Deflection Design of Cold-formed RHS Steel Beams

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SUMMARY

The moment-deflection results of cold-formed RHS (rectangular hollow section) steel beams are examined. A simple design approach is proposed to account for the effect that material non-linearity has on deflection. A general expression is also derived to predict the deflection of cold-formed RHS steel beams.

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1. INTRODUCTION

In Australia, cold-formed steel structures are generally designed to the Australian Cold-Formed Steel Structures Standard AS 1538-1988 (SAA 1988), which is similar to the American Iron and Steel Institute Specification for the Design of Cold-Formed Steel Structural Members (AISI 1986). However, cold-formed RHS (rectangular hollow section) members manufactured in accordance with the Australian Structural Steel Hollow Sections Standard AS 1163-1991 (SAA 1991) are included within the scope of the Australian Steel Structures Standard AS 4100-1990 (SAA 1990). The strength design rules in AS 4100 for hollow sections are the result of Australian research performed over the past decade (Hancock 1994, Hancock and Zhao 1992, Hancock, Sully and Zhao 1994, Key 1988, Key and Hancock 1985, 1986, 1993a, 1993b, Key, Hasan and Hancock 1988, Hasan 1987, Hasan and Hancock 1989, Sully and Hancock 1995, Zhao 1992, Zhao and Hancock 1991a, 1991b, 1991c, 1992a, 1992b, 1993, 1994, 1995a, 1995b, 1995c, 1995d, Zhao, Hancock and Trahair 1995).

The design rules in AS 4100-1990 for calculating the deflection of RHS beams are based on linear-elastic theory, whereby a constant value is assumed for the modulus of elasticity E of 200 GPa (29000 ksi). This is also the case in comparable overseas design Standards, e.g. Canadian Limit States Standard for Steel Structures CAN/CSA-S16.1-94 (CSA 1994). The rules in AS 4100 are documented in the Design Capacity Tables for Structural Steel Hollow Sections published by the Australian Institute of Steel Construction (AISC 1992). However, a value of E=200 GPa (29000 ksi) was derived for hot-rolled I-sections for which the steel exhibits essentially elastic-plastic behaviour followed by strain-hardening, and may be inappropriate for cold-formed RHS beams. The steel in this latter type of section exhibits a non-linear (rounded) stress-strain relationship up to maximum stress. This is due to the existence of large through-thickness residual stresses which arise during the cold-forming process (Clarke 1992). Therefore, the steel in a cold-formed RHS beam yields gradually as load is applied to the member, and a non-linear (rounded) moment-deflection curve results. Tests show that using E=200 GPa (29000 ksi) may cause deflections to be significantly underestimated, which is the subject of this paper.

The results of tests on cold-formed RHS beams of stress grades 350 MPa (51 ksi) (Hasan and Hancock 1989) and 450 MPa (65 ksi) (Zhao and Hancock 1991a) are examined. A simple, empirical design method is proposed to account for the effect of material non-linearity which amounts to multiplying the deflection calculated using linear-elastic theory under service-loading by a constant correction factor (=1.2). A non-linear analysis is described which was conducted using estimates for the secant modulus E_s calculated from test data using the Ramberg-Osgood stress-strain formula. A finite element analysis was also performed to predict the deflection of the test beams, and good

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agreement was obtained between the two approaches. As a consequence, a slightly more accurate estimate is proposed for the correction factor to the deflection calculated using linear-elastic theory, which amongst other aspects takes account of variation in the live-to-dead load ratio Q/G.

2. EXPERIMENTAL OBSERVATIONS

2.1 General

The moment versus mid-span deflection behaviour of a simply-supported RHS beam is shown schematically in Fig. 1, where M_u is the ultimate moment capacity of the beam, and M^* and M_s are the design bending moments corresponding to the strength and serviceability limit states, respectively. The term Δ_{el} is the mid-span service-load deflection predicted using linear-elastic theory, while Δ_n and Δ_{exp} are the mid-span service-load deflections either predicted using non-linear analysis or determined experimentally, respectively.

