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Behavior of Gusset Plates Connecting Angle Sections in Compression

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Abstract

This paper contains experimental and numerical study on the behavior of the gusset plate in compression. Gusset plate connections are usually used for the transfer of load from one member to another, they are generally used in bridge trusses and steel braced frames. However, due to lack of technical information regarding the behavior of the gusset plate under compression, these have been under research focus. It has been seen that the gusset plate shows a buckling failure by swaying on one side as there is no out of plane restraint. In this paper, a study is initiated aimed at the gusset plate connected with the angle section on a single side. Key items that were investigated in the study were strain, strength, and displacement. A total of 5 specimens were used for the experimental investigation. The parameters which were considered for the study, the number of bolt lines and the thickness of the gusset plate. Based on the experimental results of load and strain, a numerical model was developed and compared with experimental load-carrying capacity of plate and displacements. After going through the study unpredicted behavior was observed for the two-bolt line and suggestions were given accordingly.
Keywords: Gusset plate buckling, Compression, Angle sections, Whitmore width

1. Introduction

The Gusset plate connections are wide employed in steel structures to transfer forces from the bracing member to the framing components. A typical gusset affiliation in the braced steel frame is shown in **Fig. 1(a).** It's terribly tough to calculate the compressive strength of gusset plates. Whitmore [1] projected the effective width concept that are being used for the strength calculation of the gusset connections these days. The effective width of a gusset is taken as the distance between the intersection of the 30° lines originated from the very first row of fasteners and the last bolt line, as shown in **Fig. 1(b).** Then the load is calculated by keeping in mind that yielding stress of the plate is uniformly distributed only over the calculated effective width. It ought to be noted that the Whitmore technique of effective width overestimate the strength of the gusset that fails showing inelastic buckling. The length [2] of the column strip is taken as maximum of L1, L2, and L3, as shown in **Fig. 1(b).** Then the buckling strength of the gusset is calculated using the Whitmore effective dimension as mentioned and the compressive resistance using column analogy. He used column buckling formula and did not take into consideration the consequences of plate action. Besides, the column buckling formula solely considers the column strip beneath the effective dimension and also the load distribution because of the yielding isn't taken under consideration properly. Therefore, the designer technique wouldn't be acceptable if important yielding happens within the plate before buckling. Yam and Cheng [3] developed the changed designer technique on the idea of load distribution. it absolutely was projected that a 45° dispersion angle will be used to evaluate the effective dimension, in spite of 30°. The unsupported length of the unit column strip (i.e. Lc as shown in **Fig. 1(b))** is usually recommended to be evaluated from the top of the splice member to the beam and column boundary if the splicing member with spare flexural stiffness is employed. The changed designer load is then calculated supported the extended effective dimension and also the acceptable column curves. Since the changed designer technique accounts for the load distribution behavior within the gusset connections, this overestimate the compressive strength of gusset plates than the designer technique. However, the column buckling formula continues to be used, and also the effects of the plate action aren't thought to be considered.

2. Current design procedure

The current design method is based on the guideline provided by Federal Highway Administration which was proposed after the collapse of the Interstate 35W (I-35W) bridge. In this for the Thornton section are used, and gusset plate is designed as equivalent column section. The length is taken as the average of the lengths (L_1, L_2, and L_3) and effective length is calculated on the basic of the boundary conditions.

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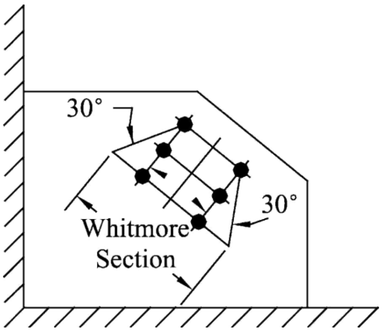

Fig. 1 (a). Whitmore section

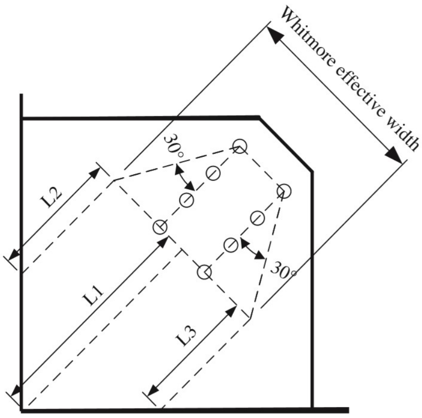

Fig. 1 (b). Whitmore effective width

Table 1. Strength calculation formula

$$f_E = E \times \left(\frac{\pi \times r}{K \times L}\right)^2 \tag{2a}$$

$$f_d = f_y \times (.66^{\lambda}) \quad \textbf{for } \lambda \leq 2.25 \tag{2b}$$

