THEORETICAL MODEL FOR PREDICTING SHEAR STRENGTH OF LOW RISE REINFORCED CONCRETE STRUCTURAL (SQUAT) WALLS

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*Abstract***— Shear strength of reinforced concrete walls is an important in design of low rise building subjected to lateral loads. In this paper, a simplified approach, using a softened strut and tie model for determining the shear strength of reinforced concrete structural (squat) walls with a height to width ratio less than or equal to 2 has been considered The proposed simplified strut and tie model satisfies equilibrium, compatibility and constitutive laws of cracked reinforced concrete elements. The shear capacities of 62 structural walls were calculated and compared with the available experimental results and ACI Code (ACI 318-08) and Hwang et al (2001)General and Simplified models and a good reasonable agreements was found. Based on the available experimental data in this paper, the proposed model was applied to study the effect of boundary elements, cyclic loading and vertical loading conditions on the wall shear strength.**

Keywords **: concrete, reinforced concrete, structural walls, strut & tie model**

INTRODUCTION

The analysis, design and behavior of reinforced concrete structural walls have received considerable attention because structural walls are commonly used elements in a wide range of different type of structures from tall buildings to offshore gravity structures. Structural walls have demonstrated extremely good performance in seismic regions. An adequate design of a structural wall has not achieved the same level of confidence presently available for seismic beams and columns.

Reinforced concrete structural walls (squat walls) with height to width ratio less than 2 have wide application in low rise buildings subjected to lateral load. Current ACI 318-08 method for squat walls is based on the empirical expression derived for beams using test results, therefore design provisions are still controversial despite the great deal of research that has been conducted through the years.

Since the original truss model for the beams with shear reinforcement was proposed by Ritter (1899) and Mörch (1909), their model has been improved by many researchers. To predict the shear strength of structural squat walls, a simplified theory based on softened strut and tie model was developed. The proposed theory is a simplified version of the softened strut and tie model that specifically predicts the shear strength of squat walls associated with diagonal compression failure. The proposed model is then verified against available experimental data.

RESEARCH SIGNIFICANCE

A simple expression of shear strength prediction is presented. The proposed softened diagonal strut model can predict the shear strength of reinforced concrete structural walls.

SOFTENED DIAGONAL STRUT MODEL

Figure 1 shows a typical reinforced concrete squat wall loaded horizontally at the top and fixed at the bottom. Based on figure 1, without vertical force, N, acting on the wall, the following relationship between vertical and horizontal shear force was assumed (Hwang et al., 2001)

$$\frac{V_{wv}}{V_{wh}} \approx \frac{H}{l} \tag{1}$$

Where V_{wv} and V_{wh} = vertical and horizontal shear force, respectively;

H = distance from point of application of V_{wh} to the base;

l = internal lever arm of couple at the base of the wall.

In order to account the vertical force, an enlarged cross sectional area of diagonal compression strut was considered in lieu of additional amount of vertical reinforcement in the walls (Mau and Hsu 1987) in this paper.

Macro Model:

Once the first cracking patterns develop in the wall, the vertical and horizontal reinforcement will be subjected to tension and concrete will act as compressive strut, thus forming a strut and tie action. There would be a three strut and tie load path, within squat wall and they are the diagonal strut, horizontal strut due to horizontal reinforcement and vertical strut due to vertical reinforcement. Figure 2 shows these mechanisms.

Based on the diagonal mechanism, diagonal compressions strut whose angle of inclination θ could be defined as:

$$\theta = \tan^{-1}(H/l) \qquad (2)$$

Also θ can be derived from a simple expression based on shear stresses less than that causing first yield of the reinforcement and is expressed as:

$$\theta = \tan^{-1}\left[\sqrt{\frac{\rho_v\ f_{vy}}{\rho_h\ f_{hy}}}\right] \qquad (3)$$

ρ_h, ρ_v = Volume of horizontal and vertical reinforcement resp.

f_{hy}, f_{vy} = steel stress in horizontal and vertical reinforcement resp.

