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Finite element modeling of block shear failure in coped steel beams

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Abstract

Block shear is a potential failure mode that is encountered in the connection regions of coped steel beams. Limited experimental studies completed so far have shown that the block shear failure in coped steel beams is a complex phenomenon, which is highly dependent on the number of bolt lines. In this paper, the use of the finite element method in predicting the block shear failure load was studied by making comparisons with experimental findings. The effects of numerical modeling details on load capacity predictions were investigated. In light of these investigations, a finite element analysis methodology has been developed and used to conduct a parametric study. Simplified load capacity prediction equations were developed based on the results of the parametric study and are presented herein.

Keywords: Block shear; Finite elements; Structural steel; Coped beam

1. Introduction

Wide flange beams that transversely frame into each other are usually connected by making use of a detail consisting of bolts and clip angles. The top flange of one of the beams is removed (coped) to provide leveling and compatibility between the structural members (Fig. 1). This kind of connection detail is prone to cause block shear failure of the coped beam web.

Design specifications [1–4] provide equations to predict the block shear capacity of structural connections. Although the equations of different specifications have variations in format, the basis remains the same. The block shear failure mode combines tensile and shear strength on planes that are perpendicular to each other.

In AISC-LRFD [1], which is one of the most widely used specifications in North America, the nominal block shear resistance (R_n) is calculated as follows:

When $F_u A_{nt} \geq 0.6 F_u A_{nv}$

$$R_n = [0.6 F_y A_{gv} + F_u A_{nt}] \leq [0.6 F_u A_{nv} + F_u A_{nt}]. \quad (1)$$

When $F_u A_{nt} < 0.6 F_u A_{nv}$

$$R_n = [0.6 F_u A_{nv} + F_y A_{gt}] \leq [0.6 F_u A_{nv} + F_u A_{nt}] \quad (2)$$

where A_{gv} is the gross area subject to shear; A_{gt} is the gross area subject to tension; A_{nv} is the net area subject to shear; A_{nt} is the net area subject to tension; F_u is the tensile strength of steel; F_y is the yield stress of steel.

The development of the AISC-LRFD specification [1] equations is based on the premise that when one plane, either the tension or the shear, reaches its ultimate strength, the other plane develops full yield. Based on this premise two failure mechanisms are possible and the one having a larger fracture strength term is assumed to govern the design.

Similar expressions are presented in Canadian [2], European [3], and Japanese [4] design specifications. In a recent study, Franchuk et al. [5] have studied the accuracy of the design equations by making comparisons with the findings from coped steel beam experiments. It was found that the equations given in the design codes could significantly overestimate the test failure loads. This overestimation was much more pronounced when two-line connections were considered as opposed to one-line connections. The aforementioned design equations for block shear were found to have drawbacks in predicting the failure mode of the specimens. Based on laboratory observations and statistical studies, a method that takes into account the number of bolt lines in calculating block shear load capacity was proposed by Franchuk et al. [5] and is given in Eq. (3). In the development of this equation it is assumed that when the tension plane fractures the average shear stress on the gross shear plane is the average of the rupture and

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yield stresses in shear. This assumption is consistent with the laboratory observations where the rupture occurs on the tension face after yielding has occurred on the shear face, but prior to shear rupture.

$$R_n = R_t A_{\mathrm{nt}} F_u + A_{\mathrm{gv}} \left(\frac{F_y + F_u}{2\sqrt{3}} \right) \tag{3}$$

where $R_t = \begin{cases} 0.9 & \text{for one-line connections} \\ 0.3 & \text{for two-line connections.} \end{cases}$

As can be understood from the above discussion, most of the load capacity prediction equations are empirical and are not based on mechanical principles. In an earlier study Topkaya [6] demonstrated that block shear load capacity equations for tension members could be found by making use of finite element analysis. In this paper, the finite element modeling that was adopted by Topkaya [6] is further improved to study the block shear failure behavior of coped steel beams. Effects of numerical modeling details on load capacity prediction of tested beams are investigated. Based on these observations a finite element analysis methodology was developed and used to conduct parametric studies. The results of the parametric study along with the design recommendations are presented.

2. Previous finite element studies on block shear

Finite element analysis was used in the past to study the behavior of structural members subject to block shear failure. Epstein [7–9] and his colleagues at the University of Connecticut studied the block shear behavior in tension members. Angles with and without staggered holes and WT members were analyzed. Rather than predicting the failure load of the specimens, their study focused on comparing the nondimensionalized finite element results for different connection geometries. A strain based criterion was used to determine the failure load of the member. Tension members were modeled with either shell elements or 20-node bricks and an elastic perfectly plastic stress–strain behavior for structural steel was used. Ricles and Yura [10] examined the block shear failure in coped beams using a two dimensional elastic analysis. The results revealed that the tension plane is subjected to a nonuniform stress distribution and, based on this observation, a modified block shear model was proposed. Barth et al. [11] conducted finite element analyses to directly predict the block shear capacity of WT tension members. Very elaborate analyses were performed which included the geometric and material nonlinearities as well as the surface-to-surface contact between the tee and the gusset plate. Eight node incompatible hexahedral elements were used to model the tee sections. A trilinear true-stress true-strain curve was used to represent nonlinear material effects. The Newton Raphson method was used to trace the load–deflection behavior beyond the limit point. The load corresponding to the load limit point was considered as the failure load. In a recent study Topkaya [6] used a similar yet less detailed analysis technique outlined by Barth et al. [11] to conduct a parametric study on block shear failure of steel tension members. Because the analysis methodology presented in this study is further improved to accommodate the failure predictions of coped beams, the details will be presented in the following sections.

