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Araştırma Makalesi Research Article

AN ANALYTICAL STUDY ON STEEL GUSSET PLATES

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Keywords
Bridges,
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Steel structures,
Steel gusset plates.

Abstract
On August 1, 2007, the Interstate 35W (I-35W) bridge over the Mississippi River in Minneapolis, Minnesota USA, collapsed, resulting in 13 deaths and 145 injuries. The cause of the collapse was a design flaw resulted in week gusset plate connection. This very tragic example showed us again that, gusset plate design and implementation on bridges have crucial importance. Designers should give significant consideration during the design of steel gusset plate connections. In this study, the buckling behavior of steel gusset plates was investigated. An automated finite-element analysis program (Higgins et al.,2016 ; Scott et al., 2008), was used to analyze the steel gusset plates behavior under compression load. An uniaxial stress strain plasticity model was considered in the analysis. The stress-strain relationship of the steel gusset plates was defined in three dimensions. Nonlinear static analysis of the gusset plates were performed. Different out of plane imperfection, different slenderness ratios, different element sizes were considered during the analysis. The effect of change in the fastener(bolt) location on the steel gusset plates were also examined. Normalization of the ultimate failure loads of the given gusset plate with the critical Euler's buckling load was defined. The variation of this normalization with respect to the change in the steel gusset plate parameters was analyzed. It was obtained that, imperfections have significant effect on the displacement capacity of the steel gusset plates under compression loads.

ÇELİK GUSE (BİRLEŞİM) PLAKLARI ÜZERİNE ANALİTİK BİR ÇALIŞMA

Anahtar Kelimeler
Çelik yapılar,
Çelik guse plakları,
İnşaat mühendisliği,
Köprüler,
Köprü birleşim elemanları.

Öz
1 Ağustos 2007 tarihinde, Minnesota Amerika'da, Mississipi Nehri üzerinde bulunan, Eyaletlerarası 35W (I-35W) adlı köprü çökmüştür. Bu trajik vaka, 13 ölüm, 145 de yaralı ile sonuçlanmıştır. Bu üzücü kazaya çelik guse plakasındaki tasarım kusuru nedeni ile oluşan zayıf birleşimin yol açtığı tespit edilmiştir. Bu trajik kaza, çelik guse plaklarının köprülerde tasarımının ve tatbikinin can alıcı öneme sahip olduğunu göstermiştir. Tasarım mühendislerinin çelik guse plaklarının dizaynına kayda değer ehemniyet vermesi gerekmektedir. Bu çalışmada çelik guse plakların burkulma davranışları incelenmiştir. Çelik guse plaklarının sonlu elemanlar yöntemine göre basınç yükü altındaki analizinde, (Higgins et al., 2016 ; Scott et al., 2008) tarafindan hazırlanmış bir program kullanılmıştır. Analizlerde tek eksenli gerilme şekil değiştirme plastisite modeli göz önüne alınmıştır. Çelik guse plaklarının üç boyutlu gerilme şekil değiştirme durumu tanımlanmış ve doğrusal olmayan statik analizler gerçekleştirilmiştir. Analizler esnasında, çelik guse plaklarına, farklı düzlem dışı kusurların, farklı narinlik katsayılarının ve farklı eleman boyutlarının etkisi incelenmiştir. Çelik guse plakalarında bulunan civataların plak içerisinde konumlarının değişiminin davranışa etkisi de araştırılmıştır. Bu parametrelerdeki değişimin, maksimum göçme yükünün kritik Euler burkulma yükü oranına etkisi de incelenmiştir. Analizler sonucunda düzlem dışı kusurların çelik guse plaklarının basınç yükleri etkisi altındaki deplasmanlarına önemli derecede etki ettiği görülmüştür.

