiffeners of web panels. Thin-Walled Struct. 2008;46(11):1213–23.
2. Xiao Y, Xue XY, Sun FF, Li GQ. Postbuckling shear capacity of high-strength steel plate girders. J Constr Steel Res. 2018;150:475–90.
3. Roberts TM, Davies AW. Fatigue induced by plate breathing. 2002;58:1495–508.
4. Maiorana E, Pellegrino C, Modena C. Influence of longitudinal stiffeners on elastic stability of girder webs. Vol. 67, Journal of Constructional Steel Research. 2011. p. 51–64.
5. Thurlimann B and. REPORT NOe 251~19. Plate girders – Strength in bending. Transactions of the American Society of Civil Engineers; 1963. 655–682 p.
6. Kim YD, Asce SM, Jung S, Asce SM, White DW, Asce M. Transverse Stiffener Requirements in Straight and Horizontally Curved Steel I-Girders. 2007;12(April):174–83.
7. Ventsel E. Thin Plates and Shells. 2001.
8. Chacón R, Mirambell E, Real E. Transversally stiffened plate girders subjected to patch loading. Part 1. Preliminary study. J Constr Steel Res [Internet]. 2013;80:483–91. Available from: http://dx.doi.org/10.1016/j.jcsr.2012.06.008
9. Chacón R, Mirambell E, Real E. Transversally and longitudinally stiffened steel plate girders subjected to patch loading. Thin-Walled Struct. 2019;138(December 2018):361–72.
10. Units CUS. AAS HTO L RFD B r idg e design specifications. Washington, DC 20001: American Association of State Highway and Transportation Officials; 2012.
11. Graciano C, Johansson B. Resistance of longitudinally stiffened I-girders subjected to concentrated loads. J Constr Steel Res. 2003;59(5):561–86.
12. Graciano C. Discussion on “Patch loading resistance of slender plate girders with longitudinal stiffeners” by B. Kövesdi [J Constr Steel Res 2018; 140: 237–246]. J Constr Steel Res [Internet]. 2018;150:261–3. Available from:

https://doi.org/10.1016/j.jcsr.2018.08.029

13. Helwig BTA, Member A, Frank KH, Yura JA. LATERAL-ToRSIONAL BUCKLING OF SINGLY SYMMETRIC I-BEAMS By Todd A. Helwig,. 1997;123(2):1172–9.
14. Kövesdi B, Alcaine J, Dunai L, Mirambell E, Braun B, Kuhlmann U. Interaction behaviour of steel I-girders; part II: Longitudinally stiffened girders. J Constr Steel Res [Internet]. 2014;103:344–53. Available from: http://dx.doi.org/10.1016/j.jcsr.2014.06.017
15. Gallego CAG, Medina DGZ. Effect of longitudinal stiffening on bridge girder webs at incremental launching stage. Ing e Investig. 2015;35(1):24–30.
16. Loaiza N, Graciano C, Casanova E. Design recommendations for patch loading resistance of longitudinally stiffened I-girders. Eng Struct [Internet]. 2018;171(August 2017):747–58. Available from: https://doi.org/10.1016/j.engstruct.2018.06.019
17. Maiorana E, Pellegrino C, Modena C. Influence of longitudinal stiffeners on elastic stability of girder webs. J Constr Steel Res. 2011;67(1):51–64.
18. Park YM, Lee KJ, Choi BH, Cho K Il. Modified slenderness limits for bending resistance of longitudinally stiffened plate girders. J Constr Steel Res. 2016;122:354–66.
19. Graciano C, Lagerqvist O. Critical buckling of longitudinally stiffened webs subjected to compressive edge loads. J Constr Steel Res. 2003;59(9):1119–46.
20. Rhodes S, Cakebread T, York N. Understanding Buckling Behavior and Using FE in Design of Steel Bridges. 2012;
21. Jáger B, Kövesdi B, Dunai L. I-girders with unstiffened slender webs subjected by bending and shear interaction. J Constr Steel Res [Internet]. 2017;131:176–88. Available from: http://dx.doi.org/10.1016/j.jcsr.2017.01.009
22. Aggarwal K, Wu S, Papangelis J. Finite element analysis of local shear buckling in corrugated web beams. Eng Struct [Internet]. 2018;162(August 2017):37–50. Available from: https://doi.org/10.1016/j.engstruct.2018.01.016

23. Kala Z, Valeš J, Martinásek J. Inelastic Finite Element Analysis of Lateral Buckling for Beam Structures. Procedia Eng [Internet]. 2017;172:481–8. Available from: http://dx.doi.org/10.1016/j.proeng.2017.02.056

24. Valeš J, Stan TC. FEM Modelling of Lateral-Torsional Buckling Using Shell and Solid Elements. Procedia Eng [Internet]. 2017;190:464–71. Available from: http://dx.doi.org/10.1016/j.proeng.2017.05.365

25. Solids Division of Engineering Brown University. ABAQUS Tutorial (EN175 : Advanced Mechanics of Solids). Mech Solids [Internet]. 2006;37. Available from: http://www.engin.brown.edu/courses/En175/Abaqustut/abaqustut.htm

