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Finite Element Analysis and Optimization of Hollow Flange Steel Girders

A Thesis Submitted to the Council of College of Engineering, University of Diyala in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

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I certify that the thesis entitled "**Finite Element Analysis and Optimization of Hollow Flange Steel Girders**" presented by "**Usamah Mahdi Salih**" was prepared under my supervision in the Department of Civil Engineering, University of Diyala, in partial Fulfillment of the Requirement for the Degree of Master of Science in Civil Engineering

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We certify that we have read the thesis titled (**Finite Element Analysis and Optimization of Hollow Flange Steel Girders**) and we have examined the student (**Usamah Mahdi Salih**) in its content and what is related with it, and in our opinion, it is adequate as a thesis for the degree of Master of Science in Civil Engineering**.**

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DEDICATION

I dedicate this study with much Gratitude and Love to;

My Dear Father;

His words of inspiration and encouragement in pursuit of excellence.

My Affectionate Mother;

Who have always encouraged and supported me,

My Family

Finally, to My Friends.

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Abstract

Finite Element Analysis and Optimization of Hollow Flange Steel Girders with Web Openings

By

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Optimization techniques may be effective in finding alternative geometries of steel girders to improve their structural behavior, particularly avoiding or reducing the bending moments. In structural, architectural and bridge engineering, conventional steel I-girders are fabricated generally by welding two plate flanges, a flat web and a series of transverse or longitudinal stiffeners together. This study investigates the structural behavior of hollow flange steel girders. A nonlinear finite element model for the analysis of hollow flange steel girders was developed. The numerical analysis was conducted by the finite element ANSYS software and was carried out on different hollow flange steel girders chosen from literature. A parametric study was conducted to investigate the effect of several selected parameters on the overall behavior of hollow flange steel girders. These parameters include the effect of cross-section, web openings shape, size and location. The results showed that the ultimate capacity increase when the section hollow, the load failure of HFSPG increased 28% and 59.6% as compared with UHFSPG and LHFSPG, the ultimate capacity of HFSPG with 4 square openings decreased 4.8% compared to HFSPG without openings, the ultimate capacity of HFSPG with circle openings increased compared with HFSPG with square openings and the ultimate capacity of HFSPG with web opening near support increased 5.23% and 3.25% compared with HFSPG with web opening in quarter and center of girder respectively.

The finite element software package ANSYS (v 12.1) was used to find the optimum volume of the hollow flange steel plate girders. Two cases were considered, which are optimization of steel girder under three-concentrated loads and two-concentrated loads. The objective function is minimization of total volume of the girder. The constraints are tensile stress in steel, shear stress in steel and displacement at mid-span of the girder. The design variables are the height of top and bottom hollow flanges, width of top and bottom hollow flanges, the thickness of top and bottom hollow flanges, the height and thickness of the web. The result showed that the optimum volume for the steel girder decreased about 13 % than the initial for the steel girder under three-concentrated loads and decreased 34.8 compared with HFSPG under two-concentrated loads.

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List of Abbreviation

Abbreviation	Definition
AASHTO	American Association of State Highway and Transportation
AISC	American Institute of Steel Construction
AS/NZS 4600	Australian/New Zealand standard for the design of cold-formed steel structures
LRFD	Load and Resistance Factor Design
HPS	High Performance Steel
FE	Finite Element
ANN	Artificial Neural Networks
LSB	Light Steel Beam
BS	British standards
FEM	Finite Element Method
HFSPG	Hollow Flange Steel Plate Girder
UHFSPG	Upper Hollow Flange Steel Plate Girder
LHFSPG	Lower Hollow Flange Steel Plate Girder

List of Symbols

Symbol	Definition
bf	Width of Flange
D	Depth of Web
E_s	Modulus of Elasticity of Steel
ε_y	Yield Strain
FH1	Height of Upper Flange Opening
FH2	Height of Lower Flange Opening
FT1	Thickness of Upper Flange
FT2	Thickness of Lower Flange
FW1	Width of Upper Flange
FW2	Width of Lower Flange
f_y	Yield Strength
Iyc	Moment of Inertia of The Compression Flange
Iyt	Moment of Inertia of The Tension Flange
ndof	Number of Degree of Freedom
Sz max.	Maximum Tensile Stress
Sz min.	Minimum Tensile Stress
WH	Height of The Web
WT	Thickness of The Web
tf	Thickness of Flange
t_w	Thickness of Web
Uy	Mid-span Deflection
ν	Poison Ratio of Steel

CHAPTER ONE
INTRODUCTION

Chapter One

Introduction

1.1General

Conventional steel I-girders have been widely used in several fields, such as structural engineering, architectural engineering, and bridge engineering. Typically, they are produced by welding two plate flanges, a flat web, and a series of transverse or longitudinal stiffeners. Since the 1960s, substantial experimental and theoretical research has been conducted on the behavior of plate girders subjected to shear or patch stress, and conventional I-girder behaviour is now well understood.

In a traditional I-girder, the web plate resists the majority of the generated shear force, while the flat-plate flanges support the majority of the bending moment. In compared to the axial force, the shear force created in the flange is much smaller. In most circumstances, the thickness of the web plate is meant to be less than the thickness of the flanges.