$$f_d = f_E \times (.88) \quad \textbf{for } \lambda > 2.25$$

$$\lambda = \frac{f_y}{f_E} \tag{2c}$$

Where,

f_E = The Euler buckling strength of the column
f_d = Design compressive strength of gusset plate based on effective section (FHWA guideline)
E = Young's modulus,
r = radius of gyration of the equivalent column,
K = effective length factor,
L = average of three lengths L_1, L_2, and L_3

3. Experimental study

Testing was done on 40-ton capacity UTM. Load cell is placed on the top of the specimen for load reading. Strain gauges are placed at the edge of the Whitmore section before fixing the strain gauges the surface was cleaned using the sand paper and cleaning agent (Alcohol or Acetone). And then the strain gauge is pasted using super glue. LVDT was place at the center of the plate between the connections. Load was applied on the load cell then transferred to the gusset plate through the angle section connected with the bolts.

3.1 Testing

The specimen was placed using some packing plates to make it steady and load cell was placed on the C.G of the angle section then calibration of the UTM was done for balancing the dead load by taring. Then the specimen is loaded and displacement and strain data are recorded through LVDT and Strain gauge.
Load vs Strain curves were plotted and load carrying capacity was compared with the FHWA design load for the different

3.2 Experimental test result and discussion

As per Whitmore theory, as the number of bolt lines increases the theoretical strength should increase as the Whitmore width increases. However, here the failure load for the specimen 4-1 was more then 2-1, which was more than that of specimen 3-1. This anomaly from the theoretical value is due to the initial imperfection in the 3-1 specimen as some eccentricity was unintentionally introduced during the fabrication due to mismatching of the bolt line drilling. For the plate, specimen 2-1 has nearly no such mismatching and there was nearly no eccentricity.
By observing the strain values, specimen 2-1 initially had compression behavior and then it changed to tension after the buckling dominance. But for the 3-1 and 4-1 the strain from the starting remains in the compression. Here strain is multiplied with -1 for 3-1and 4-1 for comparison as shown in Fig. 3 (b). This has only to do with the side on which buckling deformations occurred, rather than any other specific causes.

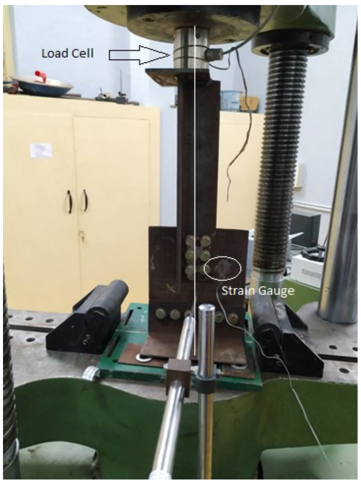

Fig 3 (a). Experimental Setup

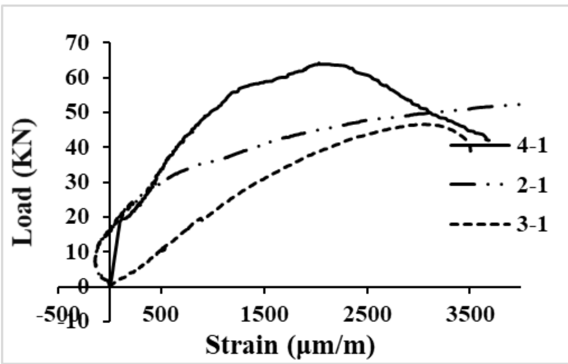

Fig 3 (b). Load vs Strain

Table 2. Details of Specimens and experimental result

Specimen	Plate thickness (t) (mm)	Plate width (W) (mm)	Plate length (L) (mm)	Gap length (L_g) (mm)	Whitmore effective width (mm)	Fastener length (L_f) (mm)	Fastener width (W_f) (mm)	P_{exp} (KN)
2-1	**3**	**150**	**210**	**75**	**64.64**	**30**	**30**	**53.03**
3-1	**3**	**240**	**240**	**75**	**100**	**60**	**30**	**46.55**
4-1	**3**	**270**	**270**	**75**	**134**	**90**	**30**	**64.17**

The deformation observed after the experiment, from the buckled shape also its clear that specimen 2-1 had no eccentricity and also both the deflected profile seen from left and right views had same side sway failure and curvature is same, but for both 3-1 and 4-1 have left eccentricity which made the strain tensile in the side of strain gauge side and due to this dual curvature buckling occurred during failure. The details of experimental specimen and the load value is show in the Table 2.