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1. Introduction

On August 1, 2007, the Interstate 35W (I-35W) bridge over the Mississippi River in Minneapolis, Minnesota USA, collapsed, resulting in 13 deaths and 145 injuries. The failure was attributed to a design flaw that resulted in weak gusset plates at connection. Gusset plates are metal plates commonly used as connections to transfer forces between structural members. A gusset plate and collapsed bridge deck truss section from this bridge are shown in Figures 1 and 2 respectively (NTSB, 2008). After this tragic collapse, researchers showed considerable interest in the strength, behavior and failure modes of steel gusset plate connections (Lehman et al.,2008; Higgins et al., 2013; Berman et al., 2012; Bucmys and Daniunas, 2015). Accident report of this bridge was prepared by (NTSB, 2008). The United States National Transportation Safety Board (NTSB) presented in the accident report that the cause of the failure was determined to be an under designed gusset plate at node U10, which failed due to sway buckling. After the failure, the Federal Highway Administration (FHWA) issued a design guide which uses a combination of buckling stress and column theory to predict the buckling load (FHWA, 2009). (Ocel et al., 2011) described the collapse and FHWA's response to the recommendations intended to prevent similar failures in the future.

Figure 1. A Gusset Plate on I-35W bridge (Figure taken from NTSB, 2008)

Figure 2. Collapsed deck truss sections of I-35W bridge.(Figure taken from NTSB, 2008)

One of the first pioneering research about gusset plates was carried out by (Whitmore, 1952). Whitmore investigated stresses in gusset plates experimentally and he defined the well-known and widely used Whitmore effective width of the stress distribution. (Fang et al., 2015a) presented experimental, numerical and analytical investigations of the compressive behavior of eccentrically loaded gusset plate connections. A numerical study on the buckling behavior of gusset plate connections under compression was carried out by (Fang et al., 2015b). A finite-element model of a gusset plate was developed and verified against experimental measurements in (Crosti and Duthinh., 2014). The design strength equations for gusset plates in the various codes were surveyed and the pertinent strength equations for gusset plates of steel truss bridges were selected by (Kasano et al., 2012). The authors of this study also proposed a pair of strength equations for compression and shear block failure for gusset plates subjected to compressive force. Several finite element models were constructed to analyze the gusset to-beam and gusset-to-column interface forces and a performance-based design method was proposed by (Lin et al., 2014). An investigation into net section rupture of stainless steel single angles connected by one leg to a gusset plate, with a single row of bolts was investigated in (Salih et al., 2013). And based on their results revised design equations for determining the net section capacity of stainless steel angles were proposed and their reliability were demonstrated by means of statistical analysis. An analytical investigation was conducted on the condition of the U10 gusset plates at the time of bridge collapse by (Liao et al., 2011). Compression tests and finite element analyses for steel dual-gusset-plate connections used for buckling-restrained braced frames were conducted by (Chou et al., 2012). The possibility and effectiveness of using a recently developed relative displacement sensor for the damage detection of gusset plate conditions in steel truss bridges was investigated by (Li and Hao., 2015). The buckling behavior of steel gusset plates in greater detail, accounting for parameters that were not explicitly included in the guidelines such as initial deformations of the gusset plate, stiffness of the framing members, and load distribution from the framing members to the plate was examined by (Crosti and Duthinh, 2010). An improved method to calculate the buckling strength of gusset plates using variable stress trajectory angles was described in (Dowswell, 2014). A nonlinear finite-element analysis approach using OpenSees for load rating gusset plates in steel structures was presented by (Walker., 2014).

(Higgins et al., 2013) performed experimental tests that focused on sway buckling mode of gusset plates. Their test variables included plate thickness, compression diagonal flexural stiffness, initial out of plane imperfection and member load combinations. A triage evaluation procedure was proposed to enable rapid yet conservative identification of gusset plates that may be yielding under service loads and to eliminate gusset plates with adequate capacity from needing further or more sophisticated analysis by (Berman et al., 2012). (Hu., 2012), examined design strength models on the basis of feasible failure patterns for the gusset plates, thereby calculating their strength capacities. (Higgins et al., 2010), identified the differences between rating outcomes from block-shear analysis at strength conditions with designs that employed allowable stresses on the Whitmore section method and proposed expected outcomes for rating of gusset plate connections. A screening process and a simplified rapid screening process were proposed for ranking gusset plate connections in steel truss bridges to help bridge engineers identify possible vulnerable connections and aid field inspections in (Higgins et al., 2010).