The shape of the moment-deflection curve shown in Fig. 1 is typical of cold-formed RHS beams acting under predominantly flexural conditions, and is discussed further in Section 2.3. In contrast, hot-rolled I-section beams do not display non-linear behaviour until much higher values of the ratio M/M_u, where M is the applied bending moment, which is described as follows.

2.2 Hot-Rolled I-Section Beams

According to Kulak et al. (1995), the response of a hot-rolled or welded I-section beam, with a maximum longitudinal residual strain in the section equal to $0.3\varepsilon_y$ (WRC-ASCE 1971), is linear-elastic until the maximum moment reaches $0.70M_y$, where M_y is the first-yield moment and ε_y is the yield strain.

An experimental investigation conducted by Suzuki and Ono (1973) showed that the response of welded I-section beams was linear-elastic until the maximum moment reached $0.7M_p$, where M_p is the plastic moment capacity of the section calculated using measured yield strengths. Similarly, experimental investigations by Suzuki and Ono (1970) and Udagawa et al. (1973) showed that the response of hot-rolled I-section beams was linear-elastic until the maximum moment reached between $0.80M_p$ and $0.90M_p$.

For the situations described above, it can be demonstrated that the moment after which behaviour becomes non-linear is generally close to or larger than M_s. Therefore, linear-elastic theory (with E=200 GPa (29000 ksi)) is generally adequate for predicting the deflection of hot-rolled I-section beams.

2.3 Cold-Formed RHS Beams

Experimental investigations by Hasan and Hancock (1989) and Zhao and Hancock (1991a) have shown that the moment-deflection response of typical Australian cold-formed RHS beams becomes non-linear when the applied moment is as low as $0.2M_p$.

Therefore, linear-elastic theory assuming E=200 GPa (29000 ksi) underestimates the deflection of cold-formed RHS beams at typical service load levels.

The tests were reported in detail by Hasan and Hancock (1989) and Zhao and Hancock (1991a). The rectangular hollow sections were manufactured in accordance with AS 1163. A schematic view of the "bending test" set-up is shown in Fig. 2, noting that "a" is the shear span. The test details, specimen dimensions, average yield stress (i.e. 0.2% proof stress for a rounded stress-strain curve) and compactness (C for compact, N for non-compact, S for slender) are summarised in Tables 1 and 2 for 450 MPa and 350 MPa RHS beams, respectively. One test was performed for each section of grade 450 MPa, and the specimen numbers are designated BS1 to BS10. Two tests (on different specimens) were performed for each section of grade 350 MPa, and the specimen numbers are designated BS11a, BS11b to BS19a, BS19b. Therefore, all together twenty-eight bending tests were performed.

3. SIMPLE EMPIRICAL DESIGN METHOD

It follows from Fig. 1 that for a rounded moment-deflection curve, the ratio Δ_{exp}/Δ_{el} increases with the ratio M_s/M_u, while the value of M_s/M_u depends on the values of the live-to-dead load ratio Q/G and the strength and serviceability load factors as described in Section 5.

In order to derive a simple approach to determine the non-linear deflection Δ_n, the value of the ratio M_s/M_u is chosen as 0.60 for the following reasons.

(i) In permissible stress design codes AISI-1986 and AS1538, a typical safety factor of 1/0.60 is used for the design of beams.
(ii) In limit states design of steel beams to AS4100-1990 (SAA 1990), the capacity factor ϕ is 0.90 for bending, and the strength load factors normally equal 1.25 for dead load G and 1.50 for live load Q (SAA 1989). Therefore, the ratio of the working load (G+Q) to the design load for strength approaches $\phi/1.5=0.60$ (AISC 1992).

For each of the tests in Tables 1 and 2, the experimental and elastic deflections, Δ_{exp} and Δ_{el} respectively, were determined as follows:

(i) Δ_{exp} was measured at a mid-span service-load moment $M_s=0.6M_u$; and
(ii) Δ_{el} was calculated assuming measured section dimensions and E=200 GPa (29000 ksi) at a mid-span service-load moment $M_s=0.6M_u$.

where M_u is the ultimate moment capacity obtained from the test. This is a slightly more conservative approach than having used M_u equal to nominal moment capacity of each beam.

