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Modeling nonlinear behavior of gusset plates in the truss based steel bridges

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Abstract. The truss based steel bridge structures usually consists of gusset plates which lose their load carrying capacity and rigidity under the effect of repeated and dynamics loads. This paper is focused on modeling the nonlinear material behavior of the gusset plates of the Truss Based Bridges subjected to dynamics loads. The nonlinear behavior of material is characterized by a damage coupled elsto-plastic material models. A truss bridge finite element model is established in Abaqus with the details of the gusset plates and their connections. The nonlinear finite element analyses are performed to calculate stress and strain states in the gusset plates under different loading conditions. The study indicates that damage initiation occurred in the plastic deformation localized region of the gusset plates where all, diagonal, horizontal and vertical, truss member met and are critical for shear type of failure due tension and compression interaction. These findings are agreed with the analytical and experimental results obtained for the stress distribution of this kind gusset plate.

Keywords: truss based steel bridges; gusset plate; dynamic analysis of bridges; nonlinear modeling

1. Introduction

The truss based steel bridge structures are known as lightweight and low cost structures that have been extensively used for the construction of infrastructure and transportation networks. Considering the large number of bridges in services and the common use of the truss systems in buildings and facilities these structures allow to transmit load back to their foundations with a more appropriate manner. This type of bridge usually consists of gusset plates which lose their load carrying capacity and rigidity under the effect of repeated and dynamics loads. The presence of this gusset plates has an appreciable effect on the stiffness of the members of the bridge and consequently on its behavior to static and dynamic loading. In their functionality the gusset plates are the integral to a truss-based bridge because they serve as the attachment point for the truss members. Especially this joint flexibility may cause to alter the vibration characteristics of the bridge system.

Therefore, the analysis of the gusset plates becomes ever more critical as these structures are nearing the end of their design life. Especially, after the tragic collapse of the bridge carrying

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Interstate Highway I-35W over the Mississippi River in Minneapolis in 2007 (Nakamura and Simulia Corp. 2008), the gusset plates have become the target of many research studies including laboratory test, theoretical studies and finite element methods.

These earlier studies focused mainly on gusset plate connections designed for tension and/or compression behavior, where the stress distribution in such gusset plates differs substantially from that in simple tension or compression connections, and varies with member arrangement. Some of the previous research had been done specifically to evaluate locations and magnitudes of stress in gusset plates, and to derive a simple way to determine maximum stresses for designing these structural members. One of the earliest works performed by Wyss (1923) was the experimental work on determination of the stress distribution in the gusset plates of a warren truss joint. Wyss noted that the maximum normal stress usually appeared at the end of the brace member and the stress trajectories were spread out along a line making angle of 30° with the connected member.

Whitmore (1952) performed a series of and experimental study for the determination of the stress distribution in the gusset plates. He tested 1/8 in. aluminum gusset plates having a yield strength of 39 ksi and a modulus of elasticity of 10,000ksi. Whitmore found out that the maximum tension while compression stresses were located around the ends of the diagonal members whereas the maximum shearing stresses were located near the chord member and toward the center of the plate and the edge of the plate experienced much lower stresses. Finally he developed the effective width by constructing lines making 30° with the axis of the connected member. In the experimental study of Vasarhelyi (1972) the photoelastic tests were conducted on the thick gusset plate of A36 steel and analytical solution were made to find the stress distribution. He found the approximately similar result as in Whitmore's work about the locating the maximum stress distribution on the gusset plate. Astaneh (1992) conducted the experiments on the gusset plate of A36 steel. The finding of the experimental study was that the one of the specimens was failed due shear yielding. As a result of this the plastic stress distribution was used to calculate the horizontal shear capacity of the plate. However, Dietrich (1999) presented the results of the cyclic tests performed on the gusset plate connections of A36, which indicated that the failure mode was due to fracture along the Whitmore effective width.

In current practice, gusset plates are designed for the stresses at the Whitmore section, block shear in tension, buckling in compression, and stresses at critical sections calculated based on simple beam equations, in addition to constructability considerations. However, it is seen that the simple design methods based on equilibrium and elastic behavior are adequate for design it gives crude approximation in calculating the actual stress distribution in a gusset plate, even in the elastic range. Therefore, numerous finite element studies on gusset plates over the last three decades have been performed in order to better understand local stress distribution, buckling in compression, stability, or seismic response of gusset plates.

Yamamot *et al*. (1985) performed the experimental and finite element analyses for a number of Warren and Pratt type trusses. The development of plasticity, local bucking, and failure in the gusset plate connection specimen were examined. It was observed that the initial yielding occurred in the inner part of the gusset plat at the earlier stage of loading and with the increase of loading the plastic region evolves towards outer part of the gusset plates.

