Shear Capacity of High-Strength Bolts in Long Connections

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ABSTRACT

Current design codes reduce the shear strength of individual bolts to account for potentially uneven distribution of force among the bolts including a 0.75/0.90 (83.3%) step function at 38 in. Available test data indicate there is no justification for a bolt shear strength reduction, especially the step function, due to the length of connection, provided that second-order effects are limited and gross and net section areas slightly exceed the AISC *Specification* limits (2010). A practical, empirical solution is proposed that maintains a reliability, β, slightly greater than 4.0 for all connection lengths, using the current AISC resistance factor, ϕ, of 0.75.

Keywords: bolt shear, reliability, resistance factor, connection length.

BACKGROUND

The exact solution for the load distribution in a long, bolted connection was developed by Fisher and Rumpf (1965) and reported by Kulak et al. (1987) and Tide (2012a). Because the load-deformation relationships for the bolts and plates must be known, it is not a practical solution for design purposes. Therefore, empirical solutions have been developed for bolted connections.

The current empirical shear strength of a high strength bolt (Tide, 2010) may be expressed by the following equation:

$$P_n = P_u A_b R_1 R_2 R_3 \tag{1}$$

where

P_u = ultimate tensile strength of bolt, ksi
R_1 = 0.625, shear-to-tension ratio
R_2 = 0.90, initial connection length reduction factor for $L \le 38$ in.
= 0.75, connection length reduction factor for $L > 38$ in.
R_3 = 1.00, threads excluded from shear plane
= 0.80, threads included in shear plane
L = connection length between end bolt center lines, in.
A_b = nominal bolt area, in.^{2}

The design shear values for ASTM A325 and A490 bolts are given in RCSC Specification Table 5.1 (RCSC, 2014). The design values for other fasteners, such as ASTM A307 bolts and threaded material, are given in the AISC *Specification for Structural Steel Buildings*, hereafter referred to as the AISC *Specification* (AISC, 2010), Table J3.2. In load resistance and factor design (LRFD) terms, the design shear strength of a bolt is ϕR_n, with $\phi = 0.75$ and $R_n = P_n$. A step function with an 83.3% reduction exists at connection length equal to 38 in.

The design values are based on an extensive research program conducted by the steel industry at the Fritz Engineering Laboratory at Lehigh University from the 1950s through the early 1970s. As was the custom at the time, the high-strength bolts were fully pretensioned, and bolt threads were excluded from the shear plane. The test data were previously reported by Tide (2010, 2012a) in U.S. customary units and in S.I. dimensional units, respectively. The data are summarized in the *Guide to Design Criteria of Bolted and Riveted Joints* (the *Guide*) by Kulak et al. (1987) and will not be repeated in this paper.

The test data have also been used to evaluate and compare the bolt shear provisions of the Australian Code (Tide, 2012b) and the Eurocode provisions as found in Comite Europeen de Normalization (CEN, 2003) and Tide (2012a, 2014). Because the Canadian provisions (CSA, 2001, 2005) are similar to the Eurocode criteria, all of these provisions utilize a variable bolt diameter–dependent connection length factor instead of a step function, including an increase in unit strength with increasing bolt diameter.

CONNECTION TEST VARIABLES

All of the connections considered by Tide (2010) and in the *Guide* (Kulak et al., 1987) were loaded uniaxially to eliminate second-order effects, the bolts were pretensioned, and the threads were excluded from the shear plane. Moore et al. (2010) recommended a resistance factor, ϕ, of 0.85, based

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on the results of approximately 1500 tests that indicated theoretical resistance factors of 0.81 and 0.87 produce a reliability of 4 for the threads excluded and threads included conditions, respectively. This can be compared to the AISC resistance factor of 0.75. Empirical data indicate that bolts will be subjected to nearly uniform shear when designs comply with current *Specification* limit states. Bendigo et al. (1963) state:

> But, experimental work with riveted connections[9] has shown that successive yielding of the outer rivets produces a redistribution of load so that at failure a more uniform distribution exists than the elastic analysis indicates.

Reference "9" is the work presented by Davis et al. (1940). The *Guide* (Kulak et al., 1987), Section 5.2.6, pages 103 and 104, indicates that nearly equal load distribution occurs when the ratio of the plate net section to the connector shear area is large. This was confirmed by the author when the referenced papers were reviewed relative to the connection failures in long connections.