It is assumed that the diagonal concrete strut coincide with the direction of principle compressive stress of the concrete. Therefore, the effective area of the diagonal strut A_{str} could be defined as

$$A_{str} = a_s \text{ x } t_w \qquad (4)$$

Where a_s = depth of diagonal strut

t_w = width of the diagonal strut and is equal to the web thickness of the wall

For simplicity, a_s the depth of diagonal strut assumed equals to the depth of flexural compressive zone of an elastic column load considered and is expressed as (Paulay and Priestley 1992)

$$a_s = \left[0.25 \quad + \quad 0.85\,(N/A_w\ f'_c\,)\right]\ l_w \qquad (5)$$

Where

A_w = net area of concrete section bounded by the web thickness;

l_w = length of section in the direction of shear force;

f'_c = compressive strength of concrete.

Based on Figure 2 the horizontal mechanism includes one horizontal tie and two flats struts. The horizontal tie is made-up of the horizontal shear reinforcement and is located within the centre half of the wall which will be fully effective. The vertical mechanism includes one vertical tie and two steep struts The vertical ties includes only vertical reinforcement within the wall web and the force of the walls and excludes the vertical reinforcement in the boundary elements. For a wall without boundary elements, $0.8l_w$ (ACI 318- 08) was considered effective to constitute the vertical tie. For diagonal compression failure, it is assumed that the shear strength of squat wall will reach its complete compressive stress capacity which includes the total concrete compressive bearing force from the diagonal, flat and steep strut as shown in Figure 2.

EQUILIBRIUM CONDITION

Since the diagonal compression is mainly transformed in *d*-direction, the maximum compression stress σ_{dmax} action on the nodal zone is assumed to govern the failure. Also since σ_d was calculated by using the softened stress – strain curve of the cracked concrete (Zhang Hsu 1998) σ_d equals to σ_{dmax} is assumed in this paper. Based on σ_{dmax} (Zhang and Hsu 1998) value, and Figure 3 the horizontal shear strength V_{wh} of reinforced squat wall can be calculated as :

$$V_{wh} = -D\cos\theta + F_v\cot\theta + F_h \qquad (6)$$

Where D = compression force in diagonal strut (positive for tension)

F_h, F_v = forces in the horizontal and vertical reinforcement, respectively.

It was assumed three load paths, i.e. concrete compression force in the diagonal strut and the tension forces in the horizontal and vertical reinforcements. These forces were proportionally resisted by wall shear and the distribution of shear forces were proportional to their relative stiffness i.e. stiffness ratios between vertical reinforcement and concrete diagonal strut and horizontal reinforcement. It was further assumed that the stiffness ratio between horizontal reinforcement and concrete diagonal strut to transfer the horizontal shear transferred by horizontal reinforcement in the absence of vertical reinforcement and similarly the stiffness ratio between vertical reinforcement and concrete diagonal strut to transfer the vertical shear in the absence of the horizontal reinforcement.

CONSTITUTIVE LAWS

Softened concrete in Compression

Vecchio and Collins (1993) and Belrarbi and Hsu (1994) models are two most accurate softened stress – strain relationship to predict the shear strength of squat walls, softening model of Zhang and Hsu (1998) is chosen due to its simplicity in mathematical formulation in this paper. The ascending branch of softened stress- strain curve of the cracked concrete is represented by Zhang and Hsu (1998).

$$\sigma_d = -\xi f'_c\left[2\left(\frac{-\varepsilon_d}{\xi\varepsilon_o}\right) - \left(\frac{-\varepsilon_d}{\xi\varepsilon_o}\right)^2\right] for\left(\frac{-\varepsilon_d}{\xi\varepsilon_o}\right) \le 1 \qquad (7)$$

where ξ = softening coefficient

$$= 0.6 + \frac{10}{f'_c} \le 0.85$$

ε_d = average principal strain in *d* direction

$$\varepsilon_0 = 0.002 + \frac{(f'_c - 20)}{80}$$

$E_c = 3320\sqrt{f'_c} + 6900$ (Carrasquillo et al , 1981)