Huns *et al*. (2006) performed a finite element model analyses in order to predict the tension and shear block failure of gusset plates. They have also conducted a reliability analysis of existing test results to evaluate current design equations. According the results of their analyses tension fracture always occurs before shear rupture and the gusset plates reach its full capacity before the rupture occurs. The result of their reliability analysis suggested using the equations proposed by Hardash and Bjorhovde (1984), Driver *et al*. (2004) that provide a good prediction of the test results of the gusset plates instead of using the equations in the design standards. Because they thought that design standards were more conservative and in some cases did not predict the failure mode accurately. In the experimental work of Hardash and Bjorhovde (1985) where they tested a series of gusset plates loaded in tension, the following finding were listed as a result of their works; i) the gusset plate specimens was failed due to tensile fracture across the last row of bolts, ii) shear yielding/tearing along the bolt lines was parallel to the tensile force, iii) the failure mode due to shear yielding were viewed as block shear failure, and iv) the compressive strength of gusset plates may was assumed to be governed by buckling.

After the tragic collapse of the bridge carrying Interstate Highway I-35W over the Mississippi River in Minneapolis, Minn.in 2007, many reports and investigations are made on the gusset plate connections in order to find the main factors that played a key role in the collapse of the bridge (Myers 2009, Liao *et al*. 2009, 2011). These studies concluded that insufficient strength of the gusset plate that plays important role to the yielding and the fracture of the substantial portion of the gusset plate is the key factor of collapse. Another important finding of these studies is to indicate that the interaction of compression and shear played an important role in the gusset plate failure and should be taken into account in gusset plate design.

In order to have better understanding of the behavior of the gusset plate subjected to combined loads many researchers have used finite element modeling to investigate how these connections behave. Li *et al*. (2007) conducted a research on the behavior of the gusset plate connection based on multi-scale numerical analysis on long-span bridges, focused on local damage and dynamic responses in such bridges. They used the stiffening truss of a suspension bridge in China as a case study. This study concluded that such multi-scale modeling was necessary for the evaluation of long-span bridges and the effects of damage on them.

Liao *et al*. (2009) made an extensive computational study on the investigation of the I-35W Bridge Collapse. This report describes detailed nonlinear, three-dimensional finite element models to calculate stress and strain states of the gusset plates. Their results of the detailed finite element analyses in explaining possible major reasons causing the catastrophic failure of the I-35W Bridge Collapse are as follows; i) substantial portions of the gusset plates were yielded at the time of collapse, ii) the substantial yielding was due to insufficient strength of the gusset plate, along with weight increase due to the past deck reconstruction and construction material and the equipment staged on the day of collapse, and iii) the interaction of compression and shear played an important role in the gusset plate failure.

Myers (2011) focused on the investigation of stresses created in the gusset plates by various types of live loading. The results are compared to the Method of Sections approach recommended by FHWA following the I-35W Bridge collapse to determine if better analysis specifications are needed. Although the results of the finite element analysis and the Method of Sections approach are similar, the authors conclude that the value of the Method of Sections approach is strongly dependent on the accuracy of the load data input.

Ganz (2012) studied to determine if carbon fiber gusset plates offer superior strength characteristics compared to those made from structural steel by performing finite element analyses of a single span Warren Truss bridge using Abaqus. Performance of the two materials was evaluated based on failure margin and deflections. Ganz concluded that carbon fiber plates do not offer a performance advantage versus steel based upon failure margin and deflection.

Crosti and Duthinh (2014) proposed a model for describing nonlinear behavior of the connection. In their approach the gusset plates were modeled by five user- defined semi rigid springs, that each has a full 6 by 6 stiffness matrix which was derived by applying forces and moments to the free end of each portion of member. However, their proposed method focused on only a specific joint that need be investigate in detail

Considering the large number of steel truss bridges in service, and the common use of the truss system in buildings and facilities, further understanding of the behavior of the gusset plate connections is still essential since this interaction is not well understood from previous research. Standard design methods do not explicitly account for the interaction. Another important effect of this interaction mechanism at the gusset plate connections can be seen on the material response of the plates where substantial yielding and the fracturing occur due to combined loading resulting from these interactions. However, most of the work reviewed above focused on the structural behavior of the elastic gusset plate and there have been a little effort put forward to characterize the material behavior of the gusset plate Therefore, the main objective of this study is to describes a computational model that takes into account the material characterization and is suitable for large scale finite element simulation.

2. Material model

2.1 Theoretical framework

In order to take into account of yielding and fracture of the gusset plate in the computational model the material behavior of the gusset plate is characterized by the elastoplastic and ductile damage model. Thermoelasto-plastic model is characterized in effective stress space with the principle of strain equivalence principle (Lemaitre 1996). Within an effective configuration, the elastic domain is described by von Mises yield criterion and the isotropic hardening rules which describe the change in the size of the yield surface during plastic deformation. This yield surface is given as

$$f=\left(\frac{3}{2}\bar{\tau}_{ij}\bar{\tau}_{ij}\right)^{1/2}-\sigma_{yp}-R(p)\leq 0 \tag{1}$$

where $\sigma_{(yp)}$ is the initial yield value obtained from simple uniaxial test, $R(p)$ is the isotropic hardening function in terms of accumulated plastic strain, p and $\bar{\sigma}_{ij}=\bar{\sigma}_{ij}-(1/3)\bar{\sigma}_{kk}\delta_{ij}$ is the deviatoric component of the effective Cauchy stress tensor, $\bar{\sigma}$ which is defined as

$$\bar{\sigma}=\frac{\sigma}{1-D} \tag{2}$$

where D is the damage variable representing the surface density of the intersections of microcracks and microvoids in the material body. For the undamaged material D equals to zero and for the fully damaged material D becomes one. However, the damage variable never reaches to a value one due to atomic decohesion that causes material breaks before D is equal to one. Therefore within the continuum damage mechanism there is no need to have predefined defects in order to predict fracture.