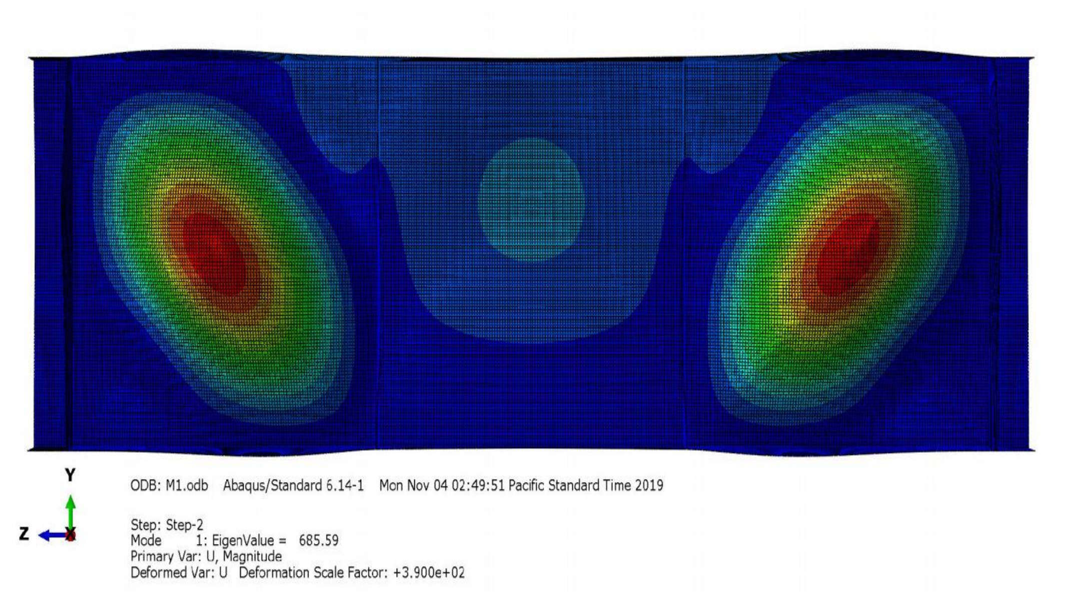
26. Zhu Y, Zhao J. Experimental and numerical study on member classification of I-section plate girders. J Constr Steel Res. 2017;137(June):192–200.

# APPENDEX A

FEA (ABACUS) RESULTS

GROUP 1: - D/tw=150, a/D=1

d/D=0.00



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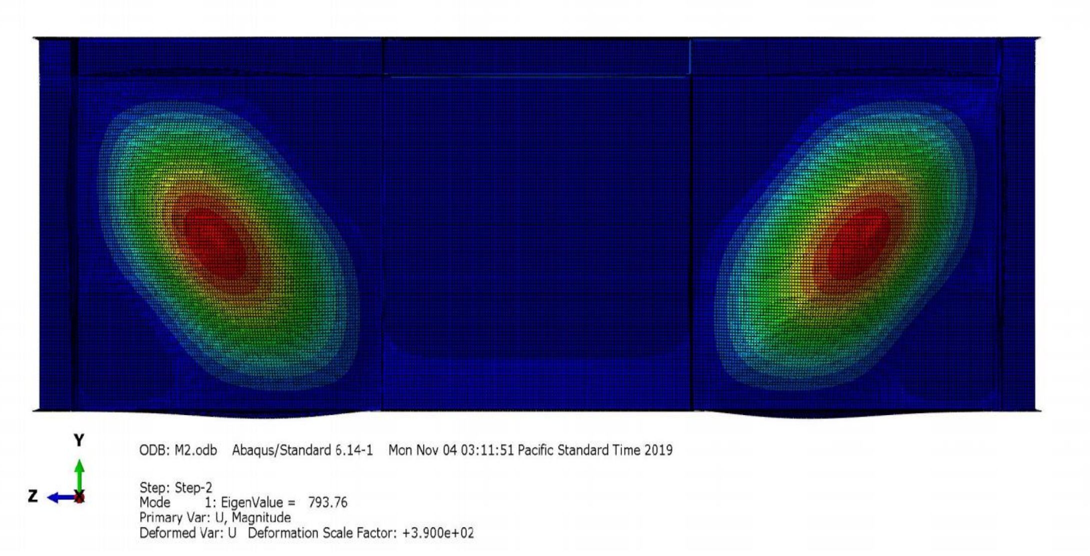

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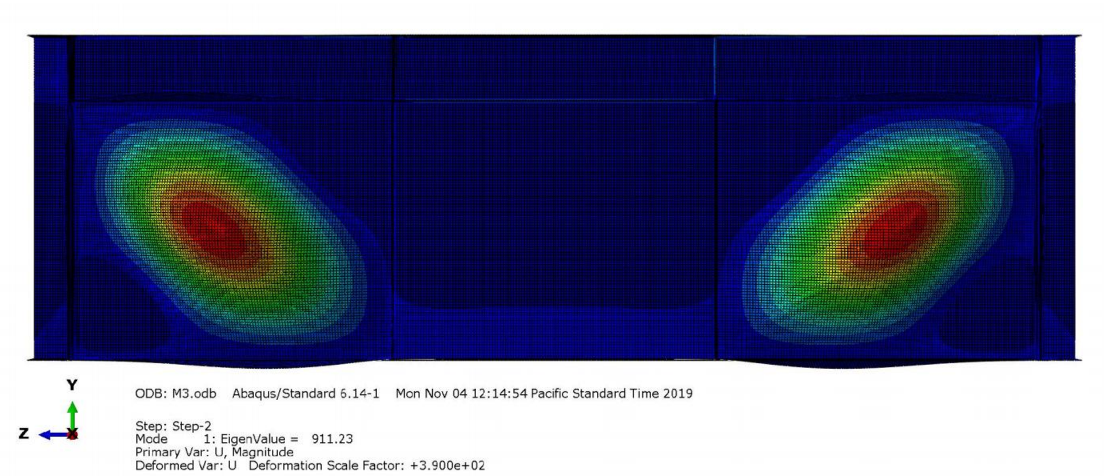