Reinforcing steel:

Based on the elastically perfectly plastic model, the stress and strain relationship of horizontal and vertical shear reinforcement are

$$f_s = E_s \, . \varepsilon_s \quad for \quad \varepsilon_s < \varepsilon_y \tag{8a}$$

$$f_s = E_s \, . \varepsilon_y \;\; for \quad \varepsilon_s \; > \varepsilon_y \tag{8b}$$

Where ,

Es = Modulus of Elasticity of Steel (200 kN/mm^2)

f_s = stress in steel (variable f_s becomes f_{yh} and f_{yv} for horizontal and vertical reinforcement, resp.)

ε_s = Strain in steel (variable ε_s becomes ε_{sh} and ε_{sv} for horizontal and vertical shear reinforcement, resp.)

The relationship between forces and strains of tension ties becomes,

$$F_h = A_{th} \, f_{yh} \;\; \leq F_{yh} \tag{9a}$$

$$F_v = A_{tv} \, f_{yv} \;\; \leq F_{yv} \tag{9b}$$

Where A_{th} and A_{tv} = area of horizontal and vertical reinforcement respectively,

F_{yh} and F_{yv} = yielding force due to horizontal and vertical reinforcement respectively.

Strain Compatibility Conditions:

The strain compatibility condition used in this paper is as follows:

$$\varepsilon_r \;\; + \;\; \varepsilon_d \;\; = \varepsilon_{sh} \;\, + \varepsilon_{sv} \tag{10}$$

ε_r = average principal strain in r direction.

Based on review of the available literature by Solanki (2004), the ratio of $\dfrac{\varepsilon_r}{\varepsilon_d}$ ranges from 7 to 12 .To avoid iterating for full response, the ultimate strength is assumed to be reached at $\dfrac{\varepsilon_r}{\varepsilon_d} \approx 10$. In case of high strength steel, the ε_{sh} and ε_{sv} steel strain values were limited to 0.2 (i.e. concrete crushed prior to yielding of steel)

PROPOSED DESIGN/SOLUTION PROCEDURE

The following procedure is used for the calculation of the wall.

1. Equation 2 is employed to calculate the angle θ.
2. Equation 4 is used to calculate effective area of the diagonal strut.
3. Equation 5 is used to estimate the depth of compression strut.
4. Equation 10 is employed to calculate ε_d assuming $\dfrac{\varepsilon_r}{\varepsilon_d} = 10$
5. Equation 7 is used to employed to calculate σ_d
6. Based on the horizontal and vertical reinforcement and using equation 8(a) or 8(b), the forces in the horizontal and vertical reinforcement is calculated.
7. Once the value of σ_d ,F_h, and F_v is calculated eq. 6 is employed to calculate V_{wh}(cal).
8. After calculating V_{wh}(cal) , it is compared with V_{wh}(test) given in col. 10 table 1.
9. Column 14 of Table 1 is the ratio of V_{wh}(test) values / V_{wh}(cal) values.

EXPERIMENTAL VERIFICATION

A total of 62 results of tests specimen from the available literature are used to verify the proposed model. The test specimens considered in this paper have three unique features:

(1) All walls were reported to have failed in web shear mode.

(2) All walls are one storey isolated wall.

(3) The horizontal and vertical reinforcement were equal and reinforcement was uniformly distributed throughout the web.

The shear strength of tests walls was calculated using the proposed model (Column14 of Table 1) and was compared with Hwang et al. (2001) General and Simple models (Column 11 and 12 of Table 1) and the ACI (318– 08) method (Column 13 of Table 1). A key feature of the proposed model is that no iteration procedure is required and a good correlation was found between experimental (test) shear force and calculated shear force.