TEST DATA

Tide (2010) compiled test data from 10 papers and reports: Bendigo et al. (1963), Fisher et al. (1963), Fisher and Kulak (1968), Fisher and Yoshida (1970), Foreman and Rumpf (1961), Kulak and Fisher (1968), Power and Fisher (1972), Rivera and Fisher (1970), and Sterling and Fisher (1965, 1966). Because of the various reporting formats and test parameters, the results were not directly comparable. Instead, the published test ultimate shear strength of each connection was reduced to an average ultimate shear strength, P_{TEST}, of a single connector, bolt or rivet, loaded on two shear planes (double shear). The predicted ultimate shear strength of the same connector was computed using appropriate single shear connector test data multiplied by 2, P_{PRED}, for each lot of bolts or rivets.

The ratio P_{TEST}/P_{PRED} was then computed and entered into a database to compare the results with connection length as the only independent variable. Tide (2010, 2012a) presents the results, which are not repeated here. Though Tide included test results for Huck bolts and rivets, these fasteners are not considered in this paper.

The test data were then plotted as shown in Figure 1 after being conditioned according to the AISC *Specification* (2010) limit states of connection gross area and net area requirements, respectively. The specifications limit states were modified by a factor of 0.90. Development of these criteria is found in Tide (2010, 2012a). Conditions for which both the gross area (A_g) and net area (A_n) limit states are satisfied—the P_{TEST}/P_{PRED} data—are shown as circles in Figure 1. The plotted data are in a nondimensional form, eliminating the variability of bolt diameter, material type and connection configuration. When only one of the limit states is satisfied, the data are shown as triangles. When neither limit state is satisfied, the data are shown as squares.

The data plotted in this form clearly indicate that when the connection gross and net area limit states were satisfied, all bolts in the connection were approximately equally loaded to their maximum shear capacity. As shown in the Appendix of Tide (2010), this load condition occurs when the gross area (A_g) and net area (A_n) comply with the following:

$$A_g \geq 0.47 A_s F_u / F_{yp} \tag{2}$$

and

$$A_n \geq 0.56 A_s F_u / F_{up} \tag{3}$$

where

A_g = connection plate gross area, in.^{2}
A_n = connection plate net area, in.^{2}
A_s = total effective bolt shear area, in.^{2}
F_u = bolt ultimate tensile stress, ksi
F_{yp} = plate yield stress, ksi
F_{up} = plate ultimate tensile stress, ksi

This condition is implied when Figures 5.24 and 5.25 of the *Guide* (Kulak et al., 1987) are examined for large A_n/A_s ratios.

It has been shown by Tide (2010, 2012a, 2012b, 2014) that bolt diameter, current rivet and bolt material, and current plate material grades do not influence the connection capacity provided the specification limit states are satisfied. These limit states have been addressed when the plate material gross area, A_g, and net area, A_n, requirements were developed as shown in Equations 2 and 3, respectively. Therefore, these subjects will not be discussed further in this paper.

Ocel (2013) has addressed bolted and riveted connections designs in steel-framed bridges. A major effort of this work appears to address the gusset plates that connect the members together. The report is essentially silent on the historic step function for long connections that deals with the bolt or rivet ultimate shear capacity regardless of applicable gross and net area limits in the connections.

It should be noted that once the number of bolts are chosen for a particular connection that meet the gross and net area limit states, adding additional bolts to the connection has limited benefit. The failure mechanism location will change from the bolts and will subsequently occur in the connected material.

DATA CONDITIONING

A total of 119 connection tests were identified. Of these, 40 tests were with rivets associated with the design and construction of the San Francisco–Oakland Bay Bridge and contained insufficient information to be included in this review. Of the remaining 79 connection tests, the connector distribution was 54 A325 bolts, 18 A490 bolts, 5 rivets, and 2 Huck bolts. Shingle connection data were also removed from the database. Furthermore, it was stipulated that connection test results would only be considered provided that the limit states of gross area and net area were also satisfied. The statistical analysis was performed using the remaining 7 A325 and 11 A490 bolted connections. Because of the many connection variables, the test data were reduced to a nondimensional form to limit the significance of all the variables. As a result, the connection length remained as the desired and predominate independent variable.