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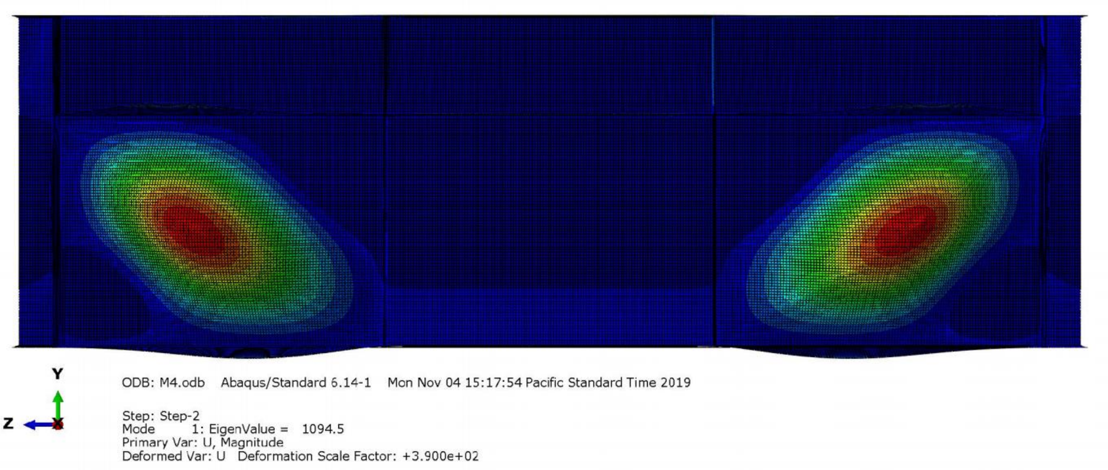
Primary Var: U, Magnitude
Deformed Var: U   Deformation Scale Factor: +3.900e+02
```

d/D=0.30



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Step: Step-2
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Primary Var: U, Magnitude
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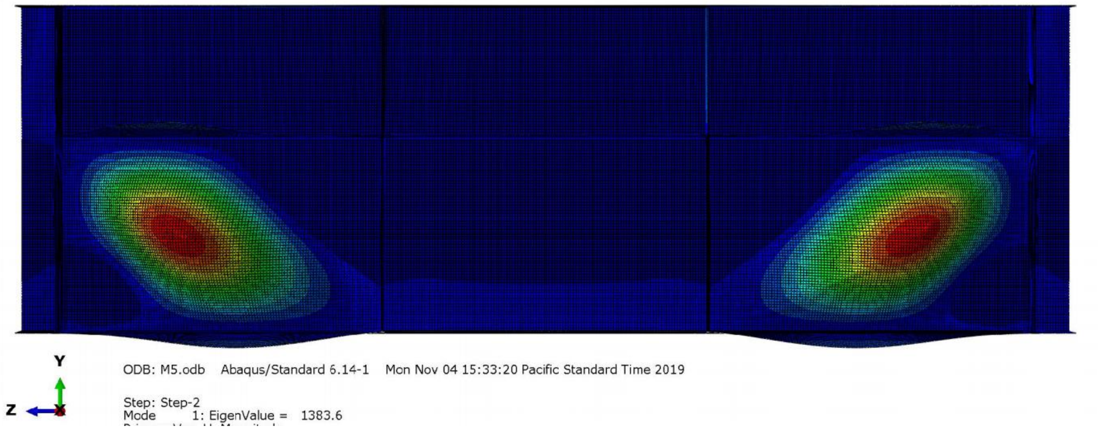

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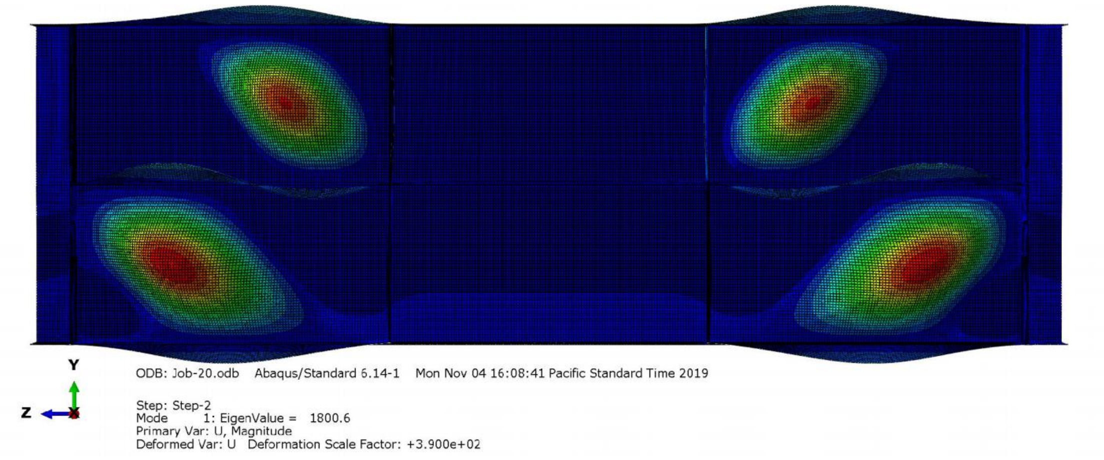