CONCLUSIONS

This paper presents a theoretical model to predict the shear strength of reinforced concrete squat walls. The theory results in a simple expression that the shear failure stress is derived from the concept of diagonal strut of cracked reinforced concrete (i.e. the softened stress and strain curve of cracked reinforced concrete). Based on the available test results and their comparison with the proposed model and Hwang et al. models (2001) and ACI (318-08) methods, the following conclusions can be made:

1. The proposed model is simple expression and is capable of predicting the shear strength of reinforced concrete squat walls for diagonal compression failure.
2. The proposed model predicts the failure shear consistent with the General and Simple models of Hwang et al. (2001) and ACI 318-08 building code.
3. The effect of vertical stress in the wall can be modeled by enlarging the cross sectional area of diagonal compression strut (Equation 5)

Additional research is needed for the walls with different boundary elements including wall openings, uneven horizontal and vertical reinforcement ratios, etc.

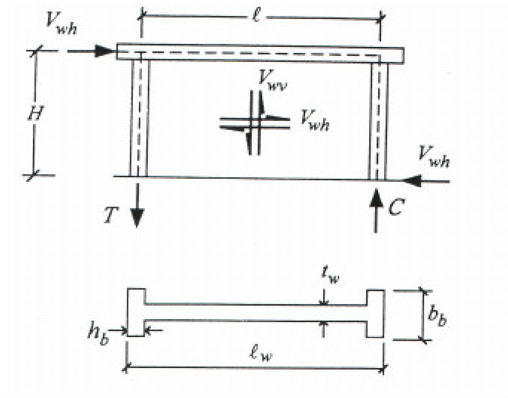

FIG. 1. External Actions and Internal Shears for Squat Wall

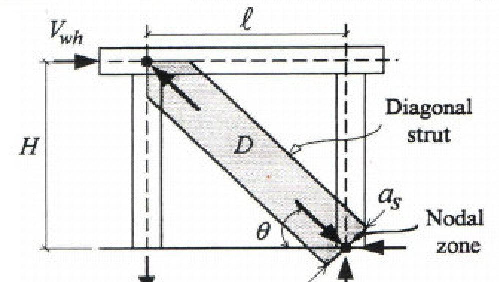

Figure 2: (a)

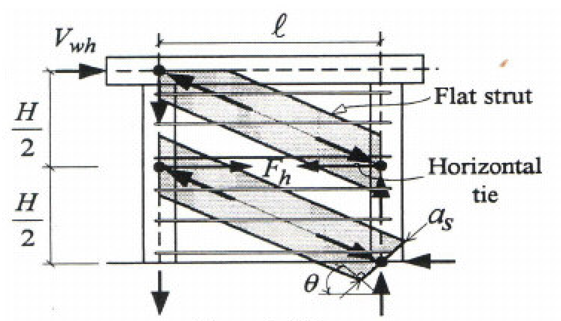

Figure 2: (b)

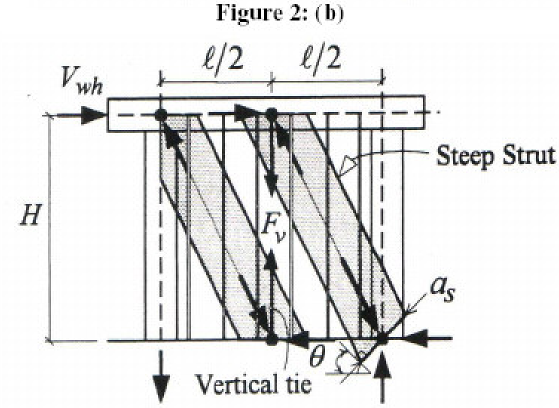

Figure 2: (c)

Figure 2: Wall Shear Mechanism : (a) Diagonal, (b) Horizontal (c) Vertical (Modified Hwang et al 2001)