The evolution of the plastic flow and the isotropic hardening rule are derived from the maximization of the dissipation energy of irreversible material within the framework of thermodynamic.

$$\dot{\varepsilon}_{ij}^{p} = \dot{\lambda}\frac{\partial f}{\partial \bar{\tau}_{ij}} = \frac{3}{2}\frac{\dot{\lambda}}{1-D}\frac{\bar{\tau}_{ij}}{\left(\bar{\sigma}_{ij}\right)_{eq}} \tag{3a}$$

$$\dot{R} = b\dot{p}e^{-\mu p} \tag{3a}$$

$$\dot{p} = \sqrt{\frac{2}{3}\varepsilon_{ij}^{p}\varepsilon_{ij}^{p}} = \frac{\dot{\lambda}}{1-D} \tag{3b}$$

where b and μ are the material constant and $\dot{\lambda}$ is the Lagrange multiplier than can be obtained from consistency condition, $\dot{f} = 0$

$$\dot{\lambda} = \frac{1}{H}\frac{\partial f}{\partial \bar{\sigma}_{ij}}E_{ijkl}\dot{\varepsilon}_{kl} \tag{4}$$

The effective the elasto-plastic tangent stiffness is defined by the rate relation such that

$$D_{ijkl} = E_{ijkl} - \frac{1}{H}E_{ijpq}\frac{\partial f}{\partial \bar{\sigma}_{pq}}E_{klmn}\frac{\partial f}{\partial \bar{\sigma}_{mn}} \tag{5}$$

and H is given by

$$H = \frac{\partial f}{\partial \bar{\sigma}_{ij}}E_{ijkl}\frac{\partial f}{\partial \bar{\sigma}_{kl}} + b(1-R) \tag{6}$$

where hardening E_{ijkl} is the fourth order elastic stiffness tensor.

In the literature many ductile fracture models were proposed however, only few of these models are well accepted by research community of the field and available in material libraries of leading finite element codes. One of the most commonly used fracture model, the Johnson-Cook fracture criterion (Johnson and Cook 1985) is used here to characterize fracture of the gusset plate. The Johnson-Cook model is a cumulative-damage fracture model that takes into account the loading history, which is represented by the strain to fracture. In this model the damage evolution is defined as

$$\dot{D} = D_c\frac{\dot{p}}{p_f - p_d} \tag{7}$$

where p_d is the damage threshold and D_c is the critical damage that controls fracturing condition of the element, i.e., fracture occurs if D_c reaches a specified value less than one. p_f is the fracture strain in terms of stress triaxiality and is given by

$$p_f = D_1 + D_2 e^{(D_3\sigma^*)} \tag{8}$$

2.2. Identification of material model parameters

ASTM A36 low carbon steel is specified for the gusset plates and truss members. Material constants of both plasticity and damage models used to characterize elasto-plastic behavior of the gusset plates are obtained by fitting the model equations to the available experimental data (Borvik *et al*. 2001, Teng and Wierzbicki 2006, Jutras 2008). For example parameters of the presented plasticity model are obtained by fitting uniaxial stress strain curves obtained from the simulation on the uniaxial tension model established within Abaqus to the available experimental data in the literature (Fig. 1(a)). Similarly, the parameters of the Johnson Cook fracture model are obtained from the available tests which builds the exponential curve of the strain to fracture in function of the stress triaxiality, and find the corresponding model parameters fitting the Eq. (8) to this curve by using nonlinear least square algorithm (Fig. 1(b)).

The calibrated model parameters of the models, plasticity and damage, respectively are summarized in Table 1.

3. Finite element model of a Warren truss bridge

3.1 Details of bride design

A generic Warren Truss Bridge consisting of plates and trusses is chosen in this work as a benchmark problem in order to predict material behavior of its gusset plates by means of comparing of bridge responses under different loading conditions. Fig. 2 shows the geometric representation of this generic bridge model where the length of bridge is assumed to be 120 ft and its height is taken as 20 ft. Diagonal truss members are assumed to be inclined with angles of 45^{o} and 135^{o}, respectively. The length of the horizontal, vertical and diagonal trusses are assumed to be 15.83ft., 16.67 ft and 24.62 ft respectively and all truss members have a cross section of 80 in^2. In