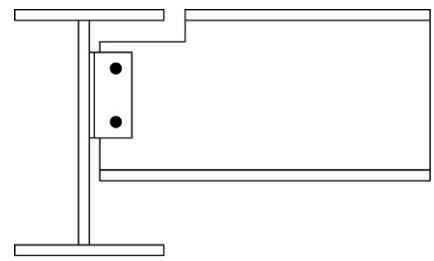

Fig. 1. A typical coped beam connection.

3. Previous experimental studies

Ricles and Yura [10] studied the behavior of double row bolted web connections. Six of the coped specimens had standard holes and failed in block shear. Primary variables investigated were the end and edge distance, bolt pitch, and bolt configuration. The specimens had edge and end distances of 25 mm and 50 mm. Connections had 21 mm standard holes and the holes were spaced 75 mm and 150 mm apart from each other. Two different grades of steel were used during the experimental program. For the first grade the yield and ultimate strengths were 264 MPa and 411 MPa, respectively. For the second grade the yield and ultimate strengths were 252 MPa and 400 MPa, respectively.

Franchuk et al. [12] systematically studied the behavior of single and double row bolted web connections. Sixteen of the specimens failed in block shear. The comprehensive study looked into the effects of end and edge distance, bolt diameter, section depth, number of bolt rows, number of bolt lines, and end rotation. End distances of 25 mm and 32 mm and bolt pitch of 75 mm and 102 mm were considered. Specimens had yield strengths ranging from 355 MPa to 378 MPa and ultimate strengths ranging from 429 MPa to 523 MPa.

Experimental data that was produced by these two independent research teams will be used in the finite element simulations in the following sections. A summary of the experimented connection geometries are given in Table 1 along with the AISC [1] predictions and experimental failure loads.

4. Simulation of previous experiments with finite element analysis

Finite element analysis was employed to predict the failure loads of the aforementioned tested specimens. A general purpose finite element program ANSYS [13] was used to perform the analysis. A typical block shear experiment for a coped beam consists of a three point bending setup. The coped portion of the beam is connected to a column using clip angles or connection plates. The load is applied vertically to the beam at a location close to the coped end. Usually a support is placed underneath the other end of the beam.

For all of the tested specimens mentioned above a finite element mesh is constructed (Fig. 2). The beam is

Table 1
Properties of analyzed specimens and analysis results

	Specimen number	F_y (MPa)	F_u (MPa)	Web thickness (mm)	Hole diameter (mm)	Connection type	End distance (mm)	Edge distance (mm)	Hole spacing (mm)	Spacing b/w bolt lines (mm)	AISC prediction (kN)	Test failure load (kN)	FEM prediction (kN)			
													$\mu = 0$	$\mu = 0.25$	$\mu = 0.5$	$\mu =$ infinity
One-line	1	367	513	7.0	21	2	25	25	75	NA	437	475	429	455	482	535
	2	366	516	7.0	21	1	25	25	102	NA	435	402	346	386	402	480
	3	377	523	7.0	27	1	32	32	103	NA	442	448	402	428	443	507
	4	370	522	7.0	21	2	25	50	75	NA	516	568	526	556	576	621
	5	370	522	7.0	21	2	50	25	75	NA	495	517	465	493	510	566
	6	368	517	7.0	21	1	25	25	75	NA	318	324	294	314	329	365
	7	368	479	7.5	21	1	25	25	75	NA	316	383	303	321	330	367
Two-line	8	366	516	7.0	21	4	25	25	102	75	630	537	596	627	652	690
	9	378	515	7.0	21	3	25	25	75	75	395	338	357	381	394	416
	10	264	411	11.2	21	5	50	25	75	75	769	494	628	656	672	705
	11	264	411	11.2	21	5	50	50	75	75	843	676	767	812	831	875
	12	252	400	10.9	21	3	50	25	150	75	726	493	603	617	636	662
	13	252	400	10.9	21	3	50	50	150	75	794	583	705	732	760	797
	14	252	400	10.9	21	6	25	25	75	75	604	449	577	602	615	630
	15	252	400	10.9	21	6	50	50	75	75	755	596	773	818	834	861

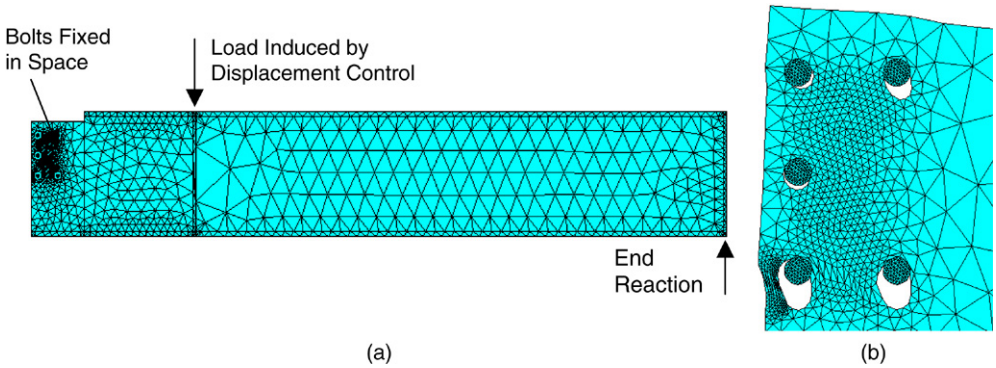

Fig. 2. Representative finite element analysis of a coped beam (a) Model of the beam. (b) Deformed shape of the connection region.