4. Numerical study

The general FE program ABAQUS was used to simulate the gusset plate models. The gusset plate specimen used in the experimental study previously was taken here for numerical study. The load was applied to gusset plate though a compression member (i.e. angle) which is connected to plate with two, three or four bolt lines each bolt line consist of two bolts. This section describes the numerical details of the specimen.

4.1 Properties of components

The entire model consisted of Angle-section and gusset-plate modelled as a solid type and generated using the extrude option (Element, type=C3D8R)**,** Bolt and shank head modelled as solid type and generated using the revolution option (Element, type=C3D8R)

The properties of the material used (steel) are taken from the coupon test are given below

Density = 7.85 x 10-9 kg/mm3
Young's Modulus = 2 x 10^5 N/mm2
Poisson's ratio = 0.3
Yield stress = 175.4MPa
Ultimate stress = 333.33MPa

4.2 Modelling process

The modeling and assembly of all the parts consists of part assembly of gusset plate**,** member (angle section), and bolt assembled using 'translate instance'. Then two-step analysis was performed. In first step static-general analysis was performed for applying preload on bolts and in the second step another static-general analysis, was performed for the plate with the actual boundary and member load was given. Then interaction between the surfaces of the bolt and the plate surfaces were applied between the following two contact pair sets: plate surface to member surface and member surface to bolt head surface with friction 0.3 and penalty 0.2.The gusset plate size used in the study was 200(mm) ×185(mm) and thickness 3(mm), 200(mm) ×215(mm) and thickness 3(mm) and 150(mm) ×155(mm) and thickness 3(mm). The member Angle section size used in the study were (300, 385, 465) (mm) long with cross section 75 × 75 × 6. Bolt of diameter 12 (mm) and shank length 9 (mm) created by solid (revolution). The bolts preload was taken as 50% of the proof stress of the bolt, in the first step of the analysis, using the bolt load option in ABAQUS. The pretension is simulated by splitting the bolt body and applying the preload force on two parallel surfaces in the bolt-shank. The appropriate side is to be chosen for applying the preload in the correct direction. For applying the boundary conditions, displacement and rotation in X, Y and Z directions in the unloaded bottom edge are restricted. This boundary condition is named as BC1 (fixed edges: U1=U2=U3=0, UR 1= UR 2= UR 3=0). And loaded upper side of angle section pinned edge with release of Y direction displacement. This boundary condition is named as BC2 (pined edges: U1=U3=0, UR 1= UR 2= UR 3=0). For meshing, global meshing size adopted for all specimens was of size 15 mm, refined mesh size of 5 mm near the holes were used and the gusset plate is messed using the local seed-number method along the thickness for getting two-layer of surface. The member and plate were meshed specifying the above seeding size and using the 'mesh part' option. The bolt was meshed with create bottom-up mesh 3-dimensional solid element (C3D8R) were used for all messing.

4.3 Mess convergence

For proper messing of the model mess convergence study was done on preliminary lap joint connection of single bolt and deflection vs number of element graph was obtained as shown in Fig. 4(b). and from the result 5 mm mess size near bolt hole were taken for modeling.