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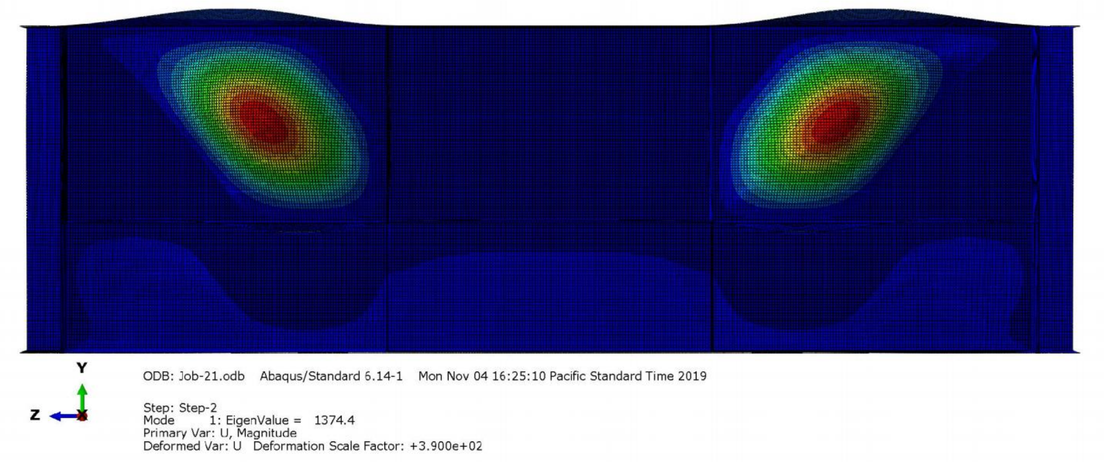

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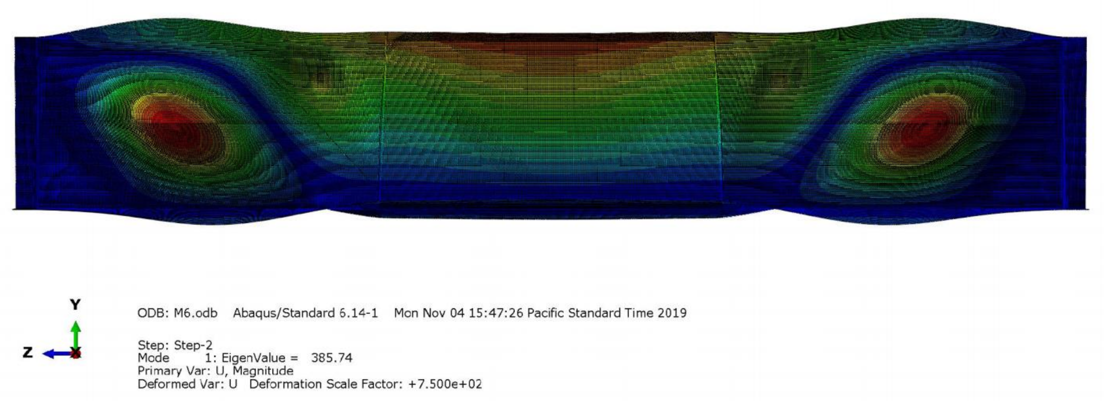
GROUP 2: - D/tw=150, a/D=2

d/D=0.00



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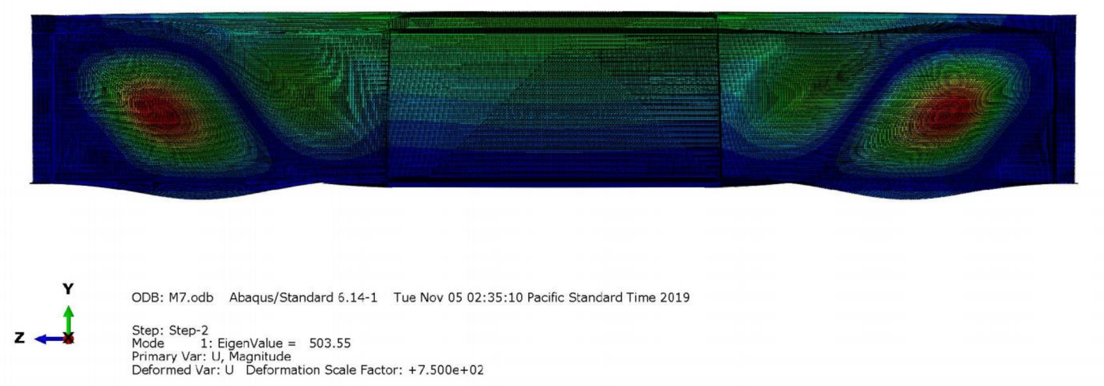

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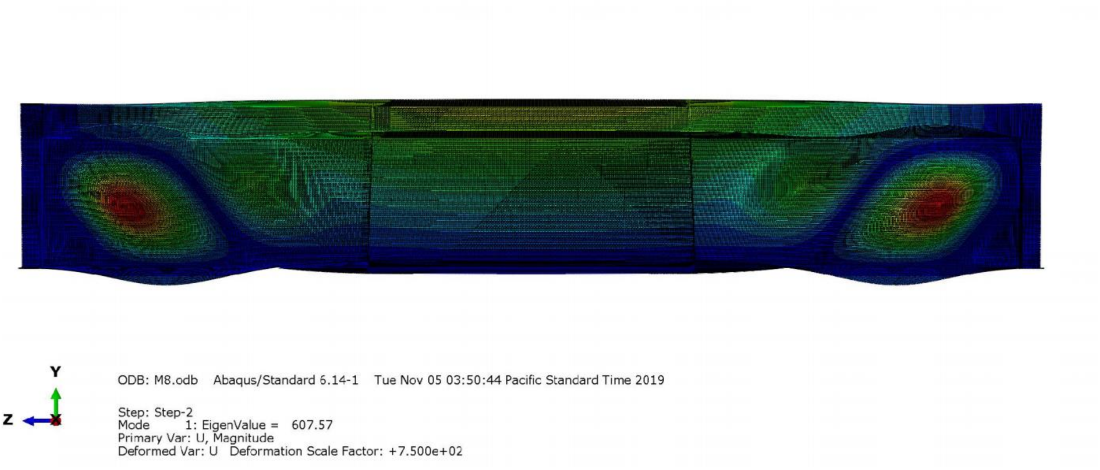