Shear buckling, compression buckling, or crippling of the web, local buckling of the compression flange, flange-induced buckling of webs, and flexural failure controlled by flexural-torsional buckling or determined by plastic hinges are all possible failure mechanisms for an I-girder with a thin web **(Hassanein and Kharoob 2010)**.

Typically, transverse or longitudinal stiffeners are added to a web to strengthen its resistance to local buckling. This method has two negative consequences: Initially, the I-self-weight of the girder is increased by installing a number of stiffeners. Second, the I-girder is susceptible to fatigue or seismic stress because the stiffeners are welded to the web and flanges, which renders specific sections close to the welds more brittle. In recent years, it has been common to substitute corrugated web for traditional plate web when constructing I-girders to minimize the number of stiffeners or avoid their installation **(Sause and Braxtan 2011)** and **(Nie et al. 2013)**.

1.2 Standards

The American Association of State Highway and Transportation Officials (AASHTO) compactness standards for bridge design have been basically identical to those included in the current American Institute of Steel Construction (AISC) steel structure specification.

The most recent AASHTO Load and Resistance Factor Design (LRFD) Specification **(AASHTO)**, prescribes compactness requirements that are identical to the AISC web compactness provisions. Previously, there were slight differences between the AISC and AASHTO web compactness provisions in earlier editions of the AASHTO Specification due to differing conventions regarding what constitutes the appropriate plate width (i.e. for the web, the full cross-sectional depth, or the clear distance between the flanges, etc), prescribes compactness requirements that are identical to those outlined in the third edition of the AISC LRFD Specification **(Orbison el al 1999)**. In these specifications, it has been implicitly assumed that differences in steel grades can be accounted for by including a scaling factor related to the inverse of the square root of the yield stress associated with the web or flange in the case of the AISC LRFD building specification or the compression flange in the case of the AASHTO LRFD **(AASHTO)**.

It appears improbable that a single scaling factor based only on yield stress could account for all of the behavioral changes that follow the considerable variances in uniaxial material responses that are typical of changes in high performance steel grades.

With the exception of variances in yield strength and strain hardening slope, the typical uniaxial stress-strain responses of most presently available constructional steel grades are comparable to those of new High Performance Steel (HPS) grades. In several key areas of the uniaxial stress–strain relationship, the material properties of high performance steels tend to differ from those of mild carbon steel or its equivalent; HPS grades frequently exhibit neither a well-defined yield plateau nor a substantial strain-hardening modulus compared to more commonly used grades. Additionally, certain HPS grades may be considerably less ductile than their more prevalent equivalents.

The often used HPS grade for bridges, A709 HPS483W, has a somewhat steep strain hardening slope and high ductility, although its yield plateau is rather short compared to other grades. All of the above-mentioned material response characteristics can have a significant impact on the observed structural ductility of I-shaped girders **(Earls 2002)**, and it is therefore believed that the current practice of accounting for different steel grades in design provisions by using a scaling factor proportional to the square root of the minimum specified yield stress may not be adequate for applications involving HPS **(Earls 2002)**.

1.3 Optimization

Optimization is the process of determining the optimal values of decision variables, given a set of constraints and in accordance with a chosen optimization objective function.

The most prevalent optimization approach applies to a design that minimizes overall cost, maximizes feasible dependability, or achieves any other particular purpose. There are several challenges in science and engineering, corporate decision-making, and industry that need the use of an optimization strategy **(Faluyi and Arum 2012)**.

1.4 Section Proportion of Steel Girder

1.4.1 Webs

The web supplies the girder with primarily shear strength. Since the web contributes little to the bending resistance, its thickness should be as minimal as possible to satisfy the web depth to thickness ratio limitations $D/Wt \leq 150$ for webs without longitudinal stiffeners and $D/Wt \leq 300$ for webs with longitudinal stiffeners (AASHTO 6.10.2.1). It is more convenient to have web depth increments of 5.08 cm or 7.62 cm. Web depths over 304.8 cm will need both longitudinal and vertical splices **(AASHTO)**.

The web thickness is preferred to be not less than 1.27 cm. A thinner plate is subject to excessive distortion from welding. The thickness should be sufficient to preclude the need for longitudinal stiffeners. Web thickness should be constant or with a limited number of changes **(AASHTO)**.

1.4.2 Flanges

The flanges provide bending resistance. Flanges must be a minimum of 30.48 cm wide. Preferred is a girder with a consistent flange width along its length. If the flange area to be expanded, it is better to alter the flange's thickness. If flange widths must be altered, it is preferable to do so only at field splices.

The increments in width should be multiples of 5.08 or 7.62 cm. The flange width of horizontally curved girders should be about one-fourth of the web depth. A flange width of around one-fifth to one-sixth of the web depth should be enough for straight girders.

The minimum flange thickness for straight girders is 7.62 cm. For curved girders, a minimum thickness of 2.5 cm is feasible. The desired maximum thickness of the flange is 1.905 cm. Steels of Grade 50 and HPS 70W are not available in thicknesses exceeding 10.16 cm. Flange thickness increments should be 0.3175 cm for thicknesses up to 2.54 cm, 0.635 cm for thicknesses between 2.54 and 7.62 cm, and 1.27 cm for thicknesses between 7.62 and 10.16 cm **(AASHTO)**.