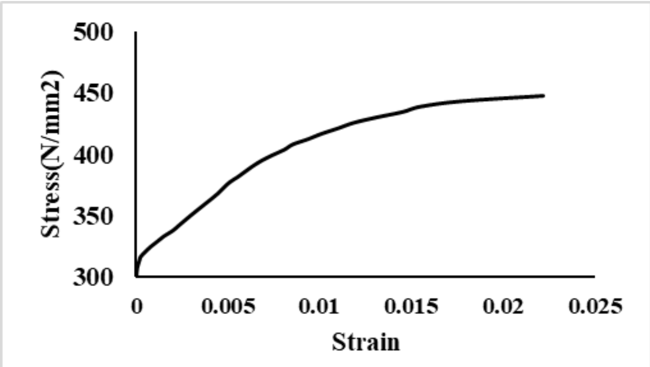

Fig. 4(a). Plastic property of gusset plate

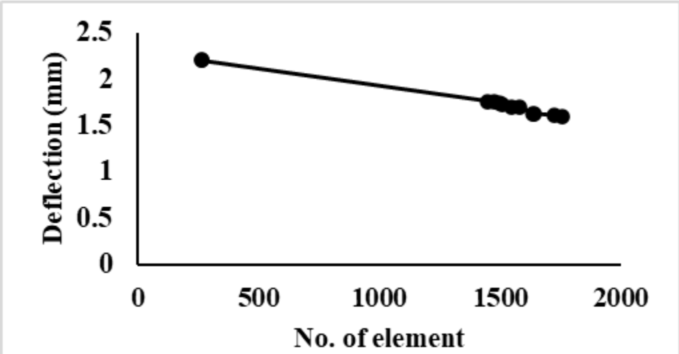

Fig. 4 (b). Mess convergence

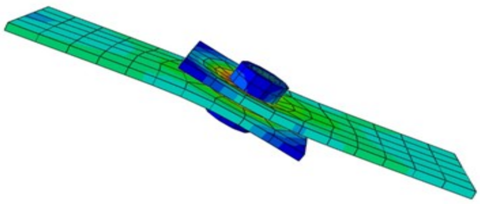

Fig. 4 (c). Preliminary lap joint

5. Comparison of design compressive strength of gusset plate

The results obtained after analysis are compared and Table 3 shows the load obtained and Fig 5(a), 5(b), and 5 (c) shows the graph of load and micro strain and from these it can be seen that the strain at the failure is nearly in range of (3500 and 4000) micro strain. The slope of curve obtained from experimental and numerical are decreasing with increase in load. The load obtained from ABAQUS is in-between the experimental and FHWA value

Table 3. Comparison of design, ABAQUS and experimental load

Specimen	L (mm)	F_{ABAQUS} (KN)	P_{FHWA} (KN)	P_{exp} (KN)
2-1	100	45.43	41.03	53.0324
3-1	100	56.6	64.48	46.5556
4-1	100	68.51	85.063	64.1736