Values of the ratio Δ_{exp}/Δ_{el} are given in Table 1 for the tests on the 450 MPa RHS beams, which vary from 1.14 to 1.32 with a mean value of 1.23. Similarly, for the tests on the 350 MPa RHS beams the values vary from 1.11 to 1.31 with a mean value of 1.21. It can be concluded that the experimental deflection Δ_{exp} at a mid-span bending moment $M=0.6M_u$ is on average about 20 per cent larger than that predicted using linear-elastic theory. A correction factor of 1.20 is proposed as a simple empirical method to determine the deflection of cold-formed RHS beams, i.e.

$$\Delta_{RHS} = 1.20\ \Delta_{el} \tag{1}$$

4. NON-LINEAR ANALYSIS

4.1 General

An approximate method for determining the deflection of stainless steel beams has been developed by Rasmussen and Hancock (1993). It involves modelling the non-linear stress-strain curve of the material with an explicit expression (i.e. modified Ramberg-Osgood formula), and calculating the secant modulus E_s. Rasmussen and Hancock obtained good agreement using finite element analysis to verify the accuracy of the approximate method.

The modified Ramberg-Osgood formula is given by the following expression, which closely represents the non-linear stress-strain curve of RHS beam material:

$$\varepsilon = \frac{\sigma}{E_o} + 0.002\left(\frac{\sigma}{\sigma_{0.2}}\right)^n \tag{2}$$

where,

$$n = \frac{\ln(4)}{\ln(\sigma_{0.2}/\sigma_{0.05})} \tag{3}$$

The deflection of a beam can be expressed as:

$$\Delta = K_v \frac{PL^3}{E_s I} \tag{4}$$

in which P is the applied load, L is the beam span, I is the second moment of area and E_s is the average of the secant moduli (E_{st} and E_{sc}) calculated at the extreme fibers in tension and compression. It is assumed in this paper that the secant modulus derived from compression coupon tests (E_{sc}) is the same as that derived from tensile coupon tests (E_{st}). The expression for E_s is:

$$E_s = \frac{\sigma}{\varepsilon} = E_o\left\{1 + 0.002\frac{E_o}{\sigma_{0.2}}\left(\frac{\sigma}{\sigma_{0.2}}\right)^{n-1}\right\}^{-1} \tag{5}$$

where E_o is the initial modulus of elasticity (i.e. tangent modulus at ε=0).

The term K_v in Eq. 4 depends on the boundary and loading conditions. It is defined such that Eq. 4 reproduces the linear-elastic expression for the deflection when E_s is replaced by E_o (Rasmussen and Hancock 1993). It is assumed that the stress at the extreme fibres can be determined using:

$$\sigma = k_\sigma \frac{M}{Z} \tag{6}$$

where M is the bending moment, Z is the elastic section modulus and k_σ is a stress factor less than or equal to unity. Suitable values of k_σ are determined by trial and error using test results and/or results obtained from finite element analysis.

4.2 Determination of Parameter Values

The value of parameter n in Eq. 3 is determined using stress-strain curves derived from tensile coupon tests of cold-formed steel sections. For the specimens referred to in Tables 1 and 2, the average value of n was about 5.0.

After a lengthy trial-and-error process, a value of k_σ=0.80 satisfactorily predicted the deflection of the 450 MPa RHS beams in Table 1 at a load of 60 per cent of the maximum load. For the 350 MPa RHS beams in Table 2, the stress factor k_σ is assumed to equal $0.8\sqrt{(\sigma_y/450)}=0.71$. This assumption is similar to the approach adopted in AS 4100 to consider the effect of material yield stress, noting that the value of 0.71 is only slightly higher than a value of 0.67 suggested by Rasmussen and Hancock (1993) for a single-span stainless SHS (square hollow section) beam.