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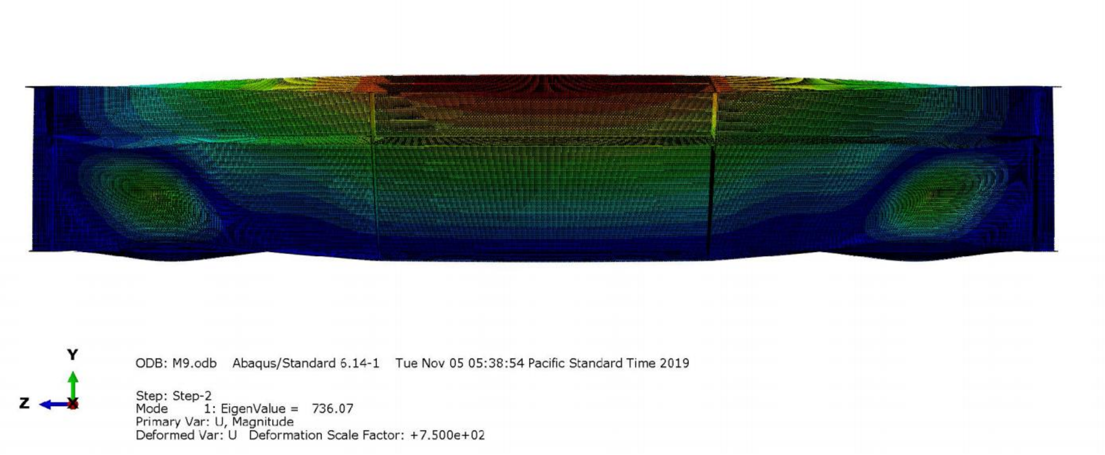
d/D=0.30



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Primary Var: U, Magnitude
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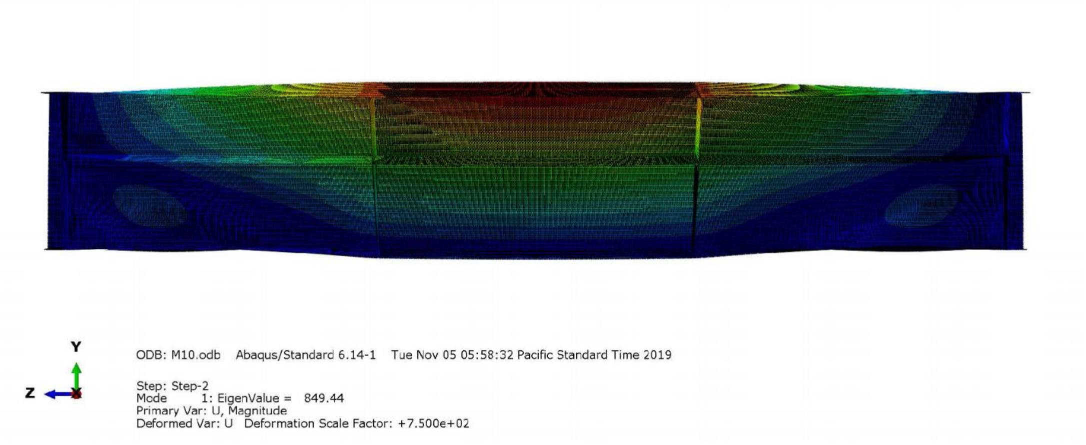
d/D=0.40



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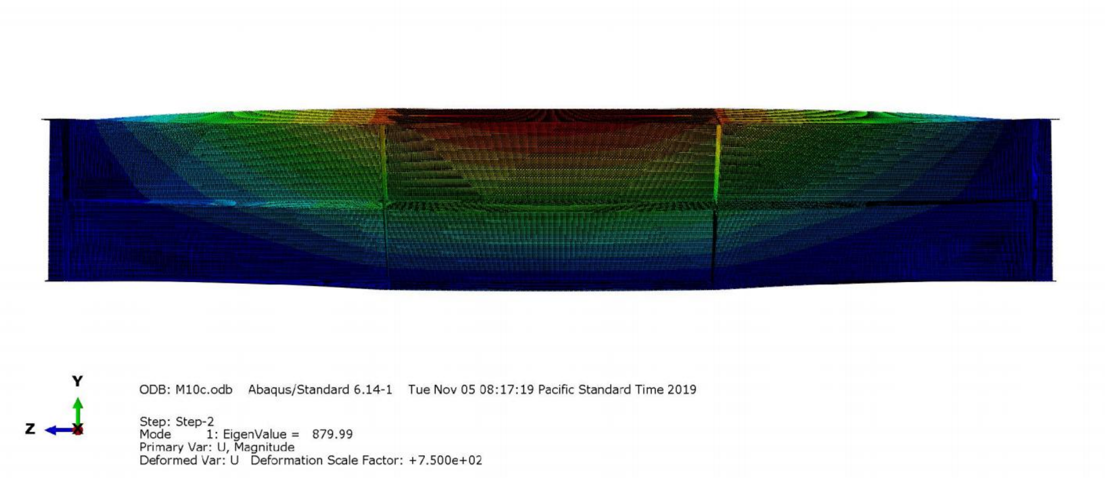
Primary Var: U, Magnitude
Deformed Var: U Deformation Scale Factor: +7.500e+02

d/D=0.50



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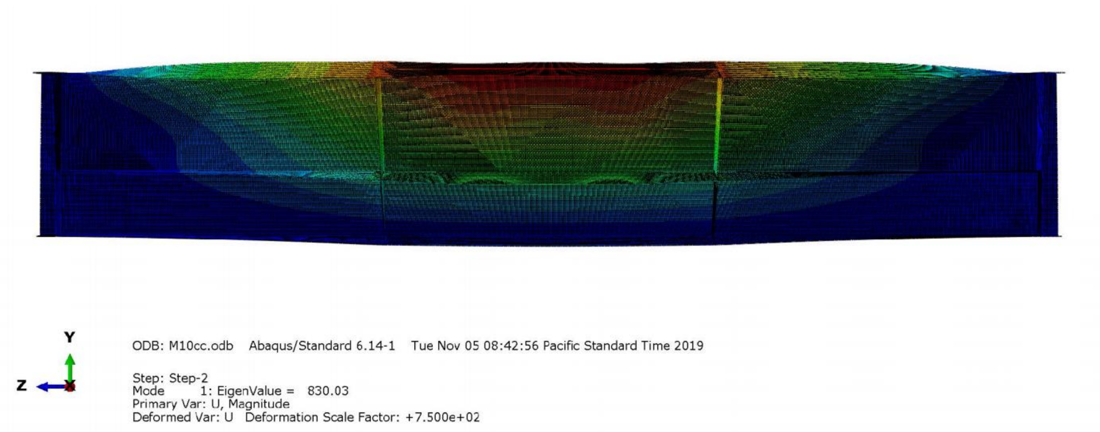