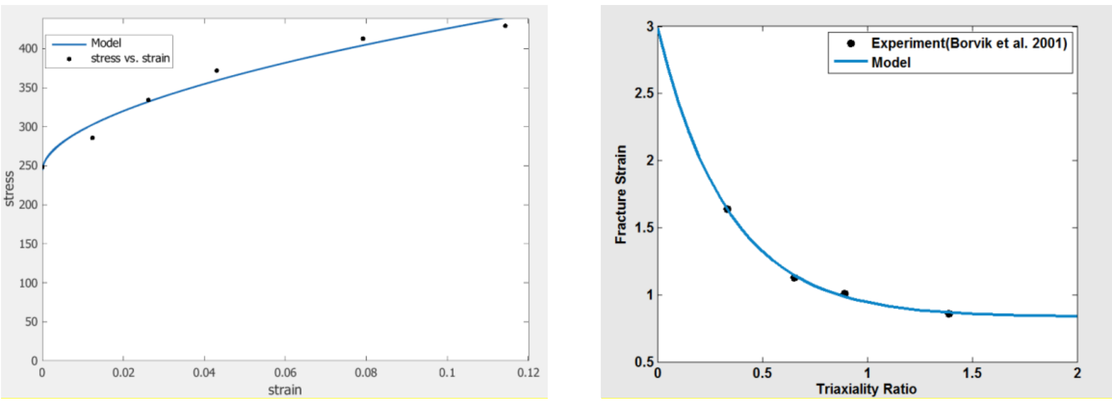

Fig. 1 (a) Fitting plasticity model to experimental stress strain curve (b) Fitting damage model to fracture strain- triaxiality ratio (Borvik *et al*. 2001)

Table 1 Material model constants for A36 steel

Elastic constants and density			Yield stress and hardening			Damage parameters			Damage evolution	
$E(ksi)$	v	$\rho(lb/in^3)$	$\sigma_{yp}(ksi)$	$b(ksi)$	μ	D_1	D_2	D_3	D_c	p_d
29,000	0.26	0.282	41.40	72.54	0.228	0.834	2.15	2.95	0.3	0

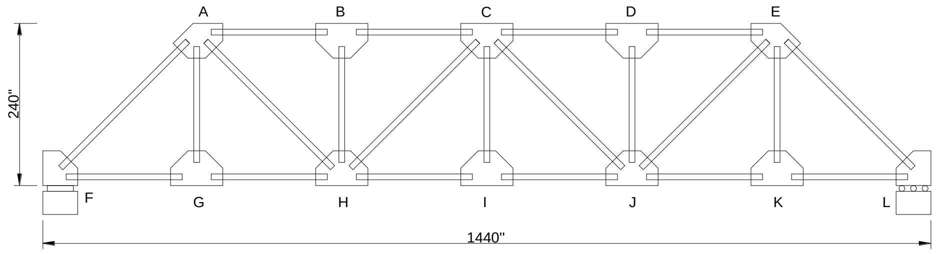

Fig. 2 Geometric dimensions of the Warren type bridge truss (Ganz 2012)

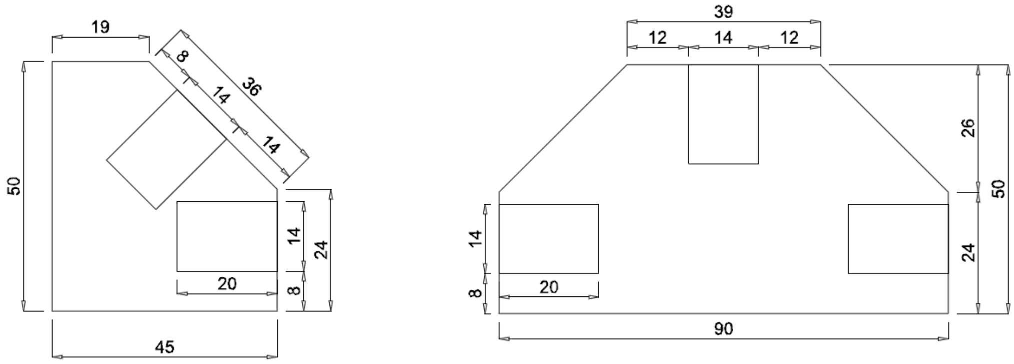

Fig. 3 Geometrical details of two types of gusset plates used at the joints, F, L, G-K and B-D

order to govern overall deflections of the model by the cumulative deflections of the gusset plates a robust truss thickness which prevent excessive bending, buckling, or deflections of the truss members is assumed to be 12 in.

Three major types of gusset plates such as the bottom ends, the top ends and the mid-span plates are used as connection members of this bridge. The dimensional details of two types are given below (Fig. 3).

During the establishment of this generic Warren Truss Bridge the procedures described in the works of Najjar *et al*. (2010), Ganz (2012) are followed. The reason to follow the approach described in those references is that this bridge model is a pretty close representation of the most typical railroad bridges.