4.3 Determination of Deflection

From Eqs 4, 5 and 6, the non-linear deflection Δ_n can be expressed as:

$$\Delta_n = K\,\Delta_{el} \tag{7}$$

where,

$$K = 1 + 0.002\left(\frac{E_o}{\sigma_y}\right)k_\sigma^4 SF^2\left(\frac{M_x}{M^*}\right)^2 \tag{8}$$

in which σ_y is the yield stress (i.e. 0.2% proof stress for a rounded stress-strain curve), SF is the shape factor of the RHS section and M* is the plastic moment capacity M_p.

A typical comparison (specimen BS1) of the deflection predicted using Eq. 7 and that determined experimentally is shown in Fig. 3, where good agreement is obtained.

4.4 FE Analysis

The finite element program Strand6 (G+D Computing 1993) was used to simulate the behaviour of the cold-formed RHS beams referred to in Tables 1 and 2. Representative stress-strain curves of the test material were used in the simulation, noting that the through-thickness residual stresses in an RHS section are incorporated in the measured stress-strain curves (Clarke 1992).

The results of the finite element analysis (again for specimen BS1) are compared in Fig. 3 with those determined using non-linear analysis and the experiment result, where good agreement is obtained. The curve predicted using linear-elastic theory is also shown in Fig. 3 for comparison purposes.

5 LIMIT STATES DESIGN

5.1 Load Combinations

It can be seen from Fig. 1 and Eqs 7 and 8 that the deflection ratio Δ_n/Δ_{el} depends on the moment ratio M_s/M^*, or equivalently on the load ratio P_s/P^*.

Design load P* is calculated as follows as a combination of dead load G and live load Q (SAA 1989, NRCC 1995):

$$P^* = 1.25G + 1.50Q \tag{9}$$

This is different to the load combination (1.20G + 1.60Q) specified in ANSI A58.1(ANSI 1982).

For normal office occupancy, the short- and long-term serviceability loads are calculated as follows (SAA 1989, Pham and Dayeh 1986):

$$P_s = G + 0.7Q \tag{10}$$

$$P_s = G + 0.4Q \tag{11}$$

This is different to the load combination (G+Q) specified in CAN/CSA-S16.1-94 (Kulak et al. 1995) and the AISI-1991 Specification (AISI 1991, Galambos and Yu 1984).

In this paper, only the load combinations expressed by Eqs 9 and 10 will be considered. Similar results will be obtained if other load combinations are used.

5.2 P_s/P^* versus Q/G

From Eqs 9 and 10 it follows that:

$$\frac{M_s}{M^*} = \frac{P_s}{P^*} = \frac{G+0.7Q}{1.25G+1.50Q} = \frac{1+0.7Q/G}{1.25+1.50Q/G} \tag{12}$$

This relationship between P_s/P^* and Q/G is shown in Fig. 4. It can be seen that the ratio P_s/P^* increases rapidly as Q/G decreases, and its value varies from 0.80 when Q/G=0 to 0.467 for a large value of Q/G.

It follows from Fig. 4 that $P_s/P^*=0.54$ when Q/G=3. This value of Q/G was used to calibrate the AISC-LRFD Specification (AISC 1993, Galambos 1995). Similarly, for Q/G=5, $P_s/P^*=0.51$. This value of Q/G was used to calibrate the AISI-LRFD Specification (AISI 1990, AISI 1991), noting however that the strength limit (M_u in Fig. 1) in AISI-1991 is M_y rather than M_p.

5.3 Δ_n / Δ_{el} versus P_s/P^*

The correction factor K (=Δ_n / Δ_{el}) given by Eq. 8 is plotted against P_s/P^* in Fig. 5 with P_s/P^* varying between 0.467 and 0.80 as explained in Section 5.2. Curves are given for both the 450 MPa and 350 MPa RHS beams in Tables 1 and 2, for which average measured values of SF and σ_y are (1.19, 1.20) and (461, 374 MPa), respectively.

It can be seen from Fig. 5 that the curves for the two steel grades are very similar, and that K varies from approximately 1.10 to 1.30. The value of K=1.20 proposed in Section 3 is clearly a mid-range value.