modeled using six node triangular plane stress elements with thickness. This kind of element enables the modeling of different thicknesses possessed by the web and the flanges. In the preliminary analysis, the connection plates were also modeled but results showed that including this detail does not significantly alter the behavior.

An important decision on modeling details is the way end reactions provided by the bolts are addressed. In an earlier study on tension members [6] nodes that lie on the half circumference of each hole where bolts come into contact were restrained against displacement in two perpendicular directions. This kind of a modeling detail was found to be acceptable for tension members where the block rotation is limited. On the other hand, for coped beams the end detail considered is not only subjected to shear but also to significant amount of bending. The bending created on the detail has influence on the block rotations. For this reason, boundary conditions should be treated in a more elaborate fashion. This requires the inclusion of contact behavior between the bolts and the bolt holes. In all models the bolts are modeled with six node triangular plane stress elements. The bolts are assumed to be fixed in space. Therefore, the movements of the nodes that lie on the bolt area are constrained in two perpendicular directions. A contact algorithm that is based on augmented Lagrangian method is employed for the analysis. A surface-to-surface contact is defined by assigning target elements to bolt surfaces and contact elements to hole surfaces. A penetration tolerance of 0.1 is selected which allows a certain amount of penetration between the contacting surfaces based on the representative element size. The friction between the contacting surfaces is included in modeling and the details will be presented in later sections of the paper.

Geometric and material nonlinearities are included to capture the necking response exhibited by the experimented specimens. The nonlinear stress–strain behavior of steel is modeled using the Von Mises yield criterion with isotropic hardening. A generic true-stress true-strain response is used in all analyses. The generic response given in Fig. 3 consists of four parts. The material behaves linearly until it yields. A yield plateau extends up to a true-strain value of 0.02. After the end of the yield plateau strain hardening commences and the true-stress increases linearly until the true ultimate stress is reached. The true-strain at the true ultimate stress is assumed

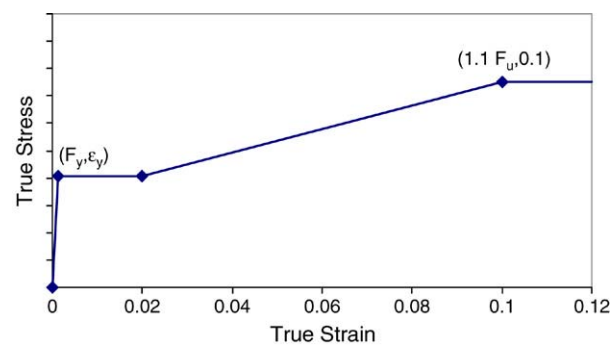

Fig. 3. Generic true-stress true-strain material response for steel.

to be 0.1. After this point there is a constant stress plateau until the material breaks at an acceptable failure strain.

The loading applied during the experiments is simulated using a displacement boundary condition at the load location. The Newton–Raphson method is used to trace the entire nonlinear load–deflection response. The total load on the connection is found by subtracting the end reaction value from the total applied load. It is observed that the load–deflection diagram peaks at a certain point due to necking of the net tension plane near the vicinity of the leading bolt hole. This observation is consistent with the observations gathered from the experiments.

As mentioned before this study aims to predict the block shear failure load of the tested specimens rather than providing qualitative comparisons between different connection geometries. In predicting load capacities, uncertainties in numerical modeling and quantification may arise. These uncertainties may be due to two sources; namely friction modeling and the failure criterion. In reality, a certain amount of friction develops between the surfaces of bolts and bolt holes. This fact has been neglected in the previous numerical studies on block shear. However, in this study it will be presented that frictional contact response has influence on the load capacity predictions especially in the case of coped beams where significant amounts of block rotation is expected. In addition, selection of an appropriate failure criterion presents an additional challenge. For structural members that are subject to pure axial load the block shear failure is triggered by uniform necking of the net tension plane. On the other hand, for coped steel beams the loading pattern is much more complex. The so

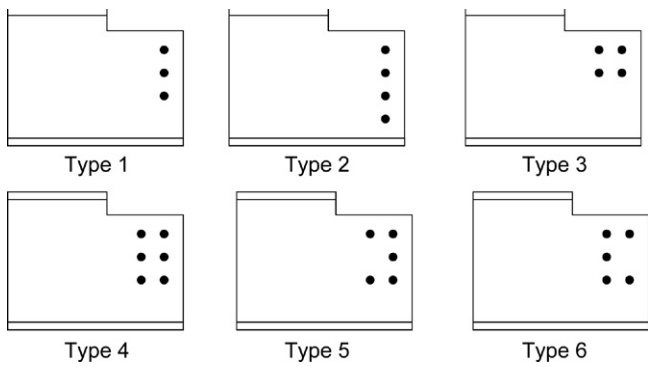

Fig. 4. Experimented connection geometries.

called "net tension plane" is under the action of multiaxial states of stress. Due to this fact the necking of the net tension plane is not uniform thereby increasing the likelihood of an earlier failure than expected for the case of uniform necking.