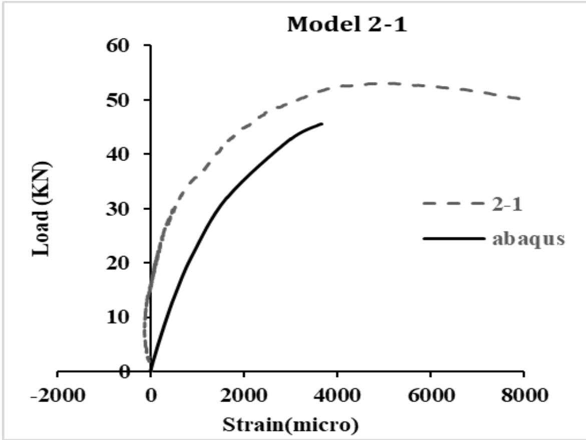

Fig. 5 (a). Load vs micro train of Model 2-1

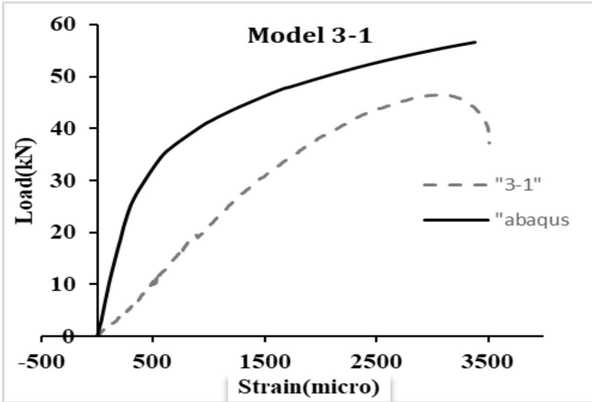

Fig. 5 (b). Load vs micro strain of Model 3-1

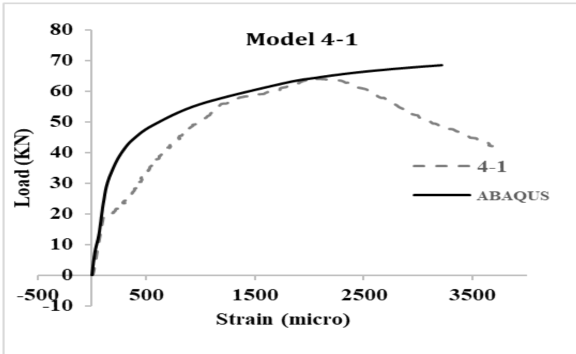

Fig. 5 (c). Load and micro train curve of Model 4-1

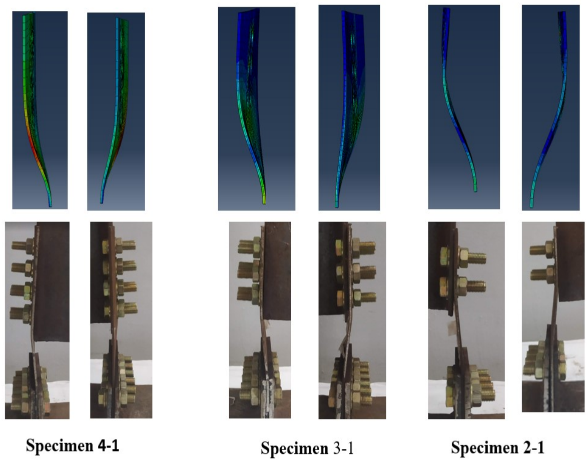

Fig 5 (d). Deflected shape of plate

The deflection of plate obtained from numerical analysis conformed to the experimental observation of a dual nature of curvature as shown in Fig 5(d). All the models swayed out of the plane, on increasing the load where initially the curvature of the free end was same but later twisting type of deflection were observed. This twisting came because load was applied on the angle section member and due to that biaxial moment was applied on the gusset plate as this moment was very less in magnitude wise but increase as the load was increased and also on increasing the number of bolt lines. This biaxial moment increases resulting in increase in the deflection.

6. Parametric study

Parametric study done by varying the number of bolt line connections (2, 3, and 4) between the gusset plate and angle section and also different thickness (2, 2.5, 3, 3.5 and 4) of gusset plate were also considered for the studied.

6.1 Load - deflection behavior

Typical load versus vertical displacement curves from each type of Model 2-1, Model 3-1 and Model 4-1 corresponding to the thickness (2, 2.5, 3, 3.5 and 4) mm of gusset plate were plotted are shown in Fig. 6.1(a), Fig. 6.1(b) and Fig. 6.1(c). It can be seen from the figure that displacement of the gusset plates showed non-linear load deflection behavior in all the specimens prior to reaching the ultimate load and the displacement of the specimens increased significantly after the ultimate load.

From Fig. 6.1(a), it can be seen that, for (2-1) model the load capacity is increases nearly linear with increase in the thickness and the maximum axial deflection is decreasing as the thickness of the gusset plate increases, and plate failure is observed for all the model.

From Fig. 6.1(b) and Fig. 6.1(c), for both (3-1) and (4-1) models, the increase in the load capacity increased linearly but the deflection was very less for higher thicknesses. This is because for higher thicknesses, the failure was member failure (shown in Fig. 6.1(e)), in others words plate failure (shown in Fig. 6.1(d)) was observed for thinner plates and angle member failure is observed for thicker plate.

For specimens with thinner plates, the stiffness was very high at the initial load and with increase in loading the stiffness decreases drastically like a slender column. But for thicker plates the decrease in stiffness was not high and nearly remains steady. This is because the plate did not reach the yield point and further load can be taken which means plate did not failed and this justify the member failure.