In the previous papers by Tide (2010, 2012a), all of the test results were included in the database. Test data that were significantly below the specification limit states were used to determine the connection reliability and related resistance factor. Alternatively, Tide (2012b, 2014) chose the data whose test results mostly satisfied the gross area and net area limit states. As seen in Figure 2, the data were further divided into two distinct groups. The first group included nine test results having a connection length of 10.5 in. The second group included nine test results having connection lengths that varied from 21.0 in to 84.0 in. The relevant test results are given in Tables 1 and 2, respectively. The two data groups were separated because it was felt that the nine test results at 10.5 in. would unacceptably influence the reliability calculations of the other nine test results having significant variation in connection lengths.

REGRESSION ANALYSIS

Because the latter nine test data occurred over considerable connection lengths (L), the results can be combined using a regression analysis that represents the nine test data from which reliability analysis can be performed at discrete lengths. A linear least-square regression analysis produced the following relationship for P_{TEST}/P_{PRED}:

$$P_{TEST}/P_{PRED} = 1.0637 - 0.00092L \tag{4}$$

This linear regression analysis is graphically shown in Figure 2.

The negative slope to the regression line is small indicating that there is minimum variation in connection strength with connection length. Furthermore, the correlation coefficient is nominally low, at −0.458 and would be expected as there are no test replicates in the nine test results.

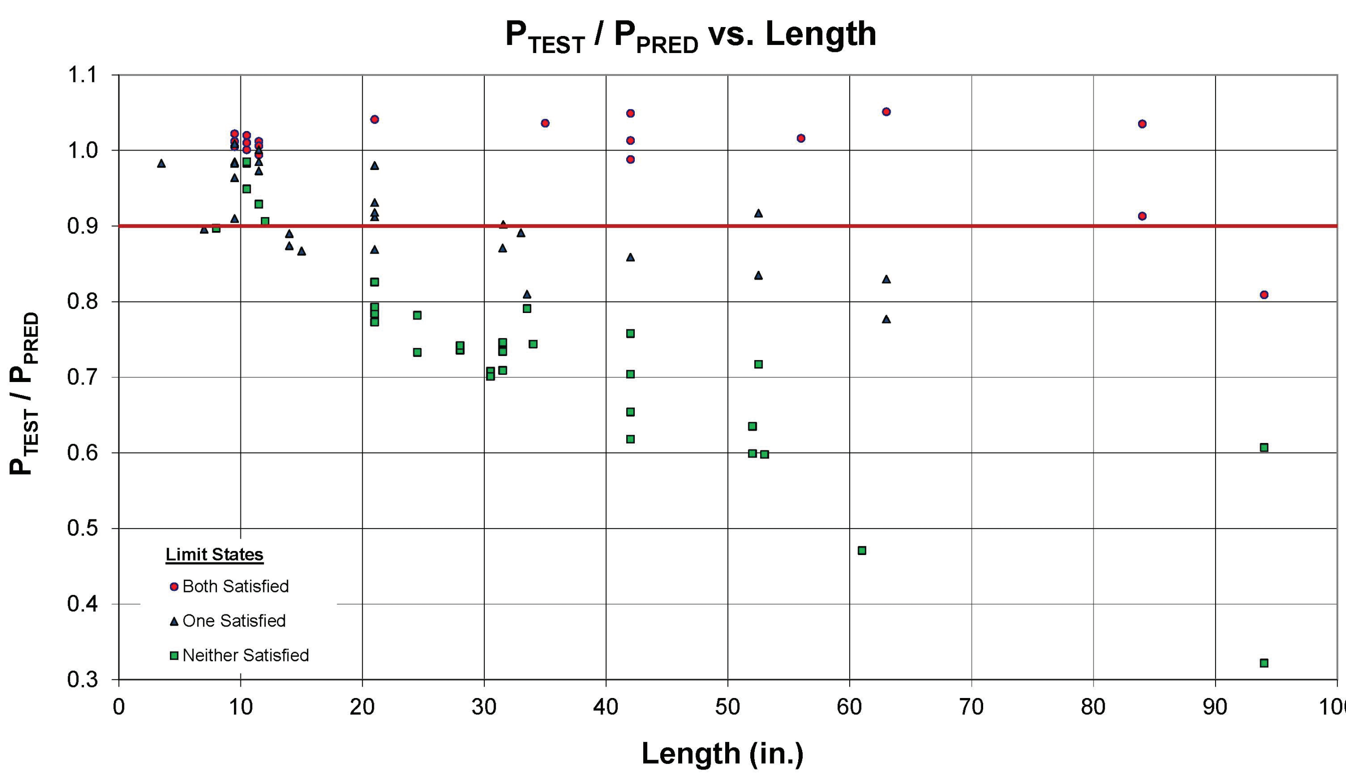

Fig. 1. Test data plotted indicating limit state considerations.