5.4 $\Delta_n / \Delta_{elastic}$ versus Q/G

Substituting Eq. 12 into Eq. 8, it follows that:

$$K = \frac{\Delta_n}{\Delta_{el}} = 1 + 0.002(\frac{E_o}{\sigma_y}) k_o^4 SF^2 (\frac{1+0.7Q/G}{1.25+1.50Q/G})^2 \qquad (13)$$

The relationship between K (=Δ_n / Δ_{el}) given by Eq. 13 and Q/G is shown in Fig. 6.

Some typical values of the Q/G ratio found in floor and roof construction are presented in Table 3 (ADCM 1993). The corresponding values of P_s/P^* and Δ_n / Δ_{el} (=K) have been calculated and are included in Table 3. It can be observed that the value of the correction factor K varies from 1.15 to 1.26, again indicating that the value of 1.20 suggested in Section 3 is a good approximation for some typical practical situations.

6. CONCLUSIONS

1) Tests on cold-formed RHS steel beams have shown that linear-elastic theory assuming a modulus of elasticity E of 200 GPa (29000 ksi) underestimates their deflection under service loading by approximately 20 per cent.

2) During design, the effect of the non-linear stress-strain curve of the RHS steel can be taken simply into account by multiplying the deflection Δ_{el} calculated using linear-elastic theory by a correction factor K (=Δ_n / Δ_{el}) to give the non-linear deflection Δ_n.

3) As a simple rule, it has been shown that K equals 1.20. A slightly more accurate expression has been derived for K which amongst other aspects includes the live-to-dead load ratio Q/G in one of its terms. For some typical practical situations, it has been shown theoretically that K varies between 1.10 and 1.30, whereby the simple design approach of assuming K=1.20 is normally satisfactory.

7. COMMENTS

1) Only limited tests on cold-formed RHS steel beams were examined in this paper. In order to draw a more general conclusion on deflection design of cold-formed RHS steel beams, more tests are needed on RHS beams with different span lengths and different load cases.

2) The effect of the first loading event was examined in this paper, where a permanent set occurs once the load is removed. However, the serviceability deflections under live load may be less than those computed for the first loading event because upon reloading the member will follow the stiffer unloading path.

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Appendix - NOTATION

a	Shear span
B	Width
D	Depth
E_o	Initial modulus of elasticity
E_s	Secant modulus
G	Dead load
I	Second moment of area
K	Correction factor ($=\Delta_n/\Delta_{el}$)
K_v	Factor used in vertical deflection calculation
k_σ	Stress factor (≤ 1)
L	Span
P	Load
P_s	Design service load
P*	Design load corresponding to strength limit state
M	Bending moment
M_p	Plastic moment capacity

M_y	First-yield moment
M_s	Design moment corresponding to service load
M^*	Design moment corresponding to strength limit
M_u	Ultimate or nominal moment capacity
n	Parameter in modified Ramberg-Osgood formula
Q	Live load
SF	Shape factor
t	Wall thickness
Z	Elastic section modulus
Δ_{el}	Elastic deflection
Δ_{exp}	Experimental deflection
Δ_n	Non-linear deflection
Δ_{RHS}	Deflection of RHS beam
β	Degree of shear connection
ε	Strain
ε_y	Yield strain
σ	Stress
$\sigma_{0.2}$	0.2% proof stress
$\sigma_{0.05}$	Stress corresponding to 0.05% strain
σ_y	Yield stress
ϕ	Capacity reduction factor

Specimen No.	D x B x t (mm)	L (mm)	a (mm)	σ_y (MPa)	Compact -ness	Δ_{exp} (mm)	Δ_{el} (mm)	$\Delta_{exp} / \Delta_{el}$
BS1	100x100x3.8	1000	250	459	N	5.47	4.63	1.18
BS2	100x100x3.3	1000	250	435	N	5.20	4.54	1.15
BS3	100x100x2.8	1000	250	466	S	5.30	4.16	1.27
BS4	75x75x3.3	1000	250	462	C	6.77	5.55	1.22
BS5	75x75x2.8	1000	250	490	N	6.97	5.94	1.17
BS6	75x75x2.3	1000	250	469	S	6.73	4.80	1.40
BS7	65x65x2.3	1000	250	479	N	7.46	6.54	1.14
BS8	125x75x3.8	1000	250	448	C	5.20	4.22	1.23
BS9	125x75x3.3	1000	250	452	C	4.91	3.72	1.32
BS10	100x50x2.8	1000	250	451	C	6.00	5.00	1.20
mean	--	--	--	461	--	--	--	1.23
Cov	--	--	--	0.034	--	--	--	0.07