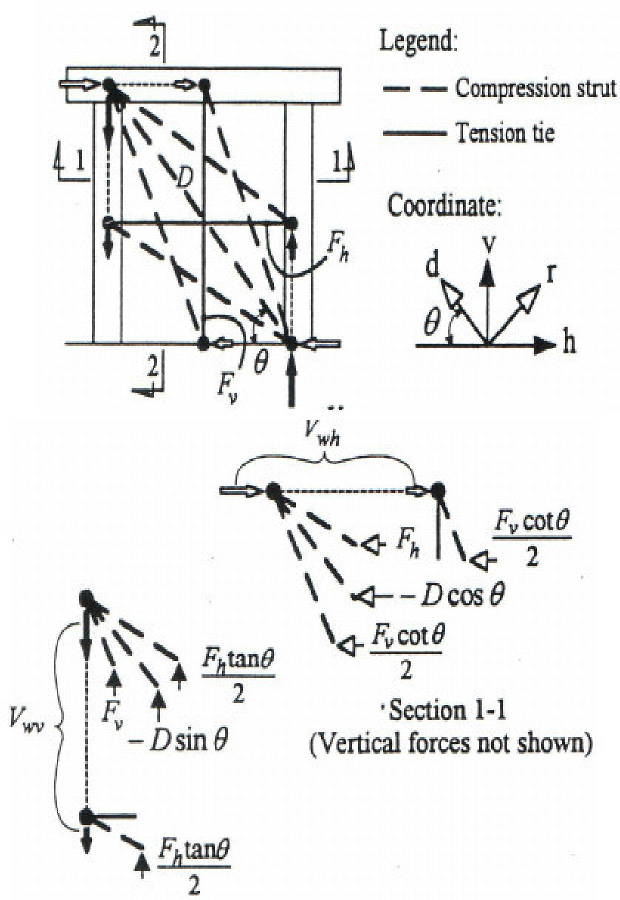

FIG. 3. Strut-and -Tie Model for Squat Wall
(adopted from Hwang et al. 2001)

References

[1] American Concrete Institute (ACI) 2008," Building Code requirements for Structural Concrete," ACI 318-08 and commentary (ACI 318R-08), Framing Hill, Mich.

[2] Barda,F.,Hanson,J.M. and Corley, W. G. (1977)," Shear strength of low rise walls with boundary elements, "Reinforced concrete structures in seismic zones, SP-53, American Concrete Institute, Framing Hill, Mich.

[3] Belarbi,A., and Hsu, T. T. C. (1990) "Constitutive Laws of concrete in tension and reinforcing bars stiffened by concrete "ACI Structural Journal , vol 92, No.5,pp 562-573.

[4] Benjamin,J.R. and Williams,H.A. (1957)"The behavior of one-storey reinforced concrete shear walls",J.Struct Di., ASCE, 83(3),I-49.

[5] Cardenas,A.E., Ruddell, H. G., and Corley, W. G.(1980)." Strength of low rise structural walls."Reinforced concrete structures subjected to wind and earthquake forces, SP -63, American Concrete Institute, Framing Hill, Mich.

[6] Carrasquillo,R.L., Nilson, A.H.,and Slate, F.O.(1981)."Properties of high strength concrete subjected to short term loads"ACI journal 78(3),171-178.

[7] CEB-FIP (1990),"Model Code for concrete Structures: CEB – FIP International Recommendations, 3rd Ed. Comité Euro-International due Béton, London England.

[8] Hirosawa,M.(1975)."Past experimental results on reinforced concrete shear walls and analysis on them"Kenchiku Kenkyu Shiro, No. 6, Build. Res. Inst., Ministry of Construction , Tokyo, 277pp (in Japanese).

[9] Hwang, S.J., Fang,W.H., Lee,J.H., and Yu, H.W. (2001) "Analytical model for predicting shear strength of squat walls"J.Struct. Engg. ASCE, 127(1), 43-50.

[10] Mau, S.T., and Hsu, T.T.C. (1987)," Shear behavior of reinforced concrete framed walls panels with vertical loads,"ACI Struct. J., 84(3), 228-234.

[11] Moesch, E.,(1909), "Concrete steel construction," McGraw Hill Inc., New York, NY (English Translation by: E.P. Goodrich)

[12] Paulay,T.,and Priestley, M. J. N. (1992). "Seismic design of reinforced concrete and masonry building.",Wiley New York.