In this study, the buckling behavior of steel gusset plates with different out of plane imperfection, different slenderness ratios, and consequently different element sizes were investigated. Another major parameter considered in the analysis was the different fastener (bolt) locations. Compression loads were applied to the steel gusset plate members. Slenderness ratios of the steel gusset plates ranged from 50 to 250. The parameter that controlled the slenderness ratio was the length of the plate. Out of plane displacement and the displacement at the centroid of the loaded members of the steel gusset plates were examined. In addition, ultimate failure loads of the given gusset plate were normalized with the critical buckling load for the corresponding gusset plate section. An automated finite-element analysis program using OpenSees (Scott et al., 2008; Mazzoni et al. 2006) was used to analyze the gusset plates. A36 steel (steel with yield stress of 36 ksi or 248.2 Mpa) mechanical properties were defined into the finite-element analysis program.

2. Method and Software (Higgins et al., 2016)

Buckling analysis under compression loads of the gusset plates were carried out using a finite element software (Higgins et al., 2016 ; Scott et al., 2008). A three-dimensional (3D) finite element model was developed from the plate corner locations and fastener locations of the steel gusset plate in this software. The material model, finite element model information, information on the load combinations used in this paper are presented below.

2.1. Material Model

The steel stress-strain response in this software was simulated using a J_2 plasticity model with linear strain-hardening. This model is shown in Figure 3. (Higgins et al.,2016).

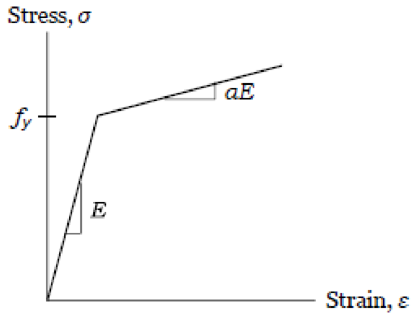

Figure 3. Uniaxial stress-strain response of the J_2 plasticity model

2.2. Plate Stress Condition

The stress-strain relationship of the steel gusset plates was defined with the following equations in three dimensions. σ_{33} (the stress in the out of plane direction) was considered as 0 in the equations given below.

$$\sigma = [\sigma_{11}\ \sigma_{22}\ \sigma_{33}\ \sigma_{44}\ \sigma_{55}\ \sigma_{66}]^T \tag{1}$$

hence the strains can be presented as

$$\varepsilon = [\varepsilon_{11}\ \varepsilon_{22}\ \varepsilon_{33}\ \varepsilon_{44}\ \varepsilon_{55}\ \varepsilon_{66}]^T \tag{2}$$

The stress strain relation can be written by using the elasticity modulus E and Poisson ratio v, as

$$C = \begin{bmatrix} E/(1-v^2) & vE/(1-v^2) & 0 & 0 & 0 \\ vE/(1-v^2) & (1-v^2)/E & 0 & 0 & 0 \\ 0 & 0 & G & 0 & 0 \\ 0 & 0 & 0 & G & 0 \\ 0 & 0 & 0 & 0 & G \end{bmatrix} \tag{3}$$

where G is the shear modulus and can be defined as

$$G = E / 2(1+v) \quad (4)$$

More information on the plasticity model and plane stress formulation of the model can be found in (Higgins et al., 2016 ; Simo, Hughes 1998). Mesh of MITC4 shell elements (Dvorkin , Bathe, 1984), were used in the software.

2.3. Imperfection and Loads

The plate was given an initial out of plane imperfection (deformed shape) based on eigenvalue analysis of the initial stiffness matrix (**K**), of the gusset plate finite element model. Imperfection (deformed shape) was based on eigenvalue analysis of the initial stiffness matrix (**K**) of the gusset plate finite element model. The eigenvector, φ, associated with the smallest eigenvalue (λ), that both satisfy the eigenvalue equation is chosen for the deformed shape.

$$K\varphi = \lambda\varphi \quad (5)$$

The member loads were applied to the gusset plate in two ways. The first loading is to centroid, which is given in Figure 4a. And the second loading is spread to loads option, given in Figure 4 b.