Table 1. Limit State Comparison for Compact Bolt Group Connections											
Test No.	Bolt Type	Bolts in Line	D (in.)	L (in.)	$\frac{P_{Test}}{P_{Pred}}$	A_g (in.^{2})	A_{gl}[(1)] (in.^{2})	A_g/A_{gl}	A_n (in.2)	A_{nl}[(2)] (in.^{2})	A_n/A_{nl}
1	A325	4	1⅛	10.5	1.001	13.0	8.3	1.52	8.07	7.8	1.04
2	A325	4	1⅛	10.5	1.012	13.8	8.3	1.66	8.9	7.8	1.14
3	A325	4	1⅛	10.5	1.005	14.5	8.3	1.75	9.66	7.8	1.24
4	A325	4	1⅛	10.5	1.010	15.4	8.3	1.86	10.5	7.8	1.35
5	A325	4	1⅛	10.5	1.022	16.3	8.3	1.96	11.4	7.8	1.46
11	A490	4	1	10.5	1.020	13.9	9.6	1.45	9.58	9.0	1.06
12	A490	4	1	10.5	1.012	14.6	9.6	1.52	10.3	9.0	1.14
13	A490	4	1	10.5	0.994	15.2	9.6	1.58	10.9	9.0	1.21
14	A490	4	1	10.5	1.006	16.0	9.6	1.67	11.6	9.0	1.29
Mean					1.009			1.663			1.214
Standard deviation					0.009			0.169			0.137

