Research Paper Engineering

Failure in tensile testing on single lap multi-fastener joint with bolted connection

* Jagdish N.Prajapati ** Dr.Rajula.k.Gujjar
*** Prof.M.M.Pomal.

* Post graduate student, applied mechanics department / government engineering college, Dahod

** H.O.D., L D College of engineering, Ahmadabad – 380015

*** Department of Applied mechanics, Government engineering college, Dahod

ABSTRACT

This paper present a finite element model was developed to study predicts their tensile capacity of single lap multi fastener joint with missing bolt system. Despite the large database of test result on tensile test in single lap multi-fastener joint. The exact progression of the failure mechanism is not clear in plate. Although current design equation predict the bearing capacity of bolts fairly well, it is important for a design equation not only to predict the capacity reliably, but also to predict accurately the failure mode. Current research at the University of Alberta makes use of a non-linear finite element model to study the block shear behavior in gusset plates. The model is developed to provide a useful tool to study tensile capacity and failure not only in plates and bolt but also in other member such as truss member, splices, lacing, braced structures and gusset plates. Finally this paper depends on the behavior study up to failure load and predicts strength of the connection.

Keywords : stress distribution with load, bolted connection, Fe 250 plate and 8.8 grade M16 bolt with washer

1. INTRODUCTION

Strength of the joint is depending on many factors. These factors are joint type, material type, loading condition, geometry. In this we have used tension member. Tension member are widely used in steel structures such as trusses, splices, lacing, gusset plates and braced structures. And all are connecting each other by connection. From above, bolted connection we have used for this joint and my joint is single lap multi-fastener joint and tensile test conduct on the single lap multi-fastener joint. One of the failure mode in plates loaded in tension is bearing failure and one of failure mode in bolt in tension is shear failure. Above bearing failure is depending on geometrical parameters when ever geometrical parameters change then the stress distribution in material around the hole has changed, so that failure of the plates and bolts may be change whenever we have change above parameters. This mode of the failure can presumably occur in welded connection and bolted connections; it is more common in the latter because of the reduced area that results from the presence of bolt holes.

Many researchers have investigated block shear in gusset plates through full scale testing. Chakrabarti and Bjorhovde (1983) and Hardash and Bjorhovde (1984) looked at the inelastic behavior of gusset plate connection in tension. From their tests and those of other investigators, model was proposed to predict the ultimate capacity of gusset plate connections in tension. They proposed that the ultimate strength of the gusset plate is the sum of the tensile strength of the net area between the bolts in the last row and the shear strength along the connection length. The proposed effective shear stress along the connection length lies between the yield strength and the ultimate shear strength, depending on the length of the connection. The effective shear stress decreases as the length of the connection increases.

An extensive test program on splice plates in tension was conducted by Udagawa and Yamada (1998). The parameter that where experimentally investigated were the gauge length, end distance, number of lines and rows of bolts, and steel grade. The test specimens were loaded in tension only and, as a result, several specimens failed by block shear. As for earlier test programs, the exact sequence of the failure process and the load sharing between the shear and tension planes was not closely examined.

The main objective of the research work presented in the following was to develop a finite element procedure to predict the behavior of a failure from initial yielding of the plate to rupture along the tension face and subsequent rupture along the shear faces. Such a model would be valuable for the investigation of the failure in other member such as truss member, splices, lacing, braced structures and gusset plates.