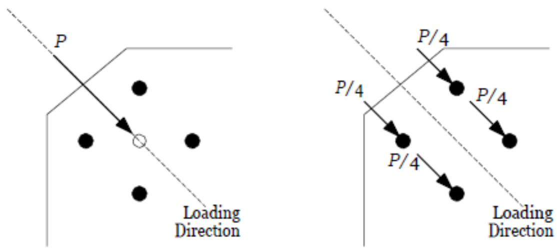

Figure 4. Gusset Plate Loading Cases (Higgins et al., 2016)

The compression loads were applied to the gusset plate incrementally with 20 kip (89 kN) increment. Please note that rectangular steel gusset plate cross sections were used in this study.

2.4. Finite Element Model

The software performs nonlinear static analysis of the gusset plate model. The system of nonlinear equilbrium equations is solved at each load step using a Krylov-subspace accelerated Newton algorithm. The finite element software was verified by using experimental data and real data in (Scott and Fenves, 2010; Higgins et al., 2016). More information on the finite element model used in the software can be obtained in these studies.

3. Analytical Example

The material properties of the steel gusset plate were chosen as; Young's (elasticity modulus) of the steel: 29,000 ksi (200,000 Mpa), yield stress of the steel: 36 ksi (248.2 Mpa), poisson ratio (v) : 0.3 and strain hardening ratio: % 1. For the single plate case, plate thickness was 0.5 inch (12.7 mm). One bolt row in each group (upper and lower parts of the cross section) and two bolt rows in each group cases were analyzed. Rectangular steel gusset plate cross sections were considered in this paper. Different slenderness ratios were taken into account in the analysis. The slenderness ratio λ of the plate can be defined as

$$\lambda = k\, L/r \quad (6)$$

where k is the effective length factor, whose value depends on how the ends of the column/plate are fixed. L is the length of the plate and r is the radius of gyration and can be written as

$$r = I/A \quad (7)$$

I is the moment of inertia and A is the area of the cross section of the gusset plate. The slenderness ratios (λ) considered in this study for one row of bolts in each group were; 75, 100, 150, 200 and 250. Similarly for two rows of bolts in each group case, the λs were; 100, 150, 200 and 250. The dimensions of the cross section for one bolt case were (30.48 cm- 32.51 cm; 30.48 cm- 41.91 cm; 30.48 cm- 59.94 cm; 30.48 cm- 78.23 cm). The example loading and schematic representation of an example steel gusset finite element model, with one row of bolts in each group is shown in Figure 5 . In one row of bolts case there are 13 bolts in upper and lower parts of the cross section. And a total of 26 bolts in the whole cross section.

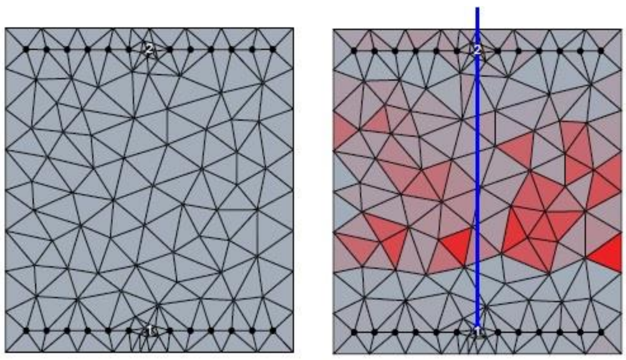

Figure 5. Schematic Representation of Steel Gusset Plate with One Row of Bolts in Each Group

The schematic representation of another finite element model of a steel gusset plate cross section with two rows of bolts in each group and an example loading case is shown in Figure 6. There are 22 bolts in lower and upper parts of the cross section in this case. This resulted in a total of 44 bolts in whole cross section. The example steel gusset plate shown in

Figure 6 has a λ of 50. The dimensions of the cross section for two bolts case were (30.48 cm- 62.48 cm; 30.48 cm- 88.90 cm; 30.48 cm- 114.81 cm; 30.48 cm-140.97 cm).