A total of 15 connection geometries for which the experimental failure loads are known were analyzed under different friction assumptions. The coefficient of friction (μ) between the surfaces of the bolts and bolt holes were considered to be 0, 0.25, 0.5, and infinity. The μ = infinity case corresponds to the non-slip contact. The load–displacement response was monitored beyond the limit point for all cases. The maximum load was considered to be the failure load. The predicted load capacities based on this failure criterion and different coefficient of friction values are presented in Table 1 along with the experimental failure loads. The six connection types mentioned in Table 1 are given in Fig. 4.

It is obvious from the analysis results that the predicted load capacities increase with the increase in the coefficient of friction. This increase is much more pronounced for one-line connections as opposed to two-line connections. Increasing the coefficient of friction from zero to infinity results on average in 25% and 13% increases in ultimate load for one-line and two-line connections, respectively. For a coefficient of friction value of 0.25, average increases of 7% and 5% in ultimate load are observed for one-line and two-line connections, respectively. The respective numbers for the $\mu = 0.5$ case would be 11% and 8%. The typical coefficient of friction values vary between 0.25 and 0.5 for steel to steel surface contact.

It is clear from the above discussion that the frictional response should be known for accurate prediction of the failure load. For design purposes, however, the formation of frictional forces could not be relied on. Therefore, any developed design equation should provide conservative estimates with the neglect of frictional behavior where friction, if there is any, can be taken as reserve strength. The comparison of experimental findings with the finite element analysis load capacity predictions without friction ($\mu = 0$) is given in Fig. 5. In Fig. 5 experimental failure loads are plotted against the finite element analysis predictions. One-line and two-line connections are presented separately. Data points appearing above the diagonal line indicate tests for which finite element analysis predictions are conservative (load capacity is underestimated) while points below the diagonal line indicate unconservative load

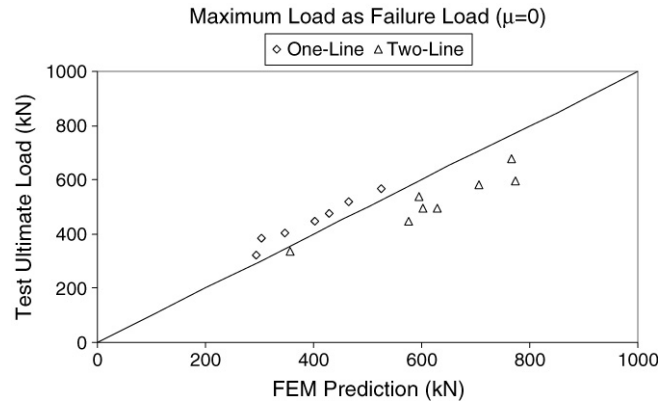

Fig. 5. Comparison of finite element analysis predictions with experimental findings—maximum load is considered as the failure load.

Table 2
Professional factor statistics

	Professional factor		
	Failure criterion		
	Maximum load	Average horizontal strain	Tension plane yield
All data			
Mean	0.977	1.195	1.191
Standard deviation	0.164	0.171	0.125
Maximum	1.264	1.473	1.434
Minimum	0.771	0.975	1.022
One-line			
Mean	1.134	1.358	1.302
Standard deviation	0.062	0.062	0.065
Maximum	1.264	1.473	1.434
Minimum	1.080	1.279	1.245
Two-line			
Mean	0.839	1.052	1.094
Standard deviation	0.064	0.071	0.067
Maximum	0.947	1.146	1.220
Minimum	0.771	0.975	1.022

predictions. It is evident from the figure that the load capacities of one-line connections are underestimated while the capacities of two-line connections are overestimated. For statistical analysis a professional factor (experimental load divided by the predicted load) was calculated for every case. A perfect agreement between the predicted load and experimental failure loads are expressed by a professional factor of unity. Factors less than unity and greater than unity represent overestimation and underestimation of the failure load, respectively. The statistical analyses of the predictions are presented in Table 2.

Statistical evaluation revealed that finite element analysis results are promising in terms of predicting the block shear load capacity of coped steel beams. The mean of the professional factors is close to unity. On the other hand there are overestimates and underestimates in load capacities on the order of 25%. The underestimations which are specific to one-line connections could be attributable to the neglect of

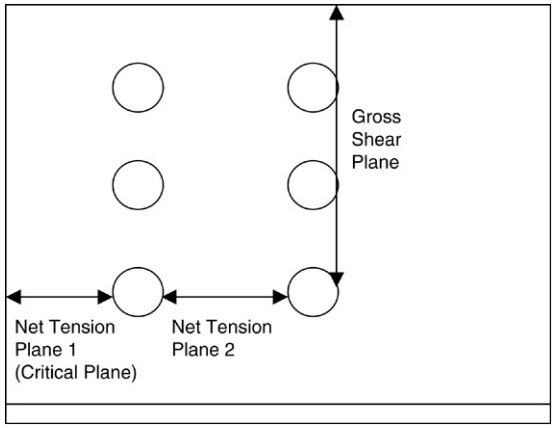

Fig. 6. Net tension, gross shear, and critical planes.

frictional forces. These do not pose any problems in terms of design. However, overestimates of failure loads which are particular to two-line connections are undesirable from a design point of view. This study aims to provide simple load capacity prediction equations based on finite element analysis results. Therefore, a conservative prediction of the failure load is essential. For this purpose the failure criterion should be updated towards obtaining conservative load capacities.