2. FINITE ELEMENT MODEL

Several researchers have used the finite element method to investigate the stress distribution in plates and also bolts. Davis (1967) and Varsarelyi (1971) carried out finite element investigation of the elastic stresses in gusset plates. In general, this investigation confirmed the findings of Whitmore's experimental investigations regarding the stresses in gusset plates loaded in the elastic range. In elastic analysis of gusset plates has also been conducted in a number of investigations. Williams and Richard (1986) performed numerical and experimental work to develop design procedures for gusset plate connections in diagonal braced frames. Their work focused on the distribution of forces in the gusset to frame and gusset to braced fasteners. Chakrabarti and Bjohovde (1983) used non linear finite element formulation to predict the structural response of gusset plates in tension. The behaviour and stress distribution of the gusset plate models were in good agreement with test result but, since the finite element formulation used for the analysis did not model buckling and material tearing, some aspect of the behaviour at ultimate load could not be compared. Walbridge et al. (1998) investigated gusset plates subjected to monotonic loading in tension and in compression and to cyclic loading. Boundary member, brace member, and fasteners were incorporated in to the model. Using an elastic-plastic material model, the peak capacity of gusset plate in tension was predicted accurately, although tearing of the gusset plate during block shear failure was not included in the model. A similar observation was made in the work of staggered and non-staggered holes using a non-linear solid element. The investigators attempted to model the onset of block shear failure by comparing the peak strain determined from an inelastic finite element analysis to an arbitrary strain of five times the yield strain, which was selected based on a comparison of the analysis results with test result. The strains at the edge of the holes were calculated from the calculated nodal displacements. The researchers reported good correlation between the finite element analysis and test results.

For the study presented in the following, the general purpose commercial finite element analysis program SAP 2000 14 was used to model single lap multi fastener joint in plates. The proposed finite element procedure is validated through comparison with results of tests presented by other researchers. A description of the development of the finite element models and the process of validating these models follows.

2.1. Preliminary Analysis

The finite element model used in the preliminary analysis was based on a 16mm thick plate. This16mm thick plate was tested practically also. Only the plate was incorporated in the finite element model, with fixed at the bottom side and only free move in vertically direction at the upper side and other are fixed. Load was applied to the upper side of the plate and bolt hole was drilled at the centre of the plate. And show the bearing of the bolt against holes. Holes were drilled in the single line.

The solid element block from SAP-14 was used to model the plate. The block element is an eight node solid element that account for finite strains and allow for change in the element thickness. It has six degree of freedom at each node (three displacement components and three rotation components). The material properties used in the finite element analysis were true stress and strain derived from the engineering STRESS VS STRAIN curve obtained from tension test. The engineering and the true stress versus strain curve are shown in figure below. A modulus of elasticity of 215000 MPA and poison's ratio of 0.3 were used in the elastic range. An isotropic hardening material model was used in the inelastic range. Also shown the large deformation, large displacement analysis and it was used in the study.

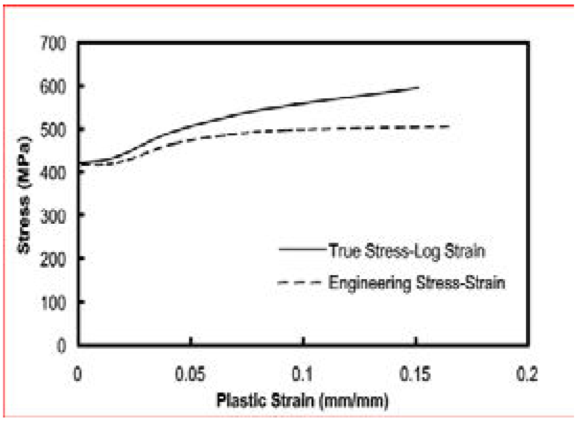
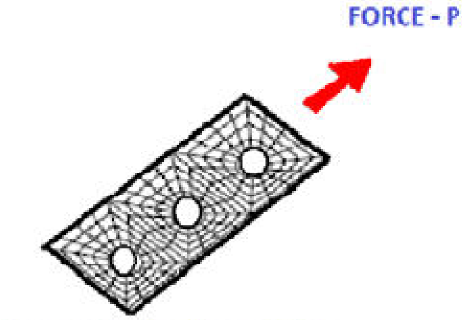


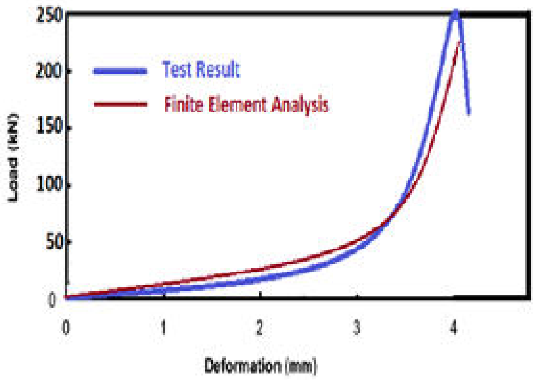
FIG: - practical result VS software basis result