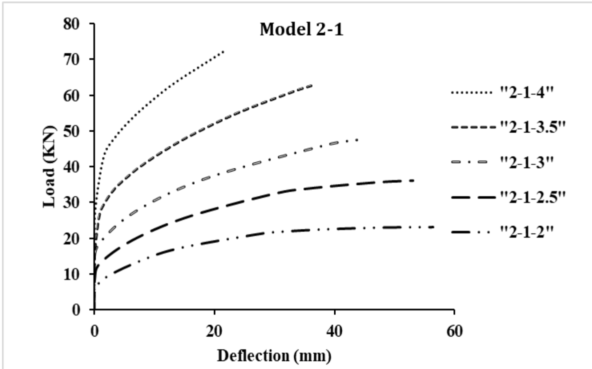

Fig. 6.1(a) Load vs deflection curve of Model 2-1

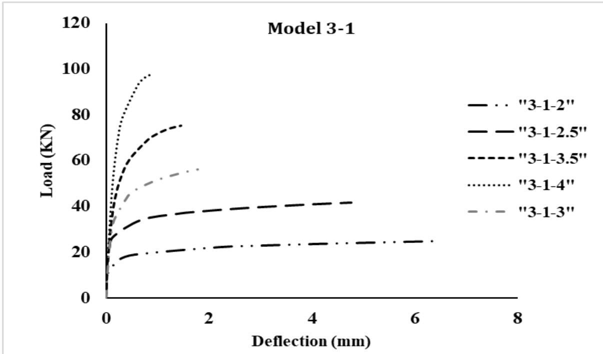

Fig. 6.1(b) Load vs deflection curve of Model 3-1

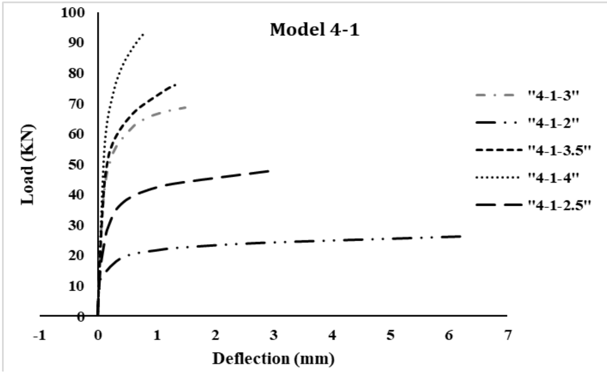

Fig. 6.1(c) Load vs deflection curve of Model 4-1

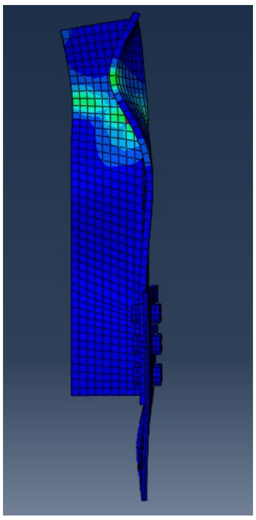

Fig.6.1(d) Plate failure for model 2-1 (3mm)

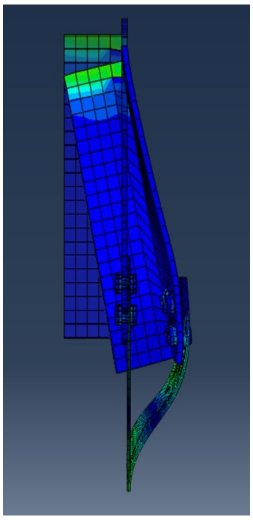

Fig.6.1(e) Member failure for model 3-1 (4mm)

Typical load versus out-of-plane displacement curves from each type of Model 2-, Model 3-1 and Model 4-1are shown in Fig. 6.2(a), Fig. 6.2(b) and Fig. 6.2(c). It can be seen from the figure that out-of-plane displacement of the gusset plates that nonlinear load deflection behavior was observed in all the specimens prior to reaching the ultimate load and the lateral displacement of the specimens increased significantly after the ultimate load was reached