Table 1 Results for Grade 450 MPa RHS Beams

Specimen No.	D x B x t (mm)	L (mm)	a (mm)	σ_y (MPa)	Compact -ness	Δ_{exp} (mm)	Δ_{el} (mm)	$\Delta_{exp} / \Delta_{el}$
BS11a	254x254x4.9	2400	800	418	S	9.83	7.92	1.24
BS11b	254x254x4.9	2400	800	418	S	10.08	7.67	1.31
BS12a	203x203x9.5	1600	400	438	C	7.11	6.00	1.19
BS12b	203x203x9.5	1600	400	438	C	7.46	6.15	1.21
BS13a	203x152x6.3	1600	400	368	C	5.40	4.80	1.13
BS13b	203x152x6.3	1600	360	368	C	5.74	4.78	1.20
BS14a	127x127x4.9	1600	400	378	C	7.77	7.00	1.11
BS14b	127x127x4.9	1600	400	378	C	8.70	6.87	1.27
BS15a	102x102x4.0	1000	250	373	C	4.57	3.91	1.17
BS15b	102x102x4.0	1000	250	373	C	4.67	3.62	1.29
BS16a	102x76x3.6	1000	250	341	C	4.35	3.77	1.15
BS16b	102x76x3.6	1000	250	341	C	4.60	3.90	1.18
BS17a	89x89x3.6	1000	250	358	C	5.30	4.11	1.29
BS17b	89x89x3.6	1000	250	358	C	4.67	3.78	1.24
BS18A	76x76x3.2	1000	250	344	C	5.84	4.65	1.26
BS18b	76x76x3.2	1000	240	344	C	6.11	4.80	1.27
BS19a	76x76x2.6	1000	250	347	N	5.00	4.40	1.14
BS19b	76x76x2.6	1000	240	347	N	5.00	4.43	1.13
mean	--	--	--	374	--	--	--	1.21
Cov	--	--	--	0.087	--	--	--	0.05

Table 2 Results for Grade 350 MPa RHS Beams

Component	Material	L/D	P_s / P^*	Δ_n / Δ_{el}
Floor	Timber	3.0	0.54	1.15
Floor	Concrete	0.50	0.68	1.23
Roof	Sheet	0.625	0.66	1.21
Roof	Tile	0.278	0.72	1.26