[13] Ritter, W.,(1899), " Bie bauweise hennebique " Schweizerische Bauzeitung, Vol. 33, No. 7, pp 59-61.

[14] Vechhio, F. J., and Collins M.P. (1993), "Compression response of cracked reinforced concrete," Journal of the Structural Engineering, ASCE, Vol. 119, No. 12, pp 3590-3610.

[15] Yamada, M., Kawamura, H. and Katagihara, K.,(1974), " Reinforced concrete shear walls without openings: Test and analysis," Shear in reinforced concrete, SP -42, American Concrete Institute, Farmington, Mich.

[16] Zang, L,X.B., and Hsu, T.T.C.,(1998), "Behavior and analysis of 100 MPa concrete membrane elements," Journal of the Structural Engineering, ASCE, Vol. 124, No. 1, pp 24-34.

[17] Solanki,H.(2004), " Discussion on strength prediction for discontinuity regions by softened strut and tie model," by Shyh Jiann Hwang etal, Journal of Structural Engineering, ASCE, Vol. 130, No. 3, pp 529.

Table -1. Experimental Verification

Specimen	H x lw x tw (cm)	b_b x h_b (cm)	f'_c (MPa)	ρ_v (percent)	f_{yv} (MPa)	ρ_h (percent)	f_{yh} (MPa)	N/Aw f'_c	$V_{wh,test}$ (kN)	$V_{wh,test}/V_{wh,cal}$ Hwang et al.(2001) General	Hwang et al.(2001) Simple	ACI 1999	Proposed Method
1	2	3	4	5	6	7	8	9	10	11	12	13	14
(a) Benjamin and Williams (1975)													
4BII-1	56 x 61 x 5	13 x 10	20.1	0.5	359	0.5	359	0	89	1.45	1.28	1	0.89
3A2-3	56 x 91 x 5	13 x 10	21.5	0.5	359	0.5	359	0	155	1.32	1.06	1.15	1.1
4BII-3	56 x122 x 5	13 x 10	19.5	0.5	359	0.5	359	0	201	1.17	0.95	1.14	1.13
4BII-4	56 x 178 x 5	13 x 10	26.4	0.5	359	0.5	359	0	294	0.82	0.63	1.07	1.12
(b) Hirosawa (1965) as reported by Hirosawa (1975)													
9 - 40 - WI - 1	70 x 60 x 3	10 x 10	25.7	0.23	293	0.21	293	0.271	86	1.48	1.21	2.54	1.95
c) Sugano (1973) as reported by Hirosawa (1975)													
140 - 1	180 x 396 x 12	36 x 36	20.6	0.66	572	0.66	572	0.12	2,354	1.32	0.9	1.31	0.91
141 - 2	180 x 396 x 12	36 x 36	20.8	0.66	572	0.66	572	0.228	2,942	1.43	0.96	1.64	1.08
142 - 3	180 x 396 x 12	36 x 36	21.3	0.66	572	0.66	572	0.155	3,138	1.62	1.12	1.72	1.19
143 - 4	180 x 396 x 12	36 x 36	19.6	0.33	572	0.33	572	0.097	1,814	1.31	0.91	1.27	0.99
144 - 5	180 x 396 x 12	36 x 36	20.8	0.33	572	0.33	572	0.097	1,912	1.32	0.92	1.32	1.03
145 - 6	180 x 396 x 12	36 x 36	20.5	0.69	284	0.66	284	0.11	2,138	1.31	0.92	1.49	1.17
146 - 7	180 x 396 x 12	36 x 36	19.6	0.69	284	0.66	284	0.106	1,981	1.27	0.89	1.39	1.09
148 - 8	180 x 396 x 12	36 x 36	20.9	0.77	397	0.74	397	0.116	2,305	1.25	0.9	1.28	0.9
(d) Yoshizaki (1973) as reported by Hirosawa(1975)													
165 - I - 56 - 2	86 x 80 x 6	-	23.5	0.22	433	0.23	433	0	102	1.12	1.12	1.12	0.8
166 - I - 56 - 8	86 x 80 x 6	-	23.5	0.73	433	0.82	433	0	147	1.1	1.16	1.08	1
167 - I - 88 - 4	86 x 80 x 6	-	23.5	0.44	433	0.41	433	0	135	1.15	1.16	0.94	0.92
168 - I - 88 - 8	86 x 80 x 6	-	23.5	0.73	433	0.82	433	0	159	1.06	1.24	0.88	1.09
169 - I - 88 - 12	86 x 80 x 6	-	23.5	1.17	433	1.17	433	0	175	1.08	1.32	0.9	1.07
170 - 2/3 - 36 - 2	86 x 120 x 6	-	24.5	0.24	433	0.23	433	0	160	1.04	1.04	1.04	0.86
171 - 2/3 - 36 - 8	86 x 120 x 6	-	24.5	0.78	433	0.82	433	0	235	1.05	1.05	0.94	0.99
172 - 2/3 - 52 - 4	86 x 120 x 6	-	24.5	0.44	433	0.41	433	0	220	1.03	1.06	1.01	0.93
173 - 2/3 - 52 - 8	86 x 120 x 6	-	24.5	0.78	433	0.82	433	0	260	1.05	1.13	0.88	1.1
174 - 2/3 - 52 - 12	86 x 120 x 6	-	24.5	1.17	433	1.17	433	0	275	1.04	1.16	0.93	1.16
175 - 1/2 - 27 - 2	86 x 120 x 6	-	25.5	0.22	433	0.23	433	0	199	0.91	0.91	0.91	1.07
176 - 1/2 - 27 - 8	86 x 120 x 6	-	25.5	0.8	433	0.82	433	0	322	0.92	0.88	0.81	1.11
177 - 1/2 - 42 - 4	86 x 120 x 6	-	25.5	0.36	433	0.41	433	0	319	0.99	0.95	1.09	1.13
178 - 1/2 - 42 - 8	86 x 120 x 6	-	25.5	0.8	433	0.82	433	0	382	0.99	1.01	0.95	1.32
179 - 1/2 - 42 - 12	86 x 120 x 6	-	25.5	1.17	433	1.17	433	0	422	1.09	1.1	1.05	1.07