At areas where the thickness of the flange changes, the thicker flange should offer about 25 percent greater surface area than the thinner flange. In addition, the thickness of the thicker flange should not exceed double that of the thinner flange. Both the compression and tension flanges shall meet the following proportion requirements (AASHTO 6.10.2.2-2) as follows:

$$\frac{bf}{2tf} \leq 12 \qquad \text{(AASHTO 6.10.2.2-1) (1.1)}$$

$$bf \geq \frac{D}{6} \qquad \text{(AASHTO 6.10.2.2-2) (1.2)}$$

$$tf \geq 1.1t_w \qquad \text{(AASHTO 6.10.2.2-3) (1.3)}$$

$$o.1 \leq \frac{Iyc}{Iyt} \qquad \text{(AASHTO 6.10.2.2-4) (1.4)}$$

Where bf and tf are the entire width and thickness of the flange; tw is the thickness of the web; Iyc and Iyt are the moments of inertia of the compression flange and tension flange around the vertical axis in the plane of the web, respectively; and D is the depth of the web. AASHTO 6.10.2.2-1 ensures that the flange won't deform excessively when welded to the web.

Equation AASHTO 1.2 ensures that stiffened interior web panels can develop post-elastic buckling shear resistance via tension field action when welded to the web. Equation AASHTO 1.3 assures that flanges may offer sufficient restraint and boundary conditions to prevent web shear buckling. Equation AASHTO 1.4 enables more efficient flange proportions and eliminates the use of sections that are potentially difficult to manipulate during construction. It also assures that the AASHTO formulae for lateral torsional buckling are valid **(AASHTO)**.

1.5 Web Openings

The shape of the web openings will depend upon the designer's choice and the purpose of the opening. There are no hard and fast rules to dictate the shapes of the openings. But, for designer's convenience, openings of regular shapes (such as circular or rectangular) are usually chosen. Introduction of openings in the web decreases stiffness of the beams resulting in larger deflections than the corresponding beams with solid webs. The strength of the beams with openings may be governed by the plastic deformations that occur due to both moment and shear at the openings.

The strength realised will depend on the interaction between the moment and shear. The moment capacity of the perforated beam will be reduced at the opening because of the reduction in the contribution of web to the moment capacity. This is not very significant, as usually the contribution of the web to the moment capacity is very small.

However, the reduction in shear capacity at the opening can be significant. Therefore, the ultimate capacity under the action of moment and shear at the cross section where there is an opening will be less compared to that at the a normal cross section without opening. i.e. some strength is lost. To restore the strength lost, reinforcement along the periphery of the openings could be provided. As a general rule, we should avoid having openings in locations of high shear, nor should they be closely spaced. Bellow some guide lines for web openings:

- The hole should be centrally placed in the web and eccentricity of the opening is avoided as far as possible.
- Unstiffened openings are not always appropriate, unless they are located in low shear and low bending moment regions.

- Web opening should be away from the support by at least twice the beam depth, D or 10% of the span ("), whichever is greater
- The best location for the opening is within the middle third of the span.
- Clear Spacing between the openings should not be less than beam depth, D.
- The best location for opening is where the shear force is the lowest.
- The diameter of circular openings is generally restricted to 0.5D.
- Depth of rectangular openings should not be greater than 0.5D and the length not greater than 1.5D for un-stiffened openings. The clear spacing between such opening should be at least equal the longer dimension of the opening.
- The depth of the rectangular openings should not be greater than 0.6D and the length not greater than 2D for stiffened openings. The above rule regarding spacing applies.
- Corners of rectangular openings should be rounded.
- Point loads should not be applied at less than D from side of the adjacent opening.

1.6 Purpose and Scope of the Study

The purpose of this study is to develop design optimization models for hollow flange steel girders with web openings by nonlinear finite element analysis. A steel girder was optimized by using the ANSYS program; it helps to obtain an optimum design complying with the design constraints. The parametric study were the width, hight, and thickness of upper and lower flanges and the hight and thickness of the web. The size of the girder was optimized under constraint of stress limited to the yield stress.

1.7 Layout of the Study

In order to achieve the objectives mentioned above, this thesis is organized in to six chapters: In chapter one contains introduction and the aim and objective of the thesis are summarized. Chapter two briefly gives the background on the previous studies about finite element and optimization of steel girders.

Chapter three focused on FE method and the comparison of the FE model predictions with the experimental findings of the laboratory tested bridge models for various load cases to verify the FE model and provide information about the nonlinear response of steel girder. Chapter four lists the parametric study which has the effect of cross-section, the effect of web openings shape and the effect of openings shape on ultimate capacity. Chapter five lists the optimization historical development, applications, methods, based on FE, numerical method, strategy, process, design variables, constrains, objective function, and optimization of hollow flange steel plate girder which has the results and discussions. Chapter six lists the most important conclusions from the discussions and the overall results of the analysis.

CHAPTER TWO
LITERATURE REVIEW