As mentioned earlier the necking response in coped steel beams is not uniform. The necking area is subjected to nonuniform tensile and shear stresses. A detailed investigation in the vicinity of the necking region revealed that very high local strains are produced even before the ultimate load is reached due to the complexity of loading. These very high local strain demands cause the region to neck in a nonuniform fashion and could lead to failure at a load level lower than the ultimate load indicated by the finite element analysis. Investigations and trials revealed that a failure criterion based on local points might be misleading due to the drastic change in stress and strain patterns in the vicinity of the necking region. Due to this reason two other failure criteria that are based on more global measures than local measures are proposed herein. The next section focuses on the development of these failure criteria and the finite element analysis predictions based on those.

4.1. Investigation of other failure criteria

4.1.1. Average horizontal strain

As explained before a global measure to quantify the failure load is sought. One way of monitoring the necking response is through the calculation of the average horizontal strain. Necking is the gradual decrease of the tension plane width. This observation suggests that the reduction in the width of the critical plane could be an indicator of failure. Here the critical plane is the portion between the edge of the beam and the edge of the leading bolt hole as depicted in Fig. 6. The change in length along the critical plane could be monitored by subtracting the deformations of the end points. Later this deformation could be divided by the original width to obtain an average horizontal strain at that critical location.

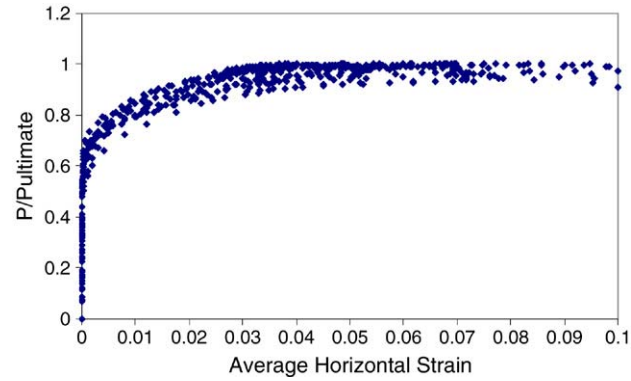

Fig. 7. Connection load versus average horizontal strain ($\mu = 0$).

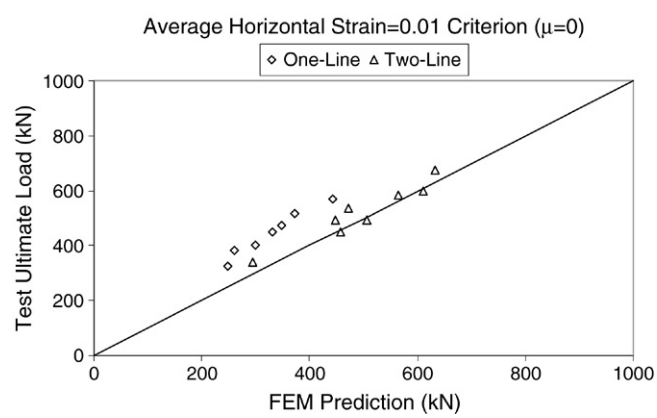

Fig. 8. Comparison of finite element analysis predictions with experimental findings—average horizontal strain criterion.

For all the 15 specimens analyzed the average horizontal strain was monitored for every load step. Responses from all analyses are presented in Fig. 7. In this figure the load value is normalized by the ultimate load and plotted against the average horizontal strain. Examination of Fig. 7 reveals that all curves show a similar trend and the data falls within a narrow band. In most of the cases the horizontal strains start to increase after 60% of the ultimate load is reached. At this point a selection has to be made to quantify the failure loads for the analyzed cases. Examination of that data revealed that if an average horizontal strain of 0.01 is chosen as a failure criterion, conservative estimates could be obtained. Fig. 8 presents experimental loads versus the predicted capacities based on this failure criterion. Professional factor statistics for this proposed failure criterion are also given in Table 2. It is evident from Fig. 8 and Table 2 that the failure criterion adopted provides conservative estimates and could be used in developing block shear load capacity prediction equations. One major drawback of this criterion is that the average horizontal failure strain of 0.01 has no physical support.

4.1.2. Tension plane yield

The failure loads obtained through applying the average horizontal strain criterion is dependent on the selection of the critical failure strain. A criterion based on stresses could be used if the failure is required to be based on measured quantities.