d/D=0.60



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Deformed Var: U   Deformation Scale Factor: +7.500e+02
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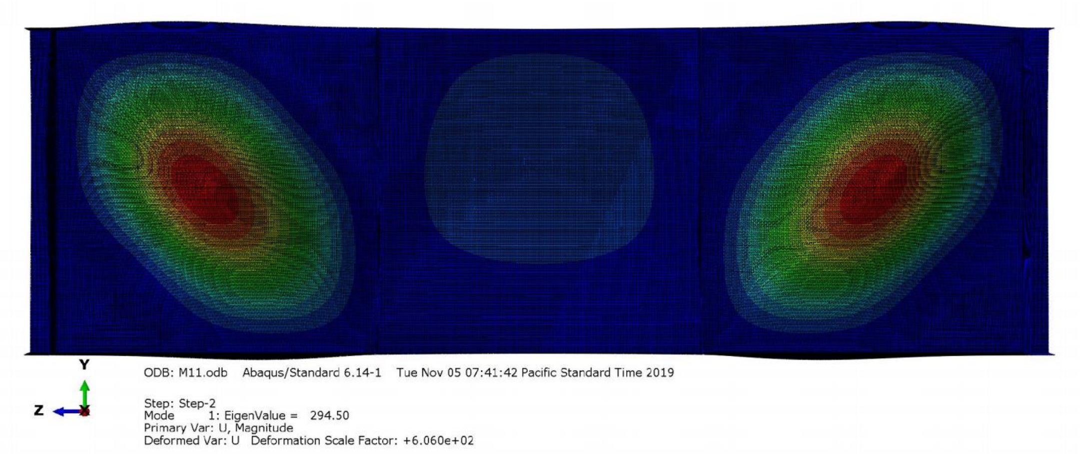
GROUP 3: - D/tw=300, a/D=1

d/D=0.00



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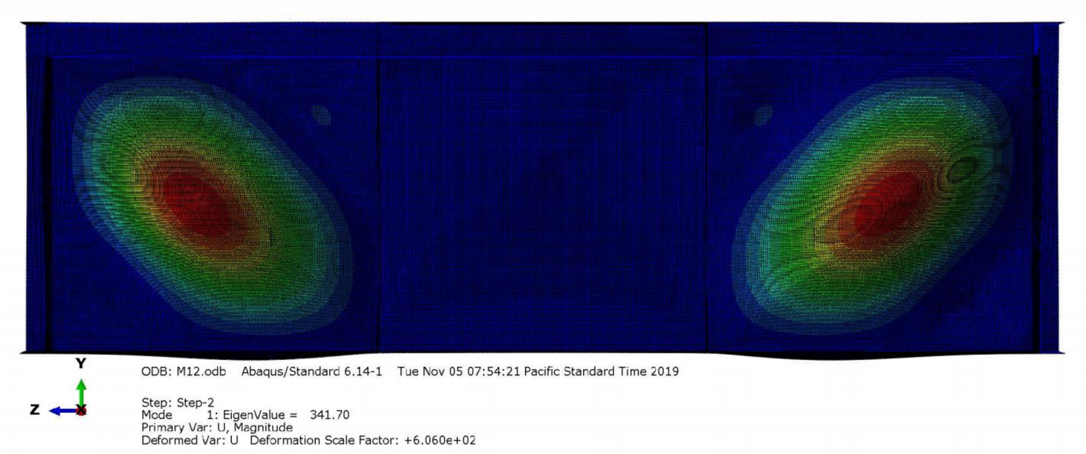

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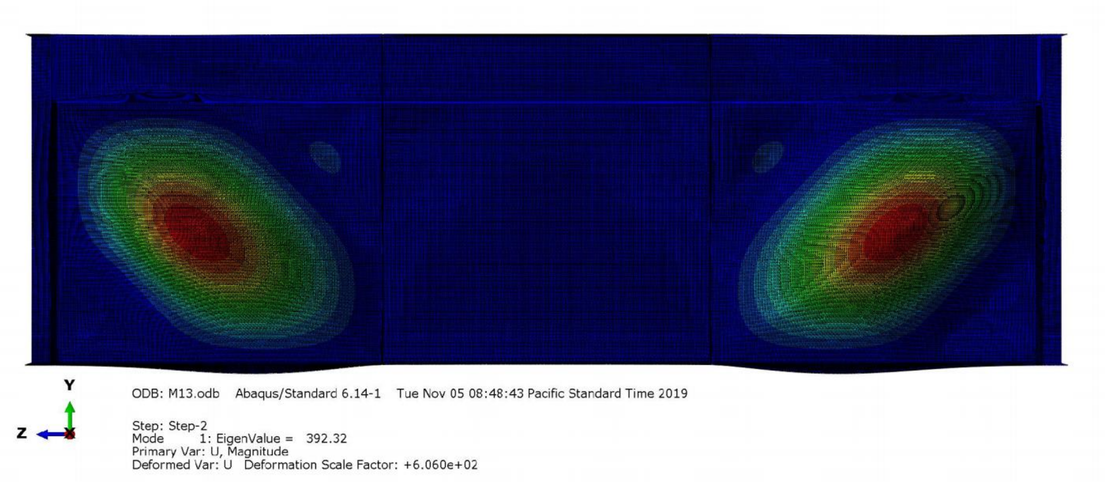
```

