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The performance of beams and columns continuously-braced with diaphragms

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THE PERFORMANCE OF BEAMS AND COLUMNS CONTINUOUSLY-BRACED WITH DIAPHRAGMS

by

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Fourth Progress Report

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THE PERFORMANCE OF BEAMS AND COLUMNS CONTINUOUSLY BRACED WITH DIAPHRAGMS

Progress Report No. 4

August 1964

1. Introduction

Under the general title of "Performance of Steel-Framed Buildings and Structural Members Braced with Light-Gage Steel Diaphragms", sponsored at Cornell University by the American Iron and Steel Institute, an investigation has been proceeding in two separate but closely interrelated phases, referred to as:

Subproject A - Performance of Steel-Framed Buildings Braced with Light-Gage Steel Diaphragms

Subproject B - Performance of Beams and Columns Continuously-Braced with Diaphragms.

The investigation now referred to as Subproject B was sponsored by the American Institute of Steel Construction from June, 1961 to June, 1963 after which A.I.S.I. assumed sponsorship with the cooperation of A.I.S.C. The summary report covering the first two years' investigation is "The Performance of Beams and Columns Continuously-Braced with Diaphragms", Third Progress Report by Fisher and Pincus, Report No.313, Department of Structural Engineering, School of Civil Engineering, Cornell University, September 1963, hereafter referred to as the Third Progress Report.

Investigations to date have covered two general areas: (a) the general characteristics of shear diaphragms, with special reference to shear rigidity, and (b) the performance of diaphragm-braced columns. While a small number of additional double-column tests remain to be performed, Subproject B is essentially at the point of completion of column studies and the initiation of tests on diaphragm-braced beams. Unexpected difficulties in staffing and materials supply have resulted in some unavoidable delays, but there is considerable reason to expect that the beam studies can be substantially completed by the end of the present contract period, May 1965.

This present report, to be known as the Fourth Progress Report, covers investigations for the period approximately October 1, 1963 to August 15, 1964.

During this reporting period, work has progressed in three related areas: (1) improvement of understanding of diaphragm behavior, (2) inelastic behavior of diaphragm-braced columns, and (3) plans for tests on diaphragm-braced beams. Each of these will be discussed in detail in the following sections of the report.

Effective shear modulus of corrugated diaphragms, as determined under Subproject B by means of double-beam assemblies (see Third Progress Report), has been correlated experimentally and, with partial success, theoretically with the shear modulus as determined under Subproject A by means of rectangular frame tests. Practically identical experimental results are obtained by both kinds of tests for small size diaphragms. In addition, the results for the large diaphragms of Subproject A have been correlated also with the small diaphragm results to a substantial degree. Cooperation between the two subprojects, therefore, has provided considerably more generality and certainty of diaphragm behavior than heretofore available. An empirical expression for the shear modulus of standard (and similar) corrugated sheets has been developed. To this end, four additional double-beam shear tests have been performed.

Two additional double-column tests have been performed and a better empirical expression for prediction of failure loads of diaphragm-braced columns in the inelastic range has been developed.

Planning for the projected diaphragm-braced beam phase of the program is well under way, including studies of suitable beam sections, range of beam slenderness, and details of the test set-up.

2. Recapitulation of Diaphragm Tests to Date and Correlation with Subproject A.

In Section 1 preceding, reference was made to the correlation which has been achieved between results of both Subprojects A and B. As a matter of history, it was discovered shortly after initiation of the braced-column and -beam program (Subproject B) in 1961 that little information was available on shear stiffness of corrugated diaphragms, information that was vital to the testing of braced members. In order to simulate in a simple fashion the action of diaphragms spanning between adjacent columns, a special shear test using a double-beam-diaphragm assembly was devised and proved to be satisfactory (see Third Progress Report for details of tests).

It has been possible subsequently to show that these tests give shear moduli that are essentially identical with those of both the large and small diaphragms tested under Subproject A by means of rectangular shear frames.* Fourteen double-beam shear tests were reported in the Third Progress Report and are repeated in Table I hereafter; four additional and similar tests also are reported in Table I. Rectangular-frame shear tests are reported elsewhere*. The consistency and extent of correlation of the various tests may be observed in Figure 1.

In Figure 1, N is the fastener spacing in the sense of one fastener occurring in every Nth valley of the corrugated sheet. Width, w, is the dimension of the sheet in the direction of the corrugations (hence, in the braced column tests it is the dimension perpendicular to the column axis); in the documentation of Sub-project A, this same dimension is referred to as "length".

When plotted on a log-log coordinate system, Figure 1, the effective shear modulus, G_{eff}, of a diaphragm was found to have a closely linear relationship both to width of sheet and to connector spacing, indicating an exponential variation. Consequently, shear modulus can be described by an expression of the form

$$G_{eff} = K(1/N)^{\alpha} w^{\beta} \qquad (2\text{-}1)$$

where the exponents α and β are the slopes of the straight lines on the log-log plots and K is taken as a constant for the type of diaphragm tested. In reality, K is a complex expression which, for any general diaphragm, must include at least the gage of the material, Young's modulus E, and a shape factor related to the corrugation configuration.