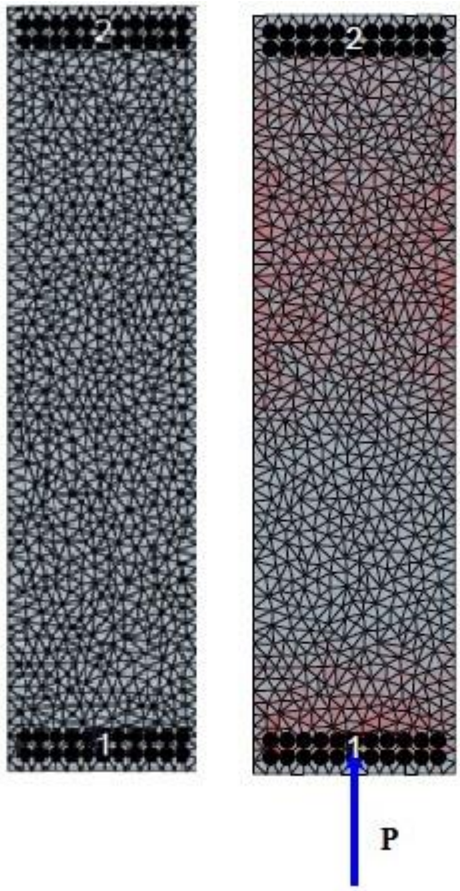

Figure 6. Schematic Representation of Steel Gusset Plate with Two Rows of Bolts in Each Group

The change in the imperfections of the gusset plate were considered in the analysis as well. Imperfection percentage was defined (Ï %) with the following formula

$$\text{Ï}(\%)= 100*(\Delta_0/t) \quad (8)$$

where Δ_0 is the initial imperfection and t is the thickness of the plate. For the one bolt row in each group case diiferent imperfections from %10 to %100 were analyzed. Two rows of bolts in each group case, imperfections of % 10 to %100 with ten percent increment were considered. Another parameter considered in the steel gusset plate analysis was the normalization of the ultimate failure loads of the given gusset plate, with the Euler's critical buckling load for the corresponding gusset plate section. This ratio can be defined as given below. This ratio is named as ultimate load ratio in the future context of this paper.

$$P(\%)= P/P_{cr} \quad (9)$$

where Pcr is the critical buckling load and P is the ultimate failure load obtained during the analysis. The Euler's critical buckling load P_{cr} can be obtained as

$$P_{cr} = \pi^2 EI/(kL^2) \quad (10)$$

by using slenderness ratio (λ) formula, given with Equation 6 and radius of gyration (r) formula given with Equation 7, the critical buckling load can be rewritten as

$$P_{cr} = \pi^2 E/(\lambda^2) \quad (11)$$

The results obtained from the analysis are given in the next section. However, the discussion on the results are presented in the conclusion section.

4. Results

For the steel gusset plate with one row of bolts in each group with different imperfection and slenderness ratios, the ultimate load ratios given with Equation 9, are shown in Figure 7. The imperfection ratios range from 10 to 100 in Figure 7. The ultimate load ratios for two rows of bolts in each group are given in Figure 8.

The centroid displacements with respect to force for two rows of bolts in each group case are shown in Figure 9. Figure 9 was composed for λ=100. Figure 9 was produced with kips and inch units. The maximum force is at the extent of 10 kips (44.5 kN) for an imperfection of %100. The maximum centroid displacement for this case was around 1.8 inch (45.7mm) if we consider the imperfection ratio of % 100 case. Force displacement curves for different imperfection values are given in Figure 10 for λ=250.