d/D=0.20



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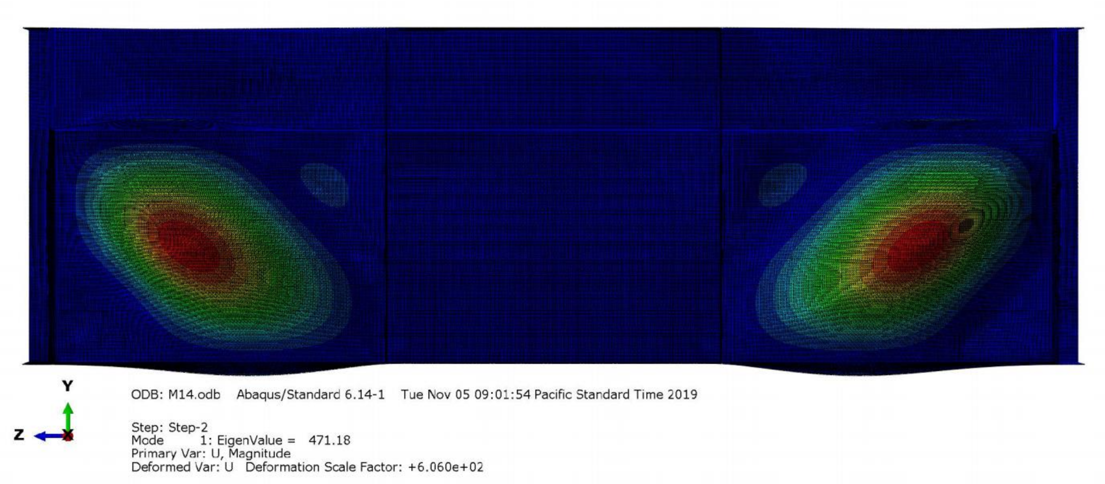