A mesh refinement study was conducted to determine the correct mesh size to use in the analysis. A mesh refinement study was performed by gradually refining the mesh and checking convergence of the major principle strains on the tension portion of the of the shear failure surface. Because of material yielding, a high strain concentration was observed near the bolt holes, necessitating a very small mesh size to reach convergence. A comparison of the finite element analysis result with test results was performed to validate the finite element model. The LOAD VS DEFORMATION curve, shown below in Figure, is used to compare the finite element result with the actual test result. Since the stiffness and the capacity predicted by the finite element model were in very good agreement with the test result.



Coarse mesh in the rifinement study

Fine mesh in the refinement study



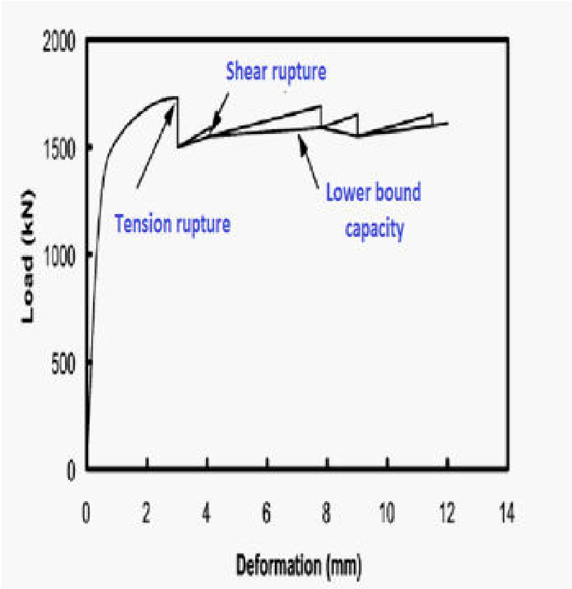
Load vs Deformation from Fine Mesh

2.2. Modeling of Fracture on the Tension Face

In order to model progression of rupture during shear failure, consistent failure criteria are required for the tension and shear faces of the shear failure surface. It was felt that the failure criterion proposed by Epstein and chamarajanagar (1994) would not be suitable for this study because the mesh size used was much coarser than the mesh size found to be necessary to ensure convergence in the present investigation, and the rupture strain proposed (five times the yield strain, which would make it less than 1.0% for this investigation) is much smaller than the rupture strain observed for steel. An investigation of the ductile fracture of steel by khoo et al. (2000) showed that the localized rupture strain is approximately 80% to 120% for structural grade steel. An average strain value of 100% was therefore used as a failure criterion for fracture along the tension face. In order to determine when fracture on the tension face would initiate, the major principal strain at the element integration point across the net tension face was plotted and extrapolated to the edge of the bolt holes using a sixth order polynomial. Rupture along the tension face was assumed to take place when the major principal strain, obtained by extrapolating to the edge of a bolt hole, reached a value of 100%. All the elements on the tension face were then removed from the model to simulate tension fracture.

2.3. Modeling of fracture on the shear planes

Two shear fracture models were investigated. In our cases two type of failure done first is plate failure due to shear and second is bolt failure due to shear. The first consisted of using the maximum principal strain as a criterion for removing element along the shear planes as the critical strain of 100% was reached along the shear plane. The predicted load versus displacement curve, shown in figure below, showed a small increase in load carrying capacity after tension rupture. As more elements were removed, the load started to decrease. However, as element were deleted along the shear failure plane at larger values of connection deformation, the load started to increase again. This type of behaviour does not seem realistic and another shear failure criterion was sought.