(e) Yamada et al. (1974)

gw = 0.003	60 x 133 x 4	13 x 13	35.6	0.31	286	0.31	286	0.186	373	1.02	0.83	2.95	2.4
gw = 0.006	60 x 133 x 4	13 x 13	30.4	0.63	286	0.63	286	0.201	370	1.08	0.88	2.19	1.6
gw = 0.012	60 x 133 x 4	13 x 13	31.5	1.26	286	1.26	286	0.218	438	1.07	0.89	1.78	1.13
t = 30; gw = 0.008	60 x 133 x 3	13 x 13	32.8	0.84	286	0.84	286	0.221	276	0.98	0.76	1.81	1.24
t = 20; gw = 0.006	60 x 133 x 2	13 x 13	30.1	0.63	286	0.63	286	0.305	211	1.32	0.85	2.5	1.74
t = 20; gw = 0.012	60 x 133 x 2	13 x 13	33.7	1.26	286	1.26	286	0.293	213	1.03	0.73	1.67	1.05

(f)Barda et al. (1977)

B1 - 1	95 x 191 x 10	61 x 10	29	0.5	543	0.5	496	0	1,276	1.73	1.38	1.75	1.19
B2 - 1	95 x 191 x 10	61 x 10	16.3	0.5	552	0.5	499	0	965	1.12	1.69	1.51	0.99
B3 - 2	95 x 191 x 10	61 x 10	27	0.5	545	0.5	513	0	1,112	1.25	1.28	1.51	1.03
B6 - 4	95 x 191 x 10	61 x 10	21.2	0.5	496	0.5	496	0	872	1.31	1.38	1.26	0.89
B7 - 5	48 x 191 x 10	61 x 10	25.7	0.5	531	0.5	501	0	1,140	0.9	0.93	1.58	1.23
B8 - 5	191 x 191 x 10	61 x 10	23.4	0.5	527	0.5	496	0	889	1.64	1.73	1.26	0.98