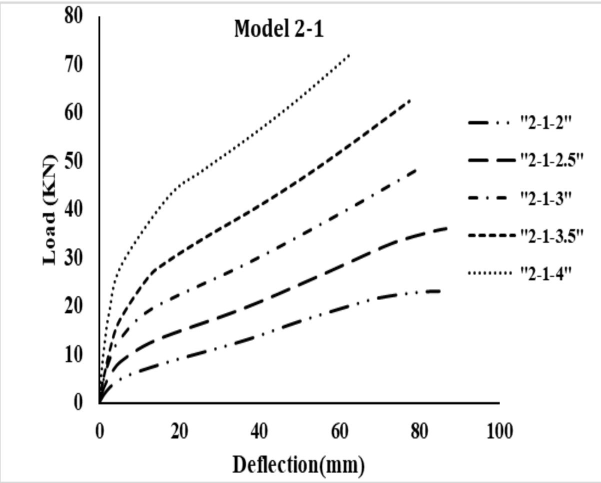

Fig. 6.2(a) Load vs lateral deflection curve of Model 2-1

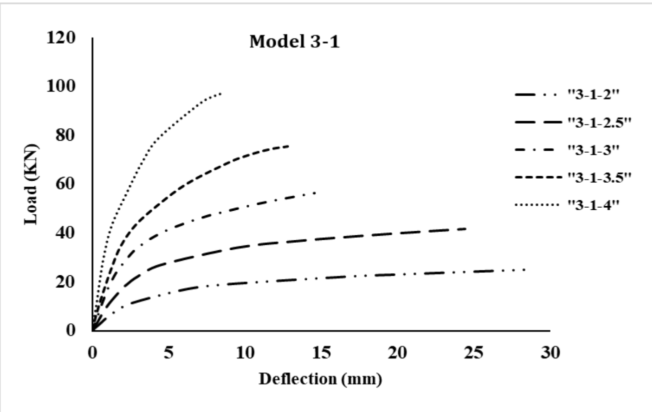

Fig. 6.2(b) Load vs lateral deflection curve of Model 3-1

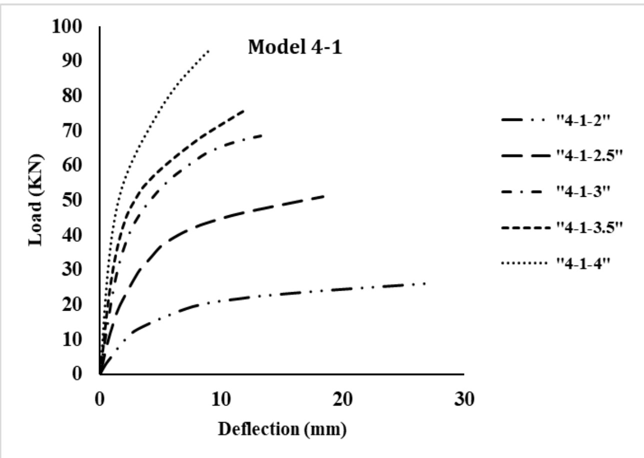

Fig. 6.2(c) Load vs lateral deflection curve of Model 4-1

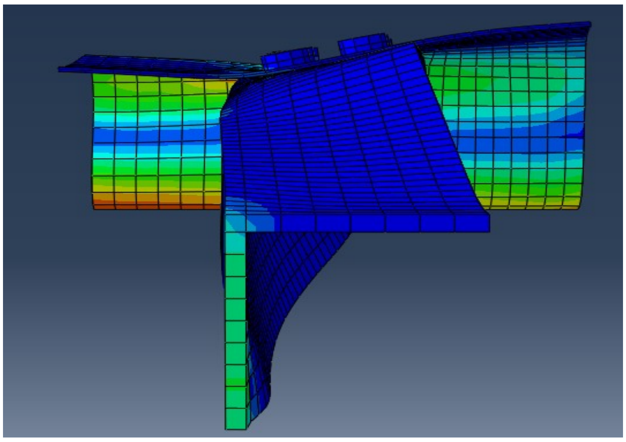

Fig. 6.2(d) Twisting effect

For all the model the lateral deflection is more than three times the axial deflection. The impact of member failure is also reflected in this and due to this the deflection for thicker plate is less as compared to thinner plate. The lateral deflection of plate was not constant at a particular horizontal plane also and is maximum for the free end which was farthest from the outstanding leg of angle, this was due to the twisting effect on the plate due to loading on the member as shown in Fig. 6.2(d).This effect increases with increase in number of bolt line in connection and decreases with increasing thickness of plate.

For all the models initial stiffness increases with increase in the gusset plate thickness. The difference of initial stiffness is not very high as compared to the axial stiffness. And no effect of member or plate failure was seen in lateral stiffness.

6.2 Ultimate load carrying capacity based on FHWA guidelines analysis

Ultimate load carrying capacity of full gusset plate was calculated form Design compressive strength f_d of gusset plate converted into an equivalent column with the cross-section of gusset plate and thickness based on effective section shown in Table 4. The length of gusset plate is kept constant.

6.3 Comparison of results of gusset palate for different thickness

The load capacity of gusset plate from theoretical and numerically and deflection according to the ABAQUS are shown in Table 5 .From table it can be seen that for model 2-1 the numerical result is higher compared to theoretical value because the Whitmore width edge was inside the angel so the stress was up to the member edge, due to this increase in width the numerical value is more. The ultimate strength for gusset plate is very less as compared to theoretical value because as number of bolt line increases the twisting effect increases and the gusset plate is subjected to additional moment, this moment was not considered while strength calculation, only direct compression was assumed for load calculation and it can be seen that as the thickness increases this effect is decreasing so the difference is also decreasing.