(1) $A_{gl} = 0.90 A_s F_{ub} / F_{yp}$
(2) $A_{nl} = 0.90 A_s F_{ub} / F_{up}$

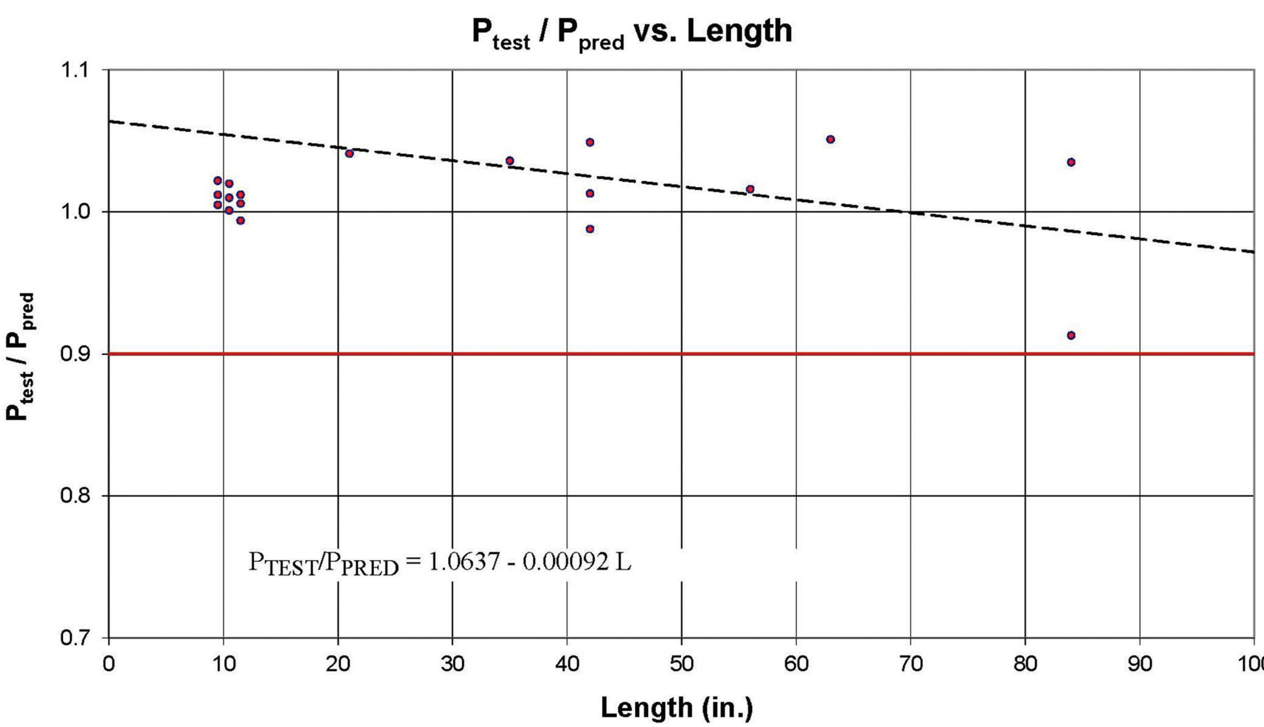

Fig. 2. Regression analysis of test data that satisfied both limit states.

Table 2. Limit State Comparison for Dispersed Bolt Group Connections											
Test No.	Bolt Type	Bolts in Line	D (in.)	L (in.)	$\frac{P_{Test}}{P_{Pred}}$	A_g (in.^{2})	A_{gl}[(1)] (in.^{2})	A_g/A_{gl}	A_n (in.2)	A_{nl}[(2)] (in.^{2})	A_n/A_{nl}
15	A490	7	⅞	21.0	1.041	9.56	7.2	1.33	7.66	6.6	1.16
6	A325	11	1⅛	35.0	1.036	18.9	14.0	1.35	15.5	13.3	1.17
16	A490	13	1⅛	42.0	1.049	28.6	22.1	1.29	23.7	20.0	1.19
9	A490	13	⅞	42.0	1.013	33.6	29.8	1.12	29.8	17.6	1.68
10	A325	13	⅞	42.0	0.988	29.8	25.7	1.16	26.1	14.8	1.76
17	A490	17	⅞	56.0	1.016	20.4	17.5	1.17	18.5	15.9	1.16
51	A490	13	⅞	63.0	1.051	33.8	30.0	1.13	30.0	18.7	1.61
18	A490	25	⅞	84.0	0.913	28.4	24.6	1.15	24.6	24.1	1.03
19	A490	25	⅞	84.0	1.035	37.6	26.6	1.41	33.7	24.1	1.40
Mean				52.1	1.016			1.234			1.351
Standard deviation				21.6	0.043			0.110			0.269

(1) $A_{gl} = 0.90 A_s F_{ub} / F_{yp}$
(2) $A_{nl} = 0.90 A_s F_{ub} / F_{up}$

RELIABILITY

With the recommended shear strength design criteria established, it is now possible to evaluate the test results in terms of LRFD procedures. The reliability index (β) is determined from Fisher et al. (1978):

$$\beta = \frac{\ln\left(\frac{\overline{R}}{\overline{Q}}\right)}{\sqrt{V_R^2 + V_Q^2}} \quad (5)$$

And the corresponding resistance, ϕ, is:

$$\phi = \frac{R_m}{R_n} \mathrm{EXP}\left(-0.55\beta V_R^2\right) \quad (6)$$

where

ϕ = bolt shear resistance
$\overline{R}$ = mean resistance
$\overline{Q}$ = mean load effect
V_r = coefficient of variation for $\overline{R}$
V_Q = coefficient of variation for $\overline{Q}$
R_m = mean test value
R_n = proposed connection length design criteria (R_2)

In Equation 6, ϕ is dependent upon knowing β. Similarly, when the step-by-step procedures are followed to solve Equation 5, ϕ is required to solve for β. This dilemma is resolved by using the current AISC (2010) and RCSC (2014) specified resistance value, ϕ, of 0.75. The corresponding ϕ and β values for the nine tests at 10.5 in. and at three connection lengths of 38 in., 60 in. and 84 in. are given in Table 3. Two possible length reduction factors were chosen—initially, $R_2 = 0.90$ was considered; subsequently, the reduction factor was eliminated or R_2 was set equal to 1.0. The reliability, β, and resistance, ϕ, in Table 3 are based on a live to dead load ratio of 3. Both β and ϕ will slightly change as the live to dead load ratio changes.