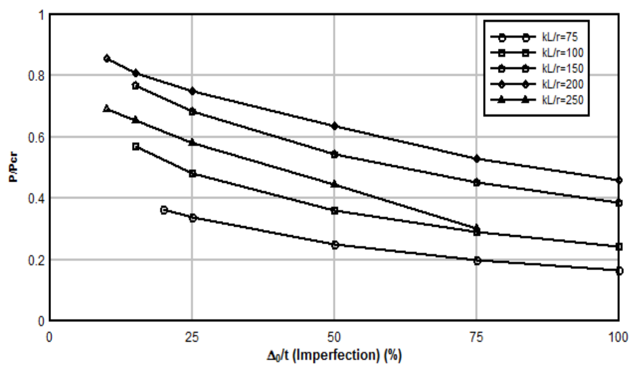

Figure 7. Ultimate load ratios one row of bolts case

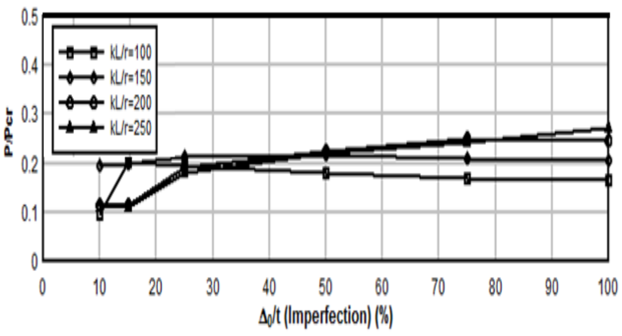

Figure 8. Ultimate load ratios of two rows of bolts case

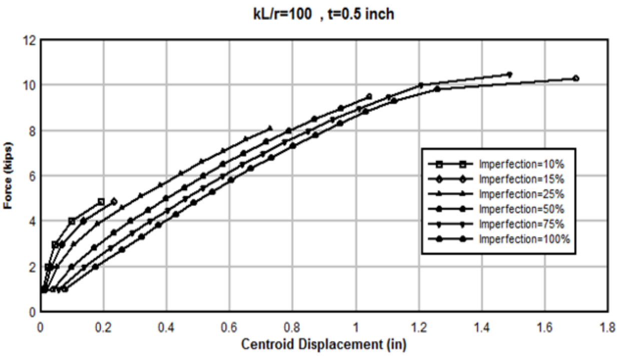

Figure 9. Force vs. centroid displacement for two rows of bolts case λ=100.

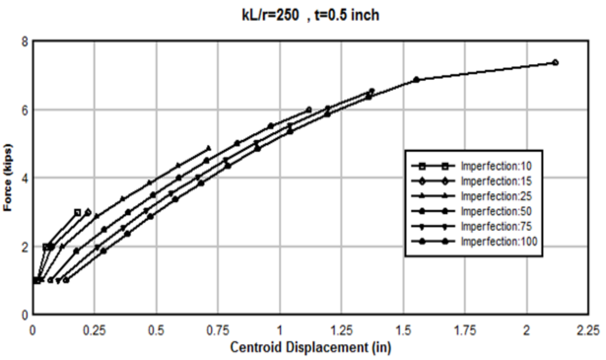

Figure 10. Force vs. centroid displacement for two rows of bolts case λ=250.

5. Conclusion

The normalization of the ultimate failure loads with the critical buckling load in gusset plates was defined and analyzed in this paper. Variation of this normalization with respect to the change in the steel gusset plate parameters was investigated. These parameters were; the imperfections, slenderness ratios, and bolt locations of the steel gusset plates. A finite element software was used for the analysis of steel gusset plates under compression loads. It was obtained that an increase in the imperfections resulted in an increase in the displacement capacity of the gusset plate, for both two rows of bolts and one row of bolt cases. It was also found that imperfection of %100 case, in comparison with %10 case had almost 9 times higher displacement capacity than %10. This conclusion is valid for both slenderness ratio values of 100 and 250 and two rows of bolts case. In addition, increasing the slenderness ratios ended in higher ultimate load ratios as expected for all bolt cases. The two rows of bolts case had smaller ultimate load ratios than the one row of bolt case. The highest ultimate load ratio for one row of bolt case was found to be 3.2 times higher than the corresponding ratio of two rows of bolts case. Increasing the slenderness ratio in two rows of bolts case resulted in higher displacement capacity than the gusset plate with smaller slenderness ratio. For the two rows of bolts case, 2.5 times higher slenderness ratio of the gusset plate provided % 25 higher centroid displacement at the ultimate load capacity with the highest imperfection ratio. It was concluded that, imperfections have significant effect on the displacement capacity of the steel gusset plates under compression loads. Moreover, slenderness ratios have moderate effect on the displacement of gusset plate under compression loads.

For future studies experimental research on ultimate load ratios of steel gusset plates considering the different parameters used in this paper would be interesting.

Conflict of Interest

No conflict of interest was declared by the authors.

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