Table 3 Typical Examples

Figure 1 Moment versus Mid-span Deflection Behaviour

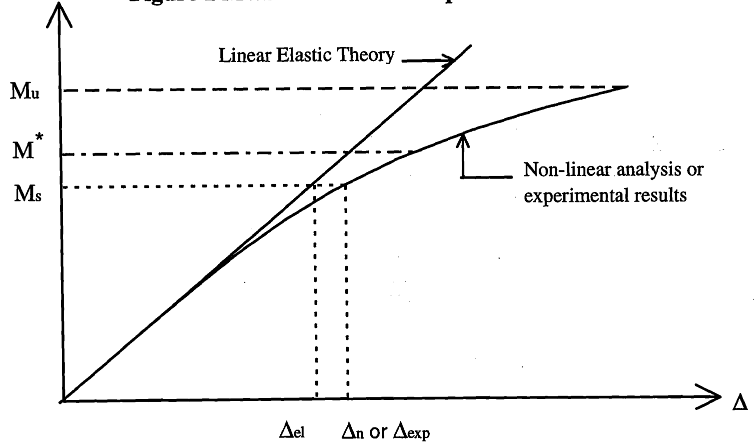

Figure 2 Scematic View of Test Setup

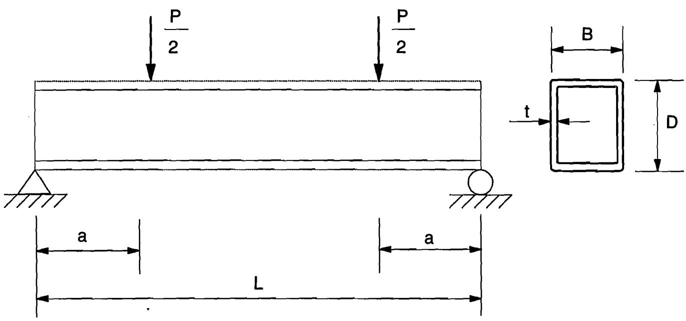

Figure 3 Comparison of Experimental Results and Theoretical Results (BS1)