(g) Cardenas et al. (1980)

SW - 7	206 x 191 x 8	-	43	0.94	448	0.27	414	0	519	0.93	0.92	1.3	0.96
SW - 8	206 x 191 x 8	-	42.5	2.93	448	0.27	465	0	569	0.95	1	1.36	0.99

(h) Mo (1993)

HN4 - 1	65 x 86 x 7	17 x 8	32.2	0.72	302	0.81	302	0.007	205	1.05	0.99	0.99	1
HN4 - 2	65 x 86 x 7	17 x 8	32.2	0.72	302	0.81	302	0.007	247	1.27	1.2	1.2	1.2
HN4 - 3	65 x 86 x 7	17 x 8	32.1	0.72	302	0.81	302	0.007	202	1.04	0.98	0.98	0.98
HN6 - 1	65 x 86 x 7	17 x 8	29.5	0.72	443	0.81	302	0.007	255	1.42	1.08	1.11	1.2
HN6 - 2	65 x 86 x 7	17 x 8	29.5	0.72	443	0.81	302	0.007	204	1.13	0.86	0.89	0.96
HN6 - 3	65 x 86 x 7	17 x 8	31	0.72	443	0.81	302	0.007	205	1.1	0.83	0.89	0.95
HM4 - 1	65 x 86 x 7	17 x 8	37.5	0.72	302	0.81	302	0.006	223	1.07	1.07	1.07	1.05
HM4 - 2	65 x 86 x 7	17 x 8	37.5	0.72	302	0.81	302	0.006	231	1.11	1.11	1.11	1.09
HM4 - 3	65 x 86 x 7	17 x 8	39.9	0.58	302	0.81	302	0.005	250	1.2	1.2	1.2	1.1
LN4 - 1	65 x 86 x 7	17 x 8	18	0.58	302	0.81	302	0.012	193	1.57	1.27	1.03	1.03
LN4 - 2	65 x 86 x 7	17 x 8	18	0.58	302	0.81	302	0.012	217	1.76	1.43	1.15	1.16
LN4 - 3	65 x 86 x 7	17 x 8	29.7	0.58	302	0.81	302	0.007	203	1.14	1.06	1.06	1
LN6 - 1	65 x 86 x 7	17 x 8	30.7	0.58	443	0.81	302	0.007	246	1.38	1.03	1.06	1.14
LN6 - 2	65 x 86 x 7	17 x 8	30.2	0.58	443	0.81	302	0.007	200	1.13	0.85	0.87	0.93
LN6 - 3	65 x 86 x 7	17 x 8	30.2	0.58	443	0.81	302	0.007	210	1.19	0.89	0.91	0.98
LM6 - 1	65 x 86 x 7	17 x 8	39.3	0.58	443	0.81	302	0.005	219	1.03	0.8	0.9	0.92
LM6 - 2	65 x 86 x 7	17 x 8	37	0.58	443	0.81	302	0.006	205	1.01	0.75	0.86	0.95
LM6 - 3	65 x 86 x 7	17 x 8	34.5	0.58	443	0.81	302	0.006	210	1.08	0.8	0.89	0.95
LM4 - 2	65 x 86 x 7	17 x 8	66	0.58	302	0.81	302	0.003	252	1.28	1.28	1.28	1.05
LM4 - 3	65 x 86 x 7	17 x 8	66	0.58	302	0.81	302	0.003	227	1.16	1.16	1.16	0.95

62	Average	1.18	1.05	1.26	1.1
	COV	0.17	0.21	0.34	0.25

NOTE: 1 MPa = 0.1450 ksi; 1 kN = 0.2248 kips