Table 4. Ultimate load carrying capacity based on FHWA guidelines

Model	Plate thickness (t) (mm)	I_g (mm^4)	A_g (mm^2)	L (mm)	r (mm)	f_E (N/mm^2)	λ	F_d (N/mm^2)	P_d (KN)
2-1	**2**	**43.09**	**129.3**	**75**	**.578**	**277.3**	**.903**	**171.78**	**22.21**
	2.5	**84.17**	**161.6**	**75**	**.721**	**433.3**	**.578**	**196.62**	**31.78**
	3	**145.44**	**193.9**	**75**	**.866**	**622.9**	**.401**	**211.60**	**41.03**
	3.5	**252.0**	**226.2**	**75**	**1.01**	**850.0**	**.295**	**221.16**	**50.03**
	4	**376.03**	**258.5**	**75**	**1.15**	**1109.3**	**.226**	**227.6**	**58.83**
3-1	**2**	**66.67**	**200**	**75**	**.578**	**277.3**	**.903**	**171.78**	**34.36**
	2.5	**130.21**	**250**	**75**	**.721**	**433.3**	**.578**	**196.62**	**49.15**
	3	**225**	**300**	**75**	**.866**	**622.9**	**.401**	**211.60**	**64.48**
	3.5	**357.3**	**350**	**75**	**1.01**	**850.0**	**.295**	**221.16**	**77.406**
	4	**533.3**	**400**	**75**	**1.15**	**1109.3**	**.226**	**227.6**	**91.04**
4-1	**2**	**89.34**	**268**	**75**	**.578**	**277.3**	**.903**	**171.78**	**46.04**
	2.5	**174.48**	**335**	**75**	**.721**	**433.3**	**.578**	**196.62**	**68.87**
	3	**301.5**	**402**	**75**	**.866**	**622.9**	**.401**	**211.60**	**85.06**
	3.5	**478.8**	**469**	**75**	**1.01**	**850.0**	**.295**	**221.16**	**103.7**
	4	**714.6**	**536**	**75**	**1.15**	**1109.3**	**.226**	**227.6**	**122**

Table 5. Comparison of results of gusset palate

Specimen	Plate thickness (t) (mm)	F_{ABAQUS} (KN)	P_{FHWA} (KN)	$\frac{P_{ABAQUS}}{P_{FHWA}}$	Deflection (mm)	Increase in load (%)	Decrease in deflection (%)
2-1	**2**	**23.237**	**22.21**	**1.05**	**56.39**		
	2.5	**36.035**	**31.78**	**1.13**	**51.45**	**55.08**	**8.76**
	3	**47.5**	**41.03**	**1.16**	**43.66**	**31.8**	**15.14**
	3.5	**62.562**	**50.03**	**1.25**	**36.18**	**31.71**	**17.13**
	4	**72.14**	**58.83**	**1.22**	**21.48**	**15.31**	**40.6**
3-1	**2**	**24.84**	**34.36**	**.73**	**6.45**		
	2.5	**41.49**	**49.15**	**.85**	**4.62**	**67.03**	**28.37**
	3	**56.11**	**64.48**	**.87**	**1.70**	**35.24**	**63.2**
	3.5	**75.26**	**77.406**	**.97**	**1.4**	**34.13**	**17.65**
	4	**97.11**	**91.04**	**1.07**	**.825**	**29.03**	**41.07**
4-1	**2**	**26.15**	**46.04**	**.58**	**6.21**		
	2.5	**47.77**	**68.87**	**.7**	**2.96**	**82.67**	**52.3**
	3	**68.51**	**85.06**	**.81**	**1.49**	**43.42**	**50**
	3.5	**76.07**	**103.7**	**.73**	**1.31**	**11.1**	**12.08**
	4	**92.998**	**122**	**.76**	**.79**	**22.25**	**38.9**

7. Conclusion

A total of 5 specimens and 15 models were studied experimentally and numerically respectively for the behavior of thin flat gusset plates in compression. Based on the results of the studies, following conclusions are drawn:

1. As maximum axial deflection was less for thick plate, this suggests that member failure is prior to gusset plate in thin gusset is to be prevent to avoid connection failure. Conversely, the members should not be overdesigned so that connection failure may be prevented in extreme loadings
2. The twisting effect on the plate due to loading on the member increases with increase in number of bolt line connection and decreases with increasing thickness of plate.
3. The maximum lateral deflection was more than the corresponding axial deformation. This suggest that the lateral deflection also depend on the connection length as it increases the eccentricity of the connection reduces and the (P- Δ) effect on the plate is reduced. So larger connection length is preferred for gusset plate connection.
4. The initial lateral stiffness increases with increase in the gusset plate thickness. The rate of stiffness is increasing at initial stage and the started decreasing and again increases at later stage. The difference of initial stiffness is not very high in this case as compared to the axial stiffness.

Disclosures

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