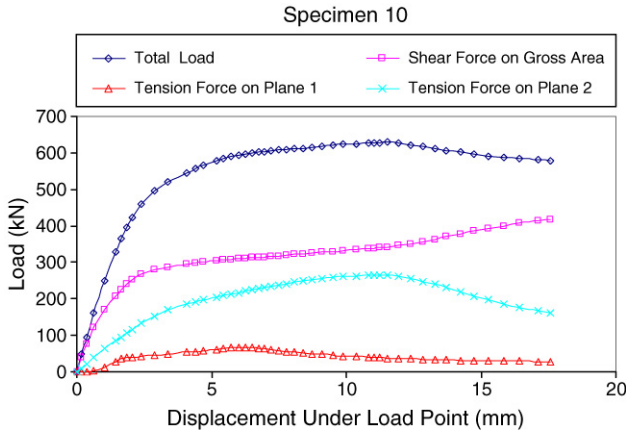

Fig. 9. Load versus displacement response for a typical two-line connection.

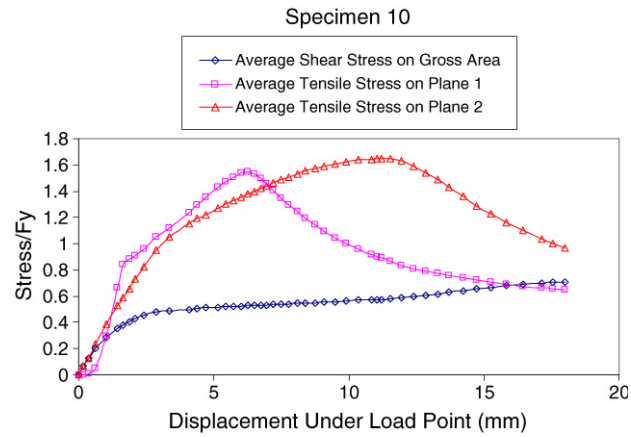

Fig. 10. Stress versus displacement response for a typical two-line connection.

As mentioned before the detailed examination of the normal stress and strain history revealed that the region of interest is the critical necking plane (Fig. 6) that was explained before. The trace of the stress history showed that the normal stress distribution on the critical plane is nonuniform. Some parts of the plane yields earlier than the others. At some point in the loading history the critical plane fully yields and some portions experience strain hardening. Beyond this point on there are large local strain demands observed at some locations along the critical plane. This suggests that if a conservative failure criterion is sought then one can limit the average normal stress value on the tension plane with the yield stress. After the attainment of full yield it will be difficult to quantify the excess capacity based on large local strain demands.

In order to observe the applicability of this failure criterion the average stresses on the tension and shear planes were monitored. For this purpose paths were defined on the net tension and gross shear planes (Fig. 6). After each load step the normal stress variation on tension plane(s) and the shear stress variation on the gross shear plane were monitored. The variation of the stress distribution on any plane is integrated to find out the total force acting on that plane. Then, this total force is divided by the area of the plane to calculate the average stress. The variation of the forces and average stresses on the net tension and shear planes for a two line connection is given in Figs. 9 and 10, respectively. An important observation gathered from Fig. 9 is that the ultimate value for the critical net tension plane load and that of the total connection load do not occur at the same load step. Usually the ultimate load on the critical net tension plane is reached before the ultimate connection load. This observation indicates that for the ultimate connection load to be reached the net critical tension plane should experience very high local strains.

For all the 15 specimens analyzed the average normal stress on the net tension plane(s) and the average shear stress on the gross shear plane were monitored. Failure load is considered as the load corresponding to the load step where the average normal stress on the critical tension plane reaches the yield stress. Fig. 11 presents the experimental loads versus the predicted loads based on this failure criterion. Professional factor statistics for this proposed failure criterion

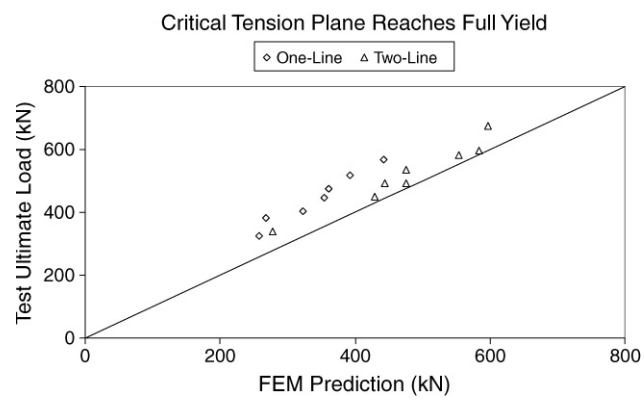

Fig. 11. Comparison of finite element analysis predictions with experimental findings—tension plane yield criterion.

are also given in Table 2. Examination of Fig. 11 and Table 2 revealed that this new failure criterion could be used to obtain conservative estimates of the block shear load capacity. This failure criterion has the superiority over the others that it is based on measured physical quantities and provides estimates with certain conservatism. Therefore, for the parametric study that is going to be used for developing capacity expressions this failure criterion will be adopted.