The design of this bridge and its gussets plates are made according to the code and guidelines specified in the AISC Specification for Structural Steel Buildings (AISC 2005) and Federal Highway Administration Bridge Design Guidance. Particularly, for the design of the gusset plates the provisions of AASHTO in accordance with Load and Resistance Factor Design Method (LRFD) are followed (Astaneh *et al.* 1989, Astaneh Asl 1992, 1998, Ibrahim 2008).

In the current design practice gusset plates are usually designed in a way that the stresses on any cross section of the plate do not exceed the capacity limits described in code and guidelines. The most crucial part in the gusset plate design is to selection of the most highly stressed region. The decision on the selection of this critical section is mostly based on judgment and experience. Therefore, reliable finite element analyses proving detailed stress distributions on the gusset plate will help the designers to make right decision on the selection of this critical section.

In this work for the design of the bridge, the loading conditions are first determined including

Table 2 Resistance summary of the gusset plate design

Location	Axial Resistance of the Gusset Plate (Kips)				Controlling Axial Resistance $C=0.9x\emptyset Rn$ (Kips)	Inventory Rating Factor $\frac{C-0.74\,DL}{1.24\,LL}$
At the end of member	Gross Section Yielding in Tension	Net Section Fraction in Tension	Block Shear Rapture	Compression Buckling		
I	1003.89	1362.08	860.72	-	774.648	1.73
B	-	-	-	910.711	819.64	2.21

Location	Orientation of the section	Shear Resistance of the Gusset Plate		Controlling Shear Resistance $C=0.9x\emptyset Rn$	Inventory Rating Factor $\frac{C-0.65\,DL}{1.24\,LL}$
At the end of member		Gross Section Yielding in Shear	Net Section Fraction in Shear		
H	Vertical	642.19	1177.4	577.9	3.46
	Horizontal	1155.943	2119.32	1040.35	6.1

dead load, dynamic load such as wind and live load (vehicles and snow) based on building requirements in accordance with AISC (2005). The calculated values of these loads are given as; 351,146 lbs for dead load including the weight of the bridge sections, sidewalks, asphalt, and roadway, 279,436 lbs. for live considering the weight of passing vehicles and snow. The value of total load (W) acting on this bridge is found to be 630,581 lbs as the combination of dead load and live load. This total load is then distributed evenly to apply one fifth of the total load at the joints and the member forces are found. These values are then used for the design of the gusset plate which are treated as axially loaded members with a cross section $L_w x\ t$. Here L_w is the effective width and t is the thickness. The calculation effective length is based on the Whitmore Section analysis where it is assumed that the trajectory of the maximum normal stress spreads through the gusset plate at an angle of 30^o with the axis of the connected member.

The design check of a critical section of the gusset plates found to be at the joint I and B are performed following the guidelines of AASHTO in accordance with LRFD specifications (Ibrahim 2008). These finding are summarized in Table 2.

According to this the design check on the specified critical section of the gusset plates it is found that the controlling inventory factor is 1.73 for block shear rapture at the end of the member of gusset plate I.

3.2 Finite element model

The finite element model of the designed Warren Type Truss Bridge is established within Abaqus 6.9 (2009) as shown in Fig. 4. Finite element model used consists of shell elements having a total number of 22329 nodes and 19193 quadrilateral elements of type S4R of Abaqus Library. Reduced integration shell elements with enhanced hourglass control are chosen for the reason that the S4R element uses a reduced integration rule with one integration point that makes this element computationally less expensive than S4 and uses enhanced hourglass control to reduce artificial strain energy in contrast to total strain energy of the explicit analysis.

In the finite element model member connections at the joints are assumed to be welded type and tie constraints are used to describe welded connection in the finite element model. Boundary conditions are assigned at the two end plates where simple support condition is given to the left end