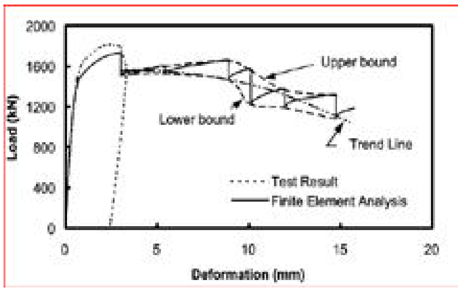
The second method consisted of using a critical maximum shear strain. In order to establish the critical shear strain, the maximum shear strain was plotted across the tension plane when the maximum principal strain had reached 100% localized tension rupture strain. The shear strain distribution was then extrapolated to the edge of the hole on the tension face to obtain the maximum shear strain associated with the 100% concentrated tension rupture strain. This critical maximum shear strain was subsequently used as a rupture criterion on the shear planes, when an element along the shear plane reached the critical shear strain, the element and as many as six other adjacent elements along the failure path were removed and equilibrium iteration allowed the new equilibrium state to be reached. Load was further increased until the critical shear strain was reached once again in the next element along the failure path. The corresponding load versus deformation curve was therefore a stepped curve reflecting the stepwise nature of this crack propagation model. The loading and element removal process was repeated until the trend on the load deformation curve showed a significant decrease in load carrying capacity.

3. COMPARISION OF FINITE ELEMENT ANALYSIS WITH TEST RESULTS

3.1. Nast et al. (1999)

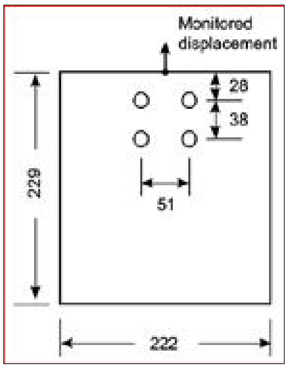
Figure shows a comparison of the finite element results with the result of a full-scale test conducted by Nast et al. (1999). The finite element model for this test specimen was presented in Figure. Although the test was conducted under cyclic loading condition, the test specimen was loaded to fracture on the tension plane in the final load cycle. The finite element model was used to investigate the post-tension fracture behaviour of the gusset plate. Since Nast ET. Al. stopped the test at fracture of the tension face, a direct comparison between the finite element analysis and test results cannot be made beyond rupture of the tension plane. The finite element model is found to predict well the observed block shear capacity of the test specimen.

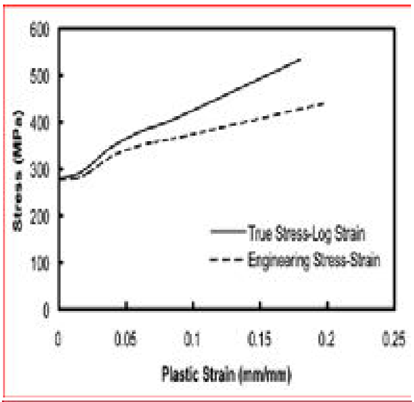
The progression of the block shear failure beyond tension rupture follows a general trend between the curve designated as upper bound and the curve designated as lower bound. The trend line was obtained by fitting a second order polynomial through the load v. deformation curve obtained after tension fracture using a least square regression analysis. This trend line indicates that the load carrying capacity of the connection increases slightly after tension fracture, but never exceeds the load just prior to tension fracture. The actual block shear failure behaviour is characterized by the gradual tearing of the material, which can only be approximated by a series of discrete steps in the model used here. These discrete steps in the load versus deformation curve represent only the general trend after tension fracture.

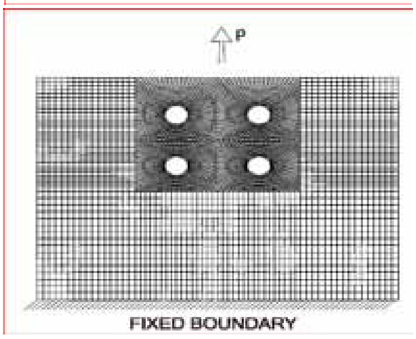


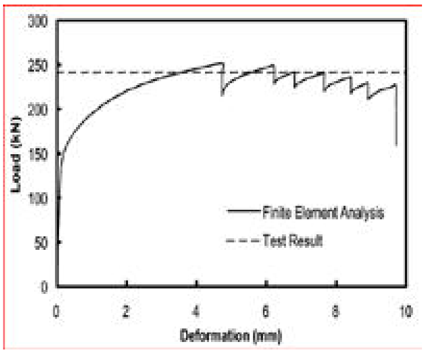
3.2.Hardash and Bjorhovde (1984)