5. Parametric study

A parametric study has been conducted to develop simple block shear load capacity equations. The analysis methodology explained earlier was adopted. In order to provide comparisons, both the maximum load and the tension plane yield criterion were used. Edge distance, number of bolt lines, number of bolt rows, bolt pitch, spacing between bolt lines, yield to ultimate ratio were considered as the variables of the parametric study. A total of 90 analyses were conducted. In all analyses the complete beam (W410 × 46) was modeled and the dimensions of the beam and test setup were identical with the dimensions used in the experimental study of Franchuk et al. [12]. For some cases the web height was increased to prevent failure from pure shear yielding of the beam. The analyzed specimens had two, three, and four bolt rows each case having either one or two

Table 3
Combinations of the variables used in the parametric study

Bolt rows	Bolt lines	Edge distance (mm)	Pitch (mm)	Spacing (mm)	F_u/F_y
2	1	25	45/60/75	NA	1.68/1.2
3	1	25/35	45/60/75	NA	1.68/1.2
4	1	25/35	45/60/75	NA	1.68/1.2
2	2	25/35	45/60/75	50	1.68/1.2
3	2	25/35	45/60/75	50/75	1.68/1.2
4	2	25/35	45/60/75	50/75	1.68/1.2

bolt lines. Bolt pitch values of 45 mm, 60 mm, and 75 mm were considered. Specimens had edge distance values of 25 mm and 35 mm and the spacing between bolt lines was taken as 50 mm and 75 mm. All specimens had a constant bolt hole diameter of 14 mm and an end distance of 25 mm. The ultimate strength of steel was kept constant at a value of 352 MPa and two yield stress values of 210 MPa and 293 MPa were considered. These values produced ultimate to yield ratios of 1.68 and 1.20. The combinations of the variables considered in the study are given in Table 3.

For all analyses the connection load was monitored beyond the limit point by considering a coefficient of friction value of zero. After each load step the average normal stress on net tension plane(s) and the average shear stress on the gross shear plane were documented. Based on the analyses results a prediction equation considering a critical tension plane yield criterion was developed. For two-line connections the average normal stress on the tension plane between two leading bolt holes were examined. The analyses results revealed that the average normal stress on this plane varies between 0.94 to 1.11 of the yield stress (F_y) with an average of $1.03F_y$. This suggests that the critical tension plane and the tension plane in between the leading bolt holes reach to yield stress roughly at the same load step (Fig. 10). Therefore, for two-line connections no special treatment of the tension plane stress is necessary. After this observation the average shear stress on the gross shear area was examined. Fig. 12 presents the variation of average shear stress normalized by the yield stress as a function of the connection length (distance between the center of leading bolt hole to coped portion of the beam; the same distance used to calculate the shear area). In Fig. 12 the data points for the two ultimate to yield ratios were separately given. Examination of this figure reveals that the average shear stress is not very much dependent on the connection length and ultimate to yield ratio. This observation suggests that a single shear stress value could be used in developing a capacity expression. The mean of the average shear stress values on the gross shear plane is $0.486F_y$ based on the finite element analyses. Therefore, using the above discussions and proper rounding off, the following equation could be proposed to calculate block shear load capacity of coped steel beams:

$$R_n = F_y A_{\mathrm{nt}} + 0.5 F_y A_{\mathrm{gv}}. \tag{4}$$

In order to compare the outcomes of using different failure criteria an equation based on the maximum load criterion was developed. For this purpose, it was assumed that the whole

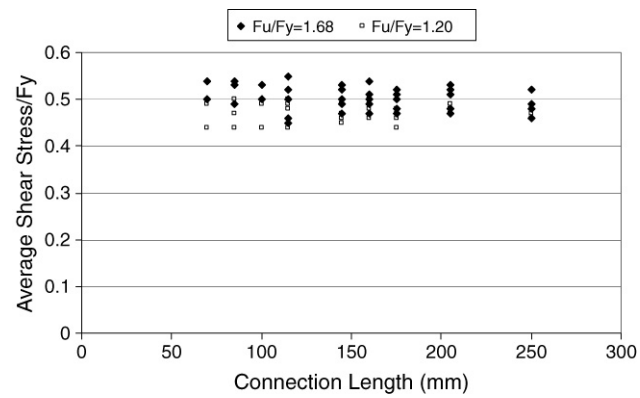

Fig. 12. Average shear stress as a function of connection length (critical net tension plane reaches yield criterion).

net tension plane reaches the ultimate stress at failure. The next task is to compute the effective shear stress on the gross shear area based on this assumption. This is accomplished by subtracting the tension plane contribution from the total maximum load observed from the analysis. Later the calculated effective shear stress values were normalized by either the yield stress or the ultimate stress. Examination of the data revealed that the standard deviation of the effective shear stress is lower if normalized by the ultimate stress when compared to the case where the normalization is performed with respect to the yield stress. Fig. 13 presents the effective shear stress values normalized by the ultimate stress as a function of connection length. It is observed that the effective shear stress is not significantly dependent on the connection length. Therefore, a single value could be used to represent the effective shear stress occurring on the gross shear plane. The mean of the effective shear stress is 41% of the ultimate stress based on the finite element analyses results. By proper rounding off, the following expression could be proposed to calculate the block shear load capacity of coped steel beams:

$$R_n = F_u A_{\mathrm{nt}} + 0.4 F_u A_{\mathrm{gv}}. \tag{5}$$

The quality of the prediction equations were assessed by making comparisons with the experimental findings. Two sets of experimental data that were mentioned before were used for comparison purposes. Figs. 14 and 15 present the comparisons of capacity predictions and test results. The professional factor statistics for the developed equations are given in Table 4. It is evident from Figs. 14 and 15 and Table 4 that Eq. (4) which is based on the critical tension plane yield criterion provides conservative estimates. On the other hand, Eq. (5), which is based on the maximum load criterion, overestimates the capacity of some beams especially the ones with two-line connections. Based on this discussion Eq. (4) is recommended for design of coped beams against block shear failure. Use of this equation will provide conservative capacity estimates for both one-line and two-line connections.