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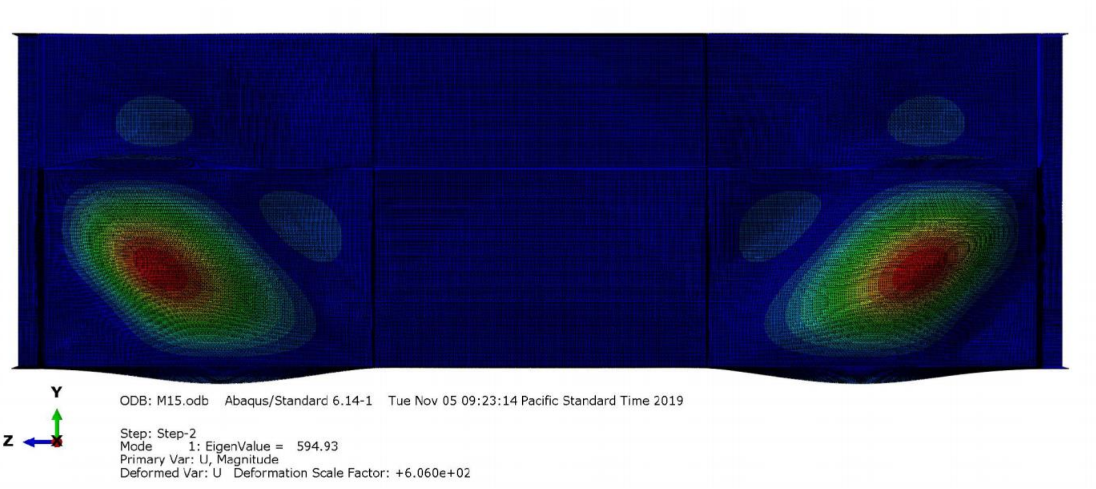
d/D=0.40



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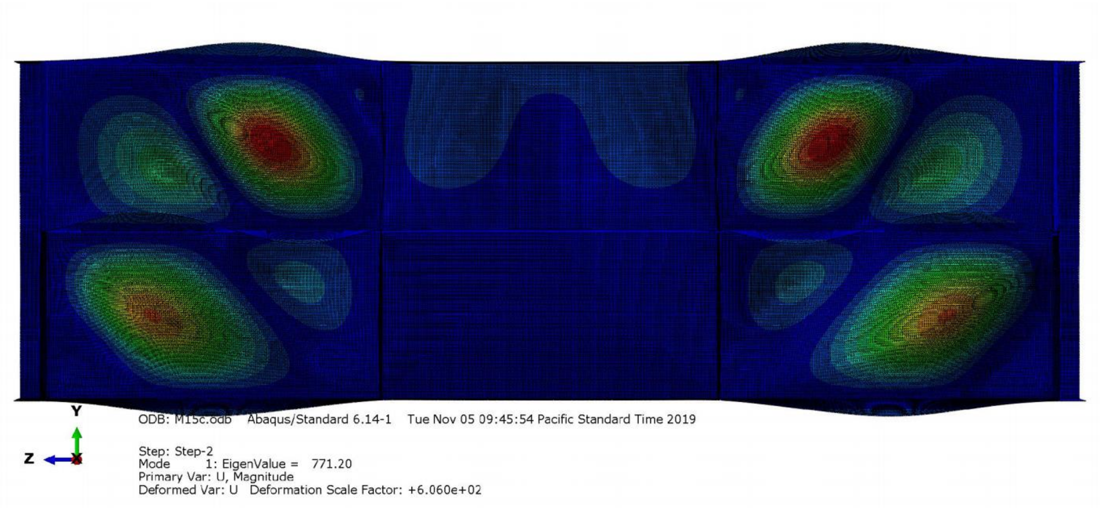
d/D=0.50



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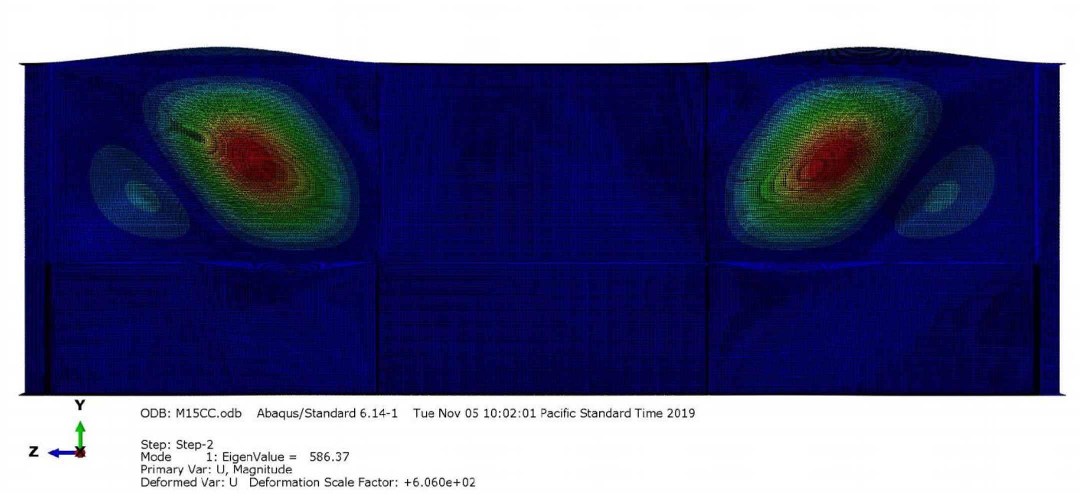
```

d/D=0.60



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Deformed Var: U   Deformation Scale Factor: +6.060e+02
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