A 6.4 mm plate of dimensions and bolt layout shown in Figure was tested to failure by Hardash and Bjorhovde (1984). The test specimen was modeled using a mesh similar to that of Nast et al. (1999). The stress v. strain curve used for the analysis is shown in Figure. The modulus of elasticity was taken as 204 400 MPa and an isotropic hardening model was used. The finite element mesh is shown in Figure. The load versus displacement curve obtained for the Hardash and Bjorhovde specimen is shown in Figure. The experimental load v. displacement curve was not available. The general behaviour predicted by the finite element analysis is similar to that of the Nast et al. test specimen presented above. The capacity predicted using the finite element model is again in good agreement with the test result. It can be observed on Ce again that the capacity of the gusset plate after tension fracture does not exceed the load level reached before tension rupture. The shear capacity after tension rupture either increases by a very small amount or decreases after tension rupture, indicating that all or most of the shear capacity has been mobilized by the time fracture takes place on the tension face of the block shear failure surface.









3.3. Jagdish prajapati (2012)

A 16 mm thick plate of dimensions and bolt lay out shown in figure was tested by jagdish prajapati (2012). The test specimen was modeled using an automatic mesh. The stress versus strain curve used for the analysis is shown in figure. The modulus of elasticity was taken as 200000 MPa and an isotropic hardening model was used. Also used size of hole is 16 mm with curve surface at the periphery of the hole both side of plate. In these testing we were used 8.8 grading M 16 bolt with nut and washers and applied torque by torque range was 135 N.m. (14 kg). Connection used in tensile testing was fixed. Also bolted connection used in tensile testing was single-lap multi fastener joint.

The load versus displacement curve obtained for the jagdish prajapati is shown in fig. the experimental load versus displacement graph was available. This graph was directly given by universal testing machine during practical. Also finite element method used with the help of SAP SOFTWARE. The general behaviour predicted by the finite element analysis is similar to test specimen presented above. The predicted capacity using the finite element model is again in good agreement with the test result. In above model can be observed under cyclic loading condition but in our case static load apply on test specimen. And failure take place in the bolt was shear failure. And failure take place in the plate was bearing failure. Also during this testing no clearance provided between two plates and this criteria shown during practical as secondary bending. In this practical bolt fail by shear failure was become first. Assume that if bolt is not fail due to shear then bearing failure is clearly shown in the tested specimen because of this material modulus of elasticity is high also appropriate distance between hole to hole, hole to end part of the plate and hole to side edge of the plate. In this tensile testing, strength does not increase after shear failure of any one bolt in this connection and suddenly fail the all bolts in the connection. Also using high strength bolt in this connection then that was not give the more deformation. Grip length provided 100 mm of total length.

3.4. Further Validation of the Finite Element Procedure

The above comparisons with test results indicate that the proposed finite element procedure is able to predict reliably the shear capacity of gusset plates and bolt in connection. It also shows that the remaining shear capacity does not exceed the capacity developed at tension fracture in the connection configurations studied. First fracture on the tension face of the block shear failure plane can therefore reasonably be taken as the ultimate capacity of gusset plates. In order to further validate the finite element model developed to predict block shear failure, the experimental block shear capacity of selected test specimens was compared with the predicted load carrying capacity at tension rupture. Another thirteen test specimens from published and unpublished sources were analyzed using the proposed finite element procedure. A brief description of the test specimens is presented in Table 1. The test specimens were selected to cover a wide range of geometric properties within test data available in the literature and some unpublished data. Each test specimen presented in Table 1 was modeled and analyzed to determine the capacity of each test specimen. The analysis in each case was stopped when the critical major principal strain was reached on the tension face. The shear failure progression was not investigated in these specimens. A comparison of the predicted block shear capacity, shear capacity and the test results is presented in Table 2. The average test-to-predicted ratio for the thirteen specimens presented in Table 2 is 1.08, with minimum and maximum values of 0.96 and 1.1983, respectively. It is therefore concluded that the finite element model developed for block shear and shear failure is able to predict reliably the block shear capacity of gusset plates and shear capacity of bolt.