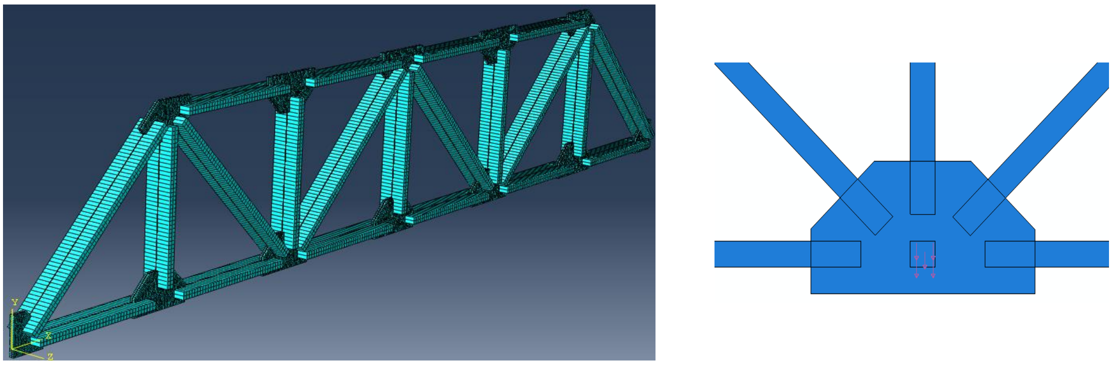

Fig. 4 Abaqus finite element model for a Warren truss bridge

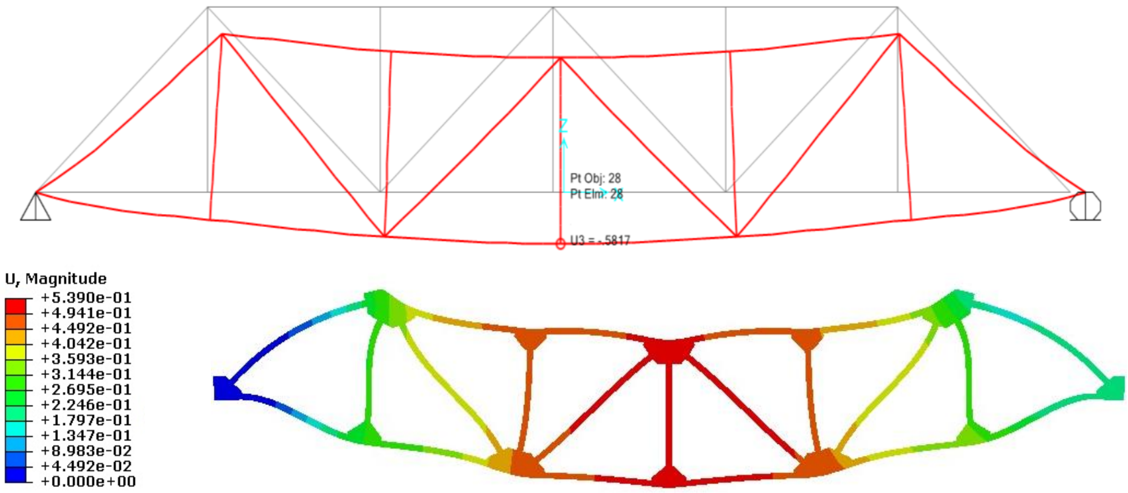

Fig. 5 Deflection of the truss bridges from static linear analyses of both SAP2000 and Abaqus models

bottom plate constraining its displacement in the *x* and *y* directions and the roller support condition is assigned to the right end bottom plate constraining its displacement in the *y* direction. Rotational degrees of freedom of these supports are left unconstrained to simulate a simple support condition.

4. Analyses and discussions

The structural models are analyzed under different types of loading conditions including, static, modal dynamics, and nonlinear dynamic in order to investigate stress and strain distribution on the gusset plate element. The results obtained in term of von Mises stress, displacement, and plastic strain from each analysis is compared to each other.

In order to validate the adequacy and reliability of the established finite element model, a mesh sensitivity analyses is first performed to avoid mesh dependent results under the static loading condition. It is determined that a seed size of two gives stabilized results and increases modeling efficiency of the model. Then the result for the maximum deflection obtained from the static linear

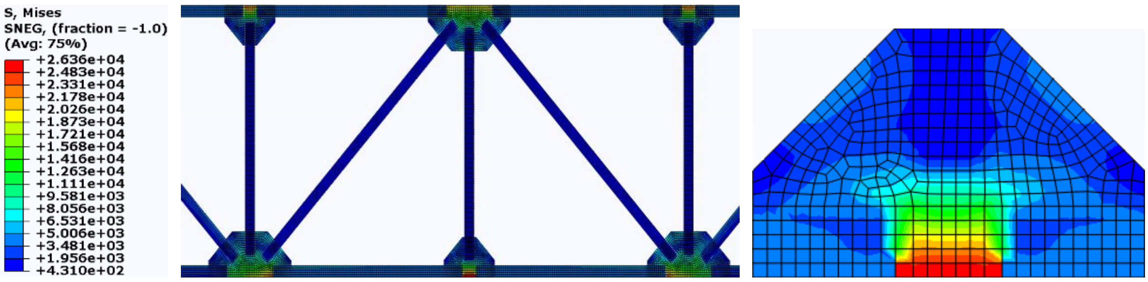

Fig. 6 Maximum von Mises distribution of the gusset plate obtained from the linear static analysis

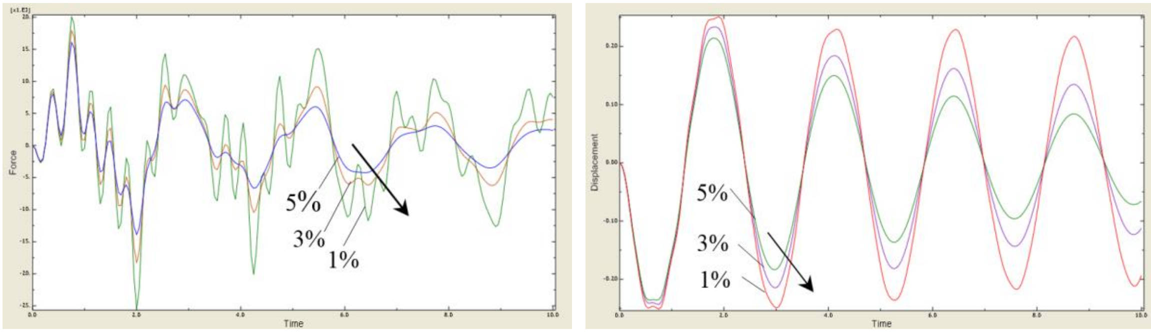

Fig. 7 Results of force and deflection versus time curves of the truss system obtained from the modal dynamic analysis

analysis of the Abaqus model is compared with the one facilitated by SAP2000 (Fig. 5)