6. Conclusions

A finite element study on block shear failure of coped steel beams was presented. Specimens tested by two independent

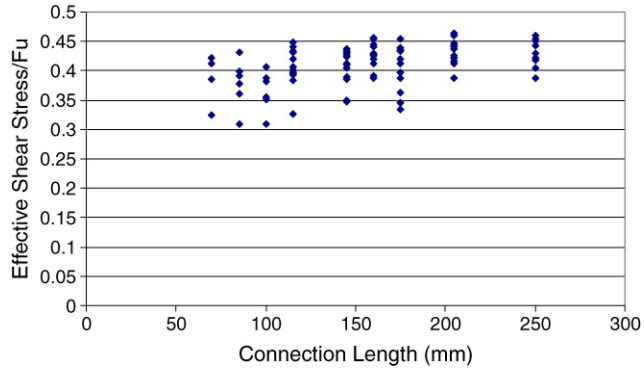

Fig. 13. Effective shear stress as a function of connection length (maximum load criterion).

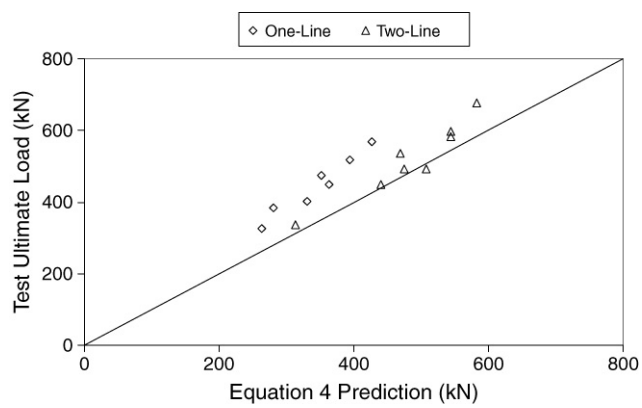

Fig. 14. Comparison of Eq. (4) predictions with experimental findings.

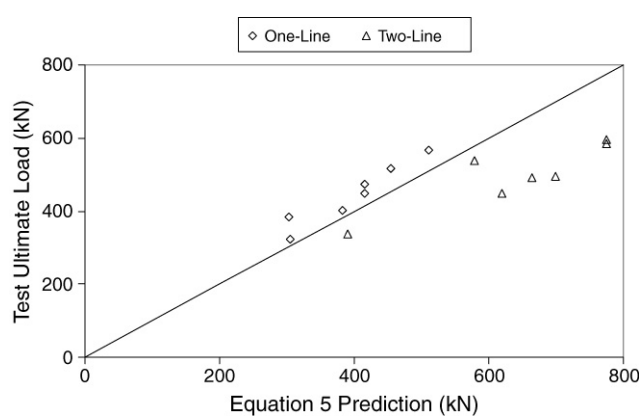

Fig. 15. Comparison of Eq. (5) predictions with experimental findings.

research teams were modeled and analyzed. During every analysis the ultimate load was monitored beyond the limit point. The effects of failure criterion and friction modeling on the prediction of ultimate connection capacity were investigated. First the ultimate load was quantified by the maximum load reached. Using this failure criterion experimented beams were analyzed under different friction assumptions. Second the ultimate load was quantified by using a failure criterion based on average strain on the critical net tension plane. Third the ultimate load was quantified based on an assumption that failure occurs when the critical net tension plane reaches full yield. The comparisons of the experimental loads and predicted capacities

Table 4
Professional factor statistics

	Professional factor	
	Eq. (4)	Eq. (5)
All data		
Mean	1.175	0.944
Standard deviation	0.128	0.185
Maximum	1.360	1.262
Minimum	0.972	0.707
One-line		
Mean	1.291	1.120
Standard deviation	0.061	0.072
Maximum	1.360	1.262
Minimum	1.216	1.049
Two-line		
Mean	1.072	0.790
Standard deviation	0.062	0.077
Maximum	1.159	0.929
Minimum	0.972	0.707

based on these failure criteria were presented. A parametric study was conducted to develop simple load capacity prediction equations. The quality of the developed equations was assessed by making comparisons with the experimental findings.
The following can be concluded from this study:

- Modeling frictional forces developing between the surfaces of bolts and bolt holes has influence on the computed block shear capacity. Friction should be modeled properly for accurate determination of the failure load. The effects of friction are much more pronounced for one-line connections as opposed to two-line connections. Neglecting friction yields conservative results.
- Using the maximum load as a failure criterion overestimates the capacity of two-line connections.
- Using an average strain on the critical net tension plane as a failure criterion could lead to conservative estimates of the connection capacity; however, this failure criterion lacks any theoretical or experimental evidence.
- The failure criterion based on limiting the ultimate load with the critical tension plane yield provides conservative estimates and is dependent on measured physical quantities. This failure criterion was adopted to develop a simple block shear load capacity prediction equation.
- The developed equation (Eq. (4)) based on a tension plane yield criterion provides load capacity estimates with certain conservatism. This equation can be an alternative to more traditional code equations.

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