4. SUMMARY AND CONCLUSION

Although many experiments have been conducted to study block shear failure, shear failure in tensile testing single lap multi-fastener joint with bolted connection. Information about progression of fracture on the tension and shear faces is scarce. A finite element model was developed to study the progression of tension and shear fracture in gusset plates and to predict their shear capacity. The model was validated by comparing analysis results with test results from gusset plates and single-lap connection. A comparison of the test and predicted capacity of gusset plates and single lap connection indicated a good correlation between the predicted strength and the measured strength. A finite element analysis of two gusset plates that modeled tension and shear failure indicated that after tension fracture the shear capacity will not exceed the tension fracture capacity. The proposed finite element model can therefore be used to predict block shear failure capacity in gusset plates and tensile capacity in single lap multi-fastener connection. It is also expected that a similar model can be used for other structural elements such as angles, tees, splices, lacing, braces, truss member or coped beams. The model presented in the previous sections is currently being used by the authors to complete the database of test results on block shear and tensile testing on single lap multi fastener joint with bolted connection. Also the practical results represent the behaviour of the failure, by load – deformation graph: - whenever loading apply at initial level then the load displacement graph is non-linear because of contact between bolt and hole surface not fully developed, this contact between bolt and hole surface increase with increase loading after some loading contact between hole and bolt is fully developed and graph becomes nonlinear to linear. And load-deformation graph linear up to failure point. Also due to practical bolt is suddenly broken by shear. These phenomena depend on the hole outer edge also, whenever outer edge of the hole is sharp then this edge act as a knife and failure take place in connection is shear failure and we are gating wrong result. For reduced this phenomena we was convert the sharp edge to curvature surface. Also shown the failure in plate is bearing type failure. Shown the secondary bending in plate during practical and in software also.

AKNOWLEDGEMENT

This work and testing of the sample complete with the help of the META LAB, odhav, Ahmadabad and economically help full to us for complete the testing during test. Also very thank full to my guide who was help to understand this topic and give proper guidance for understand critical mechanism. Thanks for support.

Table 1 – Description of Test Specimens

Specimen Source	Number of Bolts	Number of Lines	Edge distance (mm)	End Distance (mm)	Gauge (mm)	Pitch (mm)	Plate Thickness (mm)	Hole Diameter (mm)	Yield Strength (MPa)	Tensile Strength (MPa)
Huns (2002)	6	2	177.8	38.1	50.8	76.2	6.61	20.64	336	450
Huns (2002)	8	4	127.0	25.4	152.4	50.8	6.50	20.64	336	450
Mullin (2002)	4	2	50.8	38.1	101.6	76.2	6.83	20.64	306	434
Mullin (2002)	8	2	51.8	39.1	102.6	77.2	7.83	21.64	306	434
Hardash and Bjorhovde (1984)	4	2	85.7	27.9	50.8	38.1	6.02	14.29	229	323
Hardash and Bjorhovde (1984)	6	2	117.5	25.4	101.6	38.1	6.02	14.29	229	323
Udagawa and Yamada (1998)	4	2	88.5	39.8	64.2	40.0	12.00	18.00	278	443
Udagawa and Yamada (1998)	6	2	99.4	40.0	40.7	40.0	12.00	18.00	280	444
Udagawa and Yamada (1998)	8	4	23.4	23.7	40.0	40.0	12.00	18.00	278	443
Jagdish (2012)	3	1	24.5	25	_	41	16	16	258	353
Jagdish (2012)	2	1	24.5	25	_	41	16	16	172	236
Jagdish (2012)	2	1	24.5	25	-	41	16	16	170	233
Jagdish (2012)	2	1	24.5	25	-	41	16	16	176	241

Table 2 – Description of Test Specimens

Specimen	Test Capacity (kN)	Predicted Capacity (kN)	Test / Predicted
Huns (2002)	701	644	1.09
Huns (2002)	692	692	1.00
Mullin (2002)	631	579	1.09
Mullin (2002)	1078	1009	1.07
Hardash and Bjorhovde (1984)	243	251	0.97
Hardash and Bjorhovde (1984)	375	390	0.96
Udagawa and Yamada (1998)	690	625	1.11
Udagawa and Yamada (1998)	677	664	1.02
Udagawa and Yamada (1998)	677	687	0.98
Jagdish (2012)	268.40	233.527439	1.15
Jagdish (2012)	182.80	152.5517717	1.1983
Jagdish (2012)	176.80	147.5376	1.1983
Jagdish (2012)	178.90	149.293424	1.1849