As one can see from Fig. 5 that the value of the maximum deflection obtained from SAB2000 analysis is 0.5817 in whereas the analysis of Abaqus model gave this value to be 0.54in. This indicates that established finite element model by within Abaqus is quite accurate. For further testing for the reliability of Abaqus model, static linear analysis is performed in order to determine stress distribution on the gusset plates. Fig. 6 presents the result of the static analyses in terms the maximum von Mises stress distribution.

The maximum von Mises stress is appeared on the gusset plate I with a value of 26,368 psi which is less than both tensile yield strength and ultimate tensile strength of the A36 steel. This finding obeys the analytical results presented in section 3.1.

In order to see the effects of friction at the joints of the structure and localized material hysteresis modal dynamic analysis of the truss system is performed under the assumption of linear material behavior, no nonlinearity and geometric effects. Since these effects causes the dissipation of energy damping is chosen as a convenient way of including the important absorption of energy without the modeling the effects in detail. In the analyses direct modal damping that uses the fraction of critical damping, associated with each mode is used. In this simulation 1%, 3%, and 5% of critical damping are used. Fig. 7 compares the history of the reaction force at the pinned end support (point F) and maximum displacement of the midpoint.

As expected, the oscillations at lower damping levels do not diminish as quickly as those at higher damping levels, and the peak force is higher in the models with lower damping.

In the modal dynamic analysis, unlike to static analysis, the maximum won Mises stress with a value of a value of 11,573 psi is found to be appeared on the gusset plate J instead of I (Fig. 8).

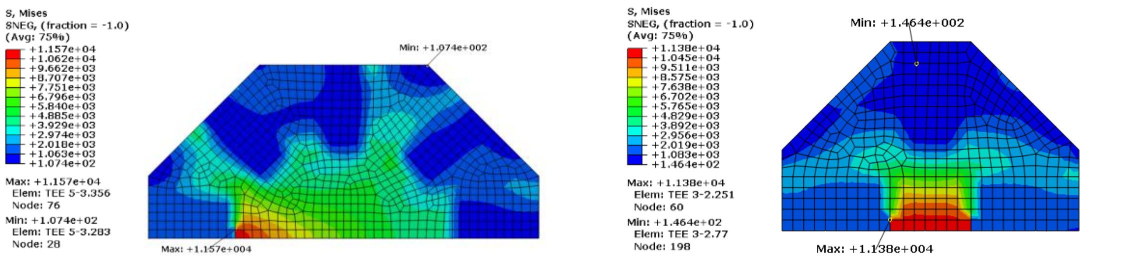

Fig. 8 von Mises stress distribution on the gusset plate obtained from the modal dynamic analysis

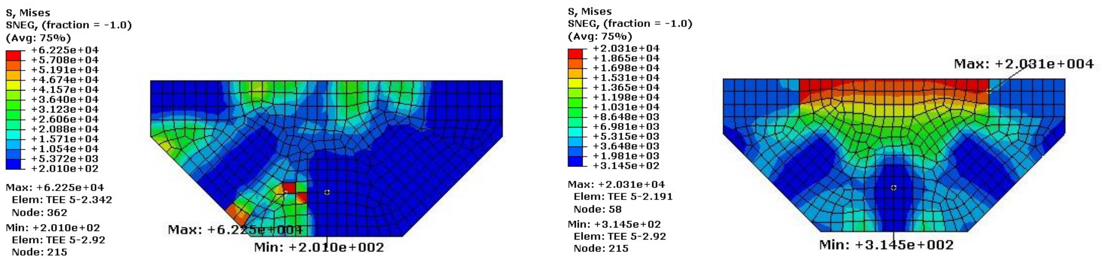

Fig. 9 von Mises stress distribution on the gusset plate obtained from the elasto-plastic damage analyses