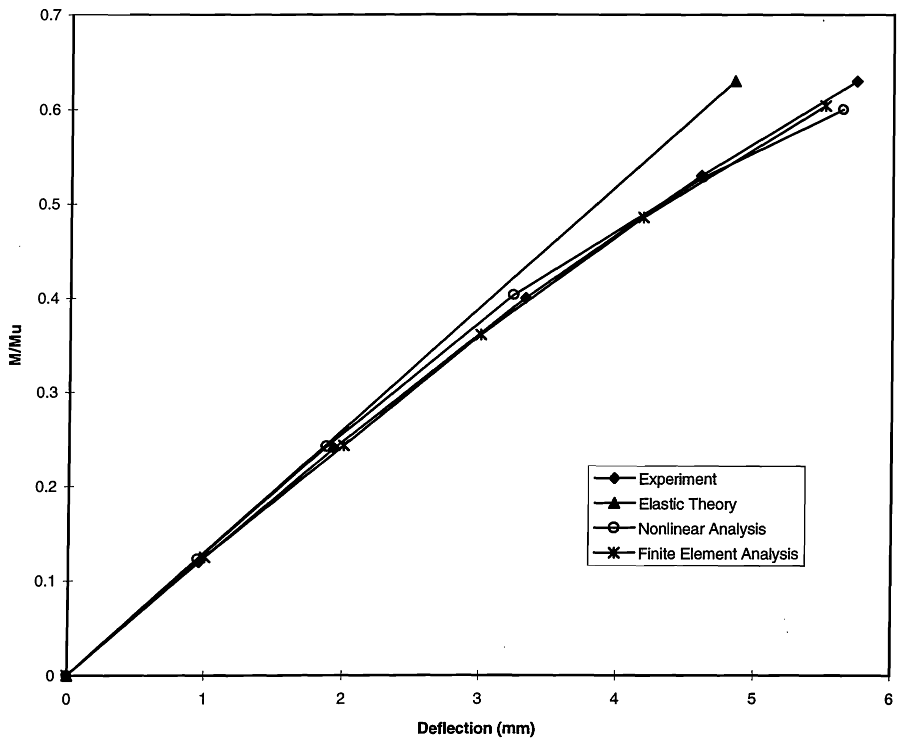

Figure 4 Ps/P* versus Q/G

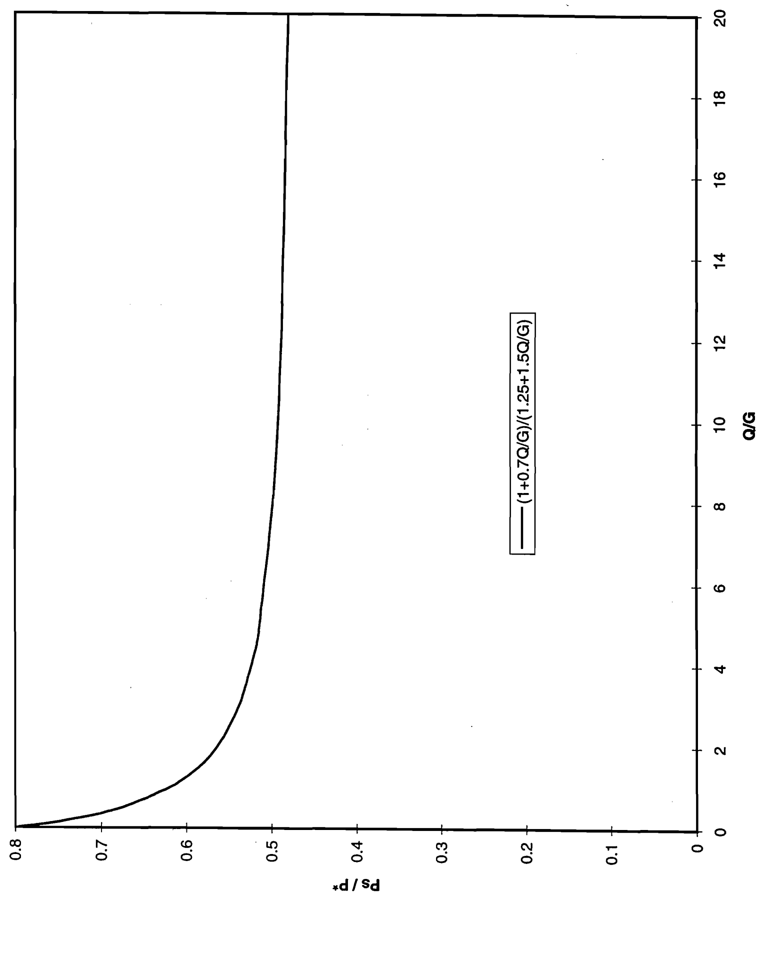

Figure 5 K versus P_s / P^*

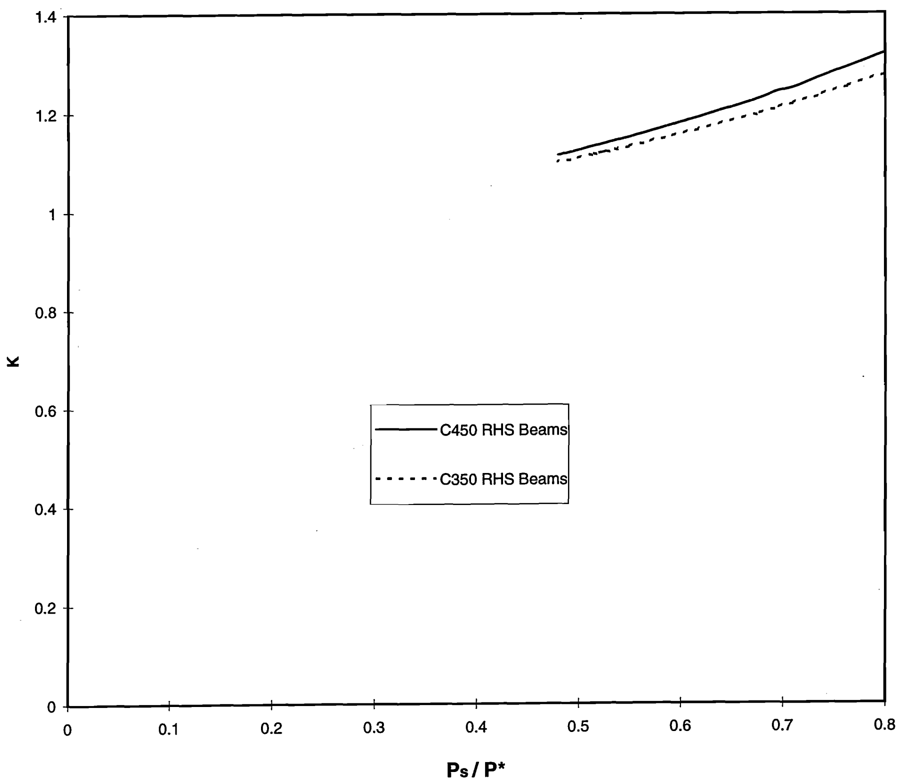

Figure 6 K versus Q/G