The critical issues were the importance of connection strength and quasi-stiffness as the connections became longer. The relatively small change in β (Table 3) as the connection length increases reinforces the small change in the value of P_{TEST}/P_{PRED} given by the linear-regression analysis in Figure 2.

When the computed values shown in Table 3 are compared to the target β value of 4.0 and the resulting resistance, ϕ, compared to the specified value of 0.75, it can be concluded, for connections that satisfy Equations 2 and 3, that there is no need to reduce the bolt shear strength because of connection length. With the reliability values higher than the target value (4.0) and resulting resistance greater than the assumed starting value (0.75), it can be considered that the test results demonstrate ample strength to accommodate small amounts of second-order effects.

SUMMARY AND CONCLUSIONS

A review of the historic research test data was made to determine bolt shear strength in terms of LRFD principles. Of the 119 identified bolted connection tests only 18 tests—7 A325 and 11 A490—satisfied the modified limit state requirements of gross and net area. These 18 tests were used in the statistical analysis. Recent tests reported by Moore et

Table 3. Reliability β and Resistance ϕ for Alternative Design Criteria (R_2)(1)

Connection Length, in.	Rm_	Standard Deviation	$R_2 = 0.9$		$R_2 = 1.0$	
			β	ϕ	β	ϕ
10.5	1.009	0.009	4.72	0.89	4.22	0.82
38	1.029	0.043	4.72	0.89	4.23	0.82
60	1.009	0.043	4.62	0.87	4.14	0.81
84	0.986	0.043	4.51	0.86	4.02	0.79

(1) Based on a live to dead load ratio of 3.

al. (2010) indicated that the reliability index (β of the shear strength of individual bolts was similar to that of plates and shapes reported in earlier literature. Based on other anecdotal information, there does not appear to be any justification to change the current AISC/RCSC resistance (ϕ) unless all second-order effects are considered and addressed.

The commentary to the AISC *Specification* (AISC, 2010) indicates an implied reliability, β, of approximately 4.0 for connections. In comparison, manufactured main members typically have β of approximately 3.0 or slightly lower. Because the bolt itself is a manufactured product, there is some leeway as to what β is acceptable. As a practical matter, it is prudent to retain a computed reliability relatively close to or greater than the stated goal of 4.0, as shown in Table 3. This eliminates the need for detailed second-order analysis for routinely used connections. To accomplish this, the current resistance, ϕ, of 0.75 was used in the computations, although the resulting computations (Table 3) and research by Moore et al. (2010) indicate the resistance could be increased.

An unexpected result of the study was the realization that, under circumstances of sufficient or slightly increased code required connection strength, as manifested by the net area (A_n), and in conjunction with connection quasi-stiffness, as manifested by the connection gross area (A_g) in comparison to the total bolt shear area (A_s), there would be no need for a connection strength reduction R_2 less than 0.90 with increasing length. The R_2 factor could possibly even equal 1.0. This condition exists when the inequalities expressed in Equations 2 and 3 are satisfied. Equation 2 is not exactly a stiffness criterion, but it indicates that the connection plates remain essentially elastic as the bolt ultimate shear strength is reached.

All of the test data represent uniaxial loaded connections with no second-order effects. In reality, many connections actually result in small amounts of unintended and unaccounted for second-order effects. Although not explicitly stated, this phenomenon is partially addressed by the specifications employing a slightly reduced resistance , ϕ, of 0.75 as compared to the value obtained from single-bolt tests as reported by Moore et al. (2010).

As a result, it is probable that the current reduction factor of 0.90 for connection lengths less than or equal to 38 in. is slightly conservative, and the step function change to a reduction factor of 0.75 for connections greater than 38 in. is excessively conservative. Removing the connection length reduction factor, R_2 = 1.0, would maintain a reliability, β, equal to or greater than 4.0 for all connection lengths. Bolted connections with obvious second-order effects would have to be properly addressed following LRFD principles.

The statistical study was based on ASTM A325 and A490 bolts; however, limited studies indicate that similar results were obtained for rivets with no inconsistencies found. The connection plate material varied from relatively low-strength to high-strength steel. This would indicate that the proposed solution is applicable for other connectors and material, provided the specification limit states for gross area, A_g, and net area, A_n, are satisfied as well as Equations 2 and 3.

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