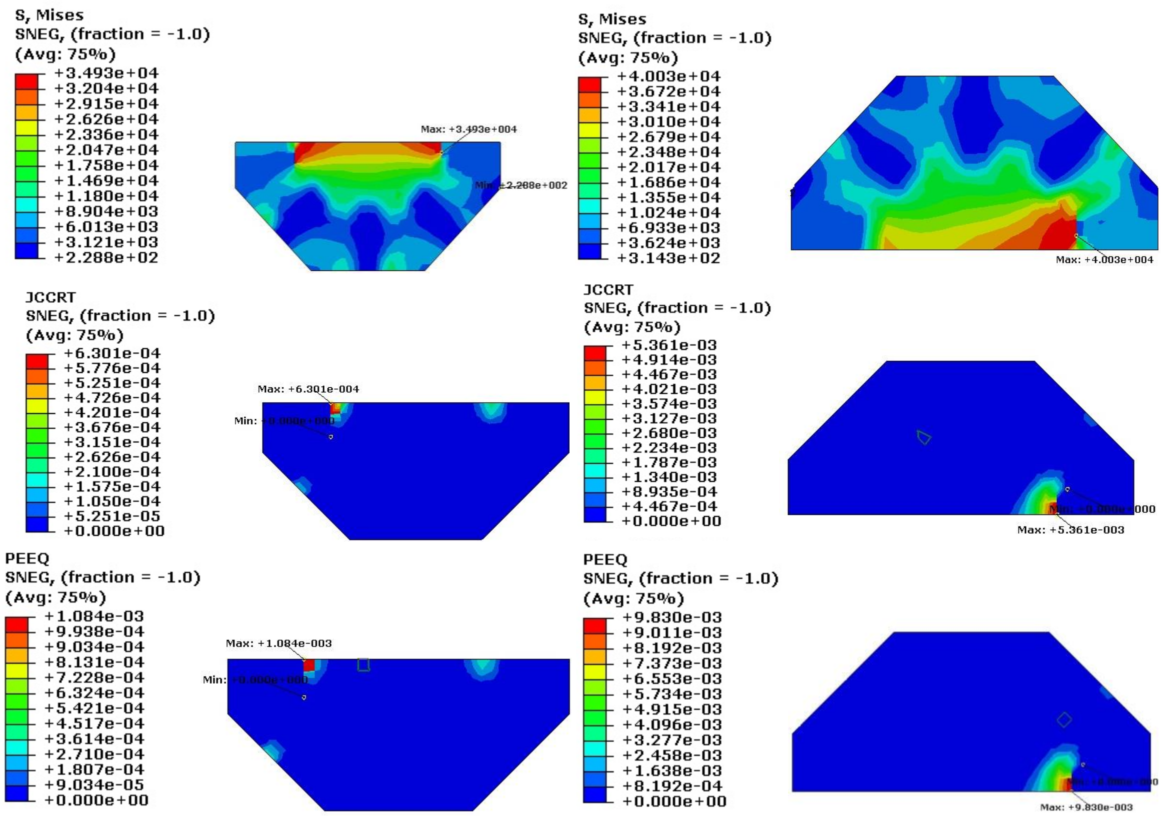

Fig. 10 von Mises stresses, equivalent plastic strain and damage initiation distribution results of the elasto-plastic damage model for the gusset plates C and H

It can also be seen that von Mises stresses of the modal dynamic analysis for the gusset plate I is almost 50% less than that of static analysis. This is an expected result since the frequency of the applied dynamic load is quite high as compared to the natural frequency of the structural systems where its minimum natural frequency is 2.76 rad/sec.

The explicit dynamic analyses are performed with the nonlinear damage coupled elasto-plastic material model. The maximum von Mises stress of the gusset plate C obtained from the damage coupled elasto-plastic analysis are compared to the results of the static analysis (Fig. 9).

As one can see from these figures, the stress distribution of the inelastic analysis is quite different than those in static one. The magnitude of the maximum von Mises Stress is found around 62,254 psi which is three times the one obtained from the static analysis. It is obviously higher than the tensile yield strength and ultimate tensile strength of the A36 steel. The results of explicit nonlinear dynamic analyses showed that the simulated maximum stress distribution usually appeared at the end of the member and along stress trajectories lines as described in the experimental and analytical studies. Fig. 10 presents the equivalent plastic distribution, damage initiation and won Mises stress distribution for the gusset plates C and H, respectively.

As one can see that the stresses are localized around the ends of the diagonal members where the tension and compression likely occur. Another observation is that the initial yielding is started in the inner part of the gusset plat at the earlier stage of loading and then the plastic region is evolved towards outer part of the gusset plates with the increase of loading (Fig. 10). This means that the damage initiation occurs in the von Mises stresses and the localized plastic deformation region of the gusset plates where all, diagonal, horizontal and vertical, truss member met. Therefore, one concludes that this region is critical for shear type of failure due to tension and compression interaction. These findings are agreed with the analytical and experimental results obtained for the stress distribution of this kind gusset plate.

5. Conclusions

The structural comparison of the Warren Truss bridge gusset plates under different loading conditions is performed using the elasto-plastic ductile damage model within the finite element framework of Abaqus. The analyses results indicated that elastic modeling itself is not enough the capture the accurate response of the gusset plate behavior and is not reliable to predict the material failure of the gusset plate subjected to dynamic loading. The stress distribution of the inelastic analysis is quite different than those in static one. The stresses are localized around the ends of the diagonal members where the tension and compression likely occur and shear plays an important role in the gusset plate failure. Therefore, one can conclude that the interaction between compression and shear played an important role in the gusset plate failure.

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