REFERENCES

Chakrabarti, S.K. and Bjorhovde, R. (1983) Tests of Full Size Gusset Plate Connections. Research Report, Department of Civil Engineering, the University of Arizona, Tucson, Arizona. | Cheng, J.J.R., Yam, M.C.H. and Hu, S.Z. (1991) Elastic Buckling Strength of Gusset Plate Connections, Journal of Structural Engineering, 120: 538-559. | Davis, C.S. (1967) Computer Analysis of the Stresses in a Gusset Plate. M.Sc. Thesis, University of Washington, Seattle. | Epstein, H. I. and Chamarajanagar, R. (1994) Finite Element Studies for Correlation with Block Shear Tests. Computers and Structures, Vol. 61, No. 5 pp. 967-974. | Hardash, S. G., and Bjorhovde, R. (1984) Gusset Plate Design Utilizing Block-Shear Concepts, the Department of Civil Engineering and Engineering Mechanics, the University of Arizona, 85721, Tucson, Arizona. | Hibbitt, Karlsson and Sorenson Inc. (1998)"ABAQUS/Standard." Theory Manual, Version 5.8, Pawtucket, R.I. | Hu, S. Z., and Cheng, J. J. R. (1987) Compressive Behaviour of Gusset Plate Connections, Structural Engineering Report No. 153, Department of Civil Engineering, University of Alberta, Edmonton, Alberta. | Huns, B. B. S. (2002) Unpublished test data. Department of Civil & Environmental Engineering, University of Alberta, Edmonton, Alberta... | Khoo, H. A., Cheng, J. J. R., and Hrudey, T. M. (2000) Ductile Fracture of Steel, Structural Engineering Report No. 232, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alberta. | Kulak, G. L., and Grondin, G. Y. (2001) Block Shear Failure in Steel Members – A Review of Design Practice, Engineering Journal, American Institute of Steel Construction, 38: 199-203. | Menzemer, C. C. and Srivatsan, T. S. (1999). Aluminium Gusset Plate in Tension, Proceedings of the 1999 Structures Congress, New Orleans, LA, USA, 893-896. | Mullin, D. (2002) Unpublished test data. Department of Civil & Environmental Engineering, University of Alberta, Edmonton, Alberta. | Nast, T. E., Grondin, G. Y., and Cheng, J. J. R. (1999) Cyclic Behaviour of Stiffened Gusset Plate-Brace Member Assemblies, Structural Engineering Report 229, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Alberta. | Rabinovitch, J. S., and Cheng J. J. R. (1993) Cyclic Behaviour of Steel Gusset Plate Connections, Structural Engineering Report 191, Department of Civil Engineering, University of Alberta, Edmonton, Alberta. | Udagawa, K. and Yamada, T. (1998) Failure Modes and Ultimate Tensile Strength of Steel Plates Jointed with High-Strength Bolts, J. Struct. Constr. Eng., AIJ, No. 505, pp 115-122. | Varsarelyi, D.D. (1971) Tests of Gusset Plate Models. Journal of the Structural Division, ASCE, February, pp. 665-678. | Walbridge, S.S., Grondin, G. Y. and Cheng. J. J. R. (1998). An Analysis of the Cyclic Behaviour of Steel Gusset Plate Connections, Structural Engineering Report No. 225, Department of Civil & Environmental Engineering, University of Alberta, Edmonton, Alberta. | Williams, G.C. and Richard, R.M. (1986) Steel Connection Design Based on Inelastic Finite Element Analysis. Report of the Department of Civil Engineering and Engineering Mechanics, University of Arizona. | Yam, C.H.M. and Cheng, J.J.R., (1993) Experimental Investigation of the Compressive Behaviour of Gusset Plate Connections." Structural Engineering Report No. 194, Department of Civil & Environmental Engineering, University of Alberta, Edmonton, Alberta.