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* See "Second Summary Report on Tests on Light Gage Steel Shear Diaphragms", by L. D. Luttrell, August 19, 1964 and related documents.

Subsequently, for 26 gage "Plenum" sheet, the following preliminary expression was found to be satisfactory for predicting shear rigidity:

$$G_{eff} = 26(1/N)^{1.74}w^{1.43} \qquad (2\text{-}2)$$

Due to slight dimensional differences between 26 gage "Plenum" and 26 gage standard corrugated sheet, this expression is not immediately applicable to the latter. For 26 gage standard corrugated sheet, K should be approximately 18.5, the exponents remaining the same.

In attempting to correlate the results of both types of 26 gage sheet, it was observed that, for the same N and w, the effective shear moduli of the two types of sheet are related nearly inversely as their respective developed (or unfolded) lengths of a single corrugation, i.e.

$$\frac{G_{Plenum}}{G_{std\ corr}} = \frac{\text{Developed length of a standard corrugation}}{\text{Developed length of a "Plenum" corrugation}}$$

$$= 1.41 \qquad (2\text{-}3)$$

This factor has been used in plotting the results for 26 gage standard corrugated sheet (from Subproject A) in Figure 1.

One shear diaphragm test was made under Subproject A using 17 3/4-inch wide Plenum sheeting (as in Subproject B) with a fastener spacing of N = 3. The resulting G_{eff}, determined as 207 ksi, is plotted as a triangular symbol in Figure 1, and is in good agreement with the predicted value. This offers further encouraging evidence that the simpler rectangular frame shear test can be substituted for the more complicated double-beam shear test.

The joint effort of personnel of both subprojects, which has been very fruitful to this point, is continuing in an attempt to add further evidence of correlative behavior of diaphragms and, hopefully, to achieve a completely general expression for shear rigidity.

3. Recapitulation of Column Studies (to date of Third Progress Report)

It may be remembered from the Third Progress Report that a theoretical solution has been accomplished for the problem of elastic failure (maximum load in elastic range) of a column braced about its weak axis by a shear-rigid diaphragm. The general solution covers also the failure of elastic beams supported against lateral instability by shear-rigid diaphragms. In particular, for centrally-loaded columns failing about the weak axis with negligible twist, the failure load is predicted to be

$$P_{pred} = P_{yy} + Q \qquad (3\text{-}1)$$

where P_{yy} is the weak-axis buckling load of an unbraced column

and Q is the effective shear rigidity of the supporting diaphragm

The effective shear rigidity is defined as

$$Q = wt\,G_{eff} \qquad (3\text{-}2)$$

where w is the width of diaphragm (normal to column axis) contributing to the support of a column

t is the thickness of the diaphragm material

and G_{eff} is the effective shear modulus of the diaphragm for given width, corrugation form, and edge connector spacing.

Equation (3-1) is valid theoretically in the elastic range only, that is for $P_{yy} \leq P_{pred} \leq A\,\sigma_{e.l.}$

where A is the cross-sectional area of the column and $\sigma_{e.\ell.}$ is the elastic limit stress of the column material. It will be noted also that if $P_{xx} \leq A\ \sigma_{e.\ell.}$, then $P_{pred} \leq P_{xx}$.

For columns failing in the <u>inelastic</u> range (i.e. $A\ \sigma_{e.\ell.} < P_{pred} \leq P_{xx}$), no suitable theory is available yet and it is thus necessary to rely on experimental information.

The results of twelve double-column tests covering both the elastic and inelastic ranges and using diaphragm bracing on either one or both flange faces of the columns, were reported in Table III, Third Progress Report and are repeated in Table III hereafter. They were in substantial agreement with the elastic theory or with the tentative empirical inelastic load expression, as appropriate.

The magnitude of the increment by which the failure load exceeds P_{yy} for diaphragm-braced columns depends on the slenderness of the column and on the shear rigidity of the diaphragm. For very slender columns (low P_{yy}) the load may be increased as much as tenfold over P_{yy} (see Fig. 3).

Three distinct cases can be recognized:

a) P_{yy} elastic, P_{xx} elastic - In this case, Eq. (3-1) is valid but $P_{pred} \leq P_{xx}$ irrespective of Q. No tests of this kind have been performed; at least one should be performed, probably with $Q >> P_{xx} - P_{yy}$, in order to determine whether failure in the strong direction can be forced to take place.

b) P_{yy} elastic, P_{xx} inelastic - In this case Eq. (3-1) is valid provided $P_{pred} \leq A\sigma_{e.\ell.}$ (Columns COO, CPP, CII). If the maximum load exceeds $A\ \sigma_{e.\ell.}$, then it can be predicted, in the absence of a suitable theory, only by comparison with experimental results (Columns CBB, CFF, CNO, CNN, CPQ). In the latter instance, presumably P_{max} could reach P_{xx} with sufficient Q supplied, but in tests to date failure has always occurred in the weak direction with P_{max} never exceeding about 90 percent of P_{xx}. The reasons for this are not yet known, but concern over such a deviation should not overshadow the really significant fact that P_{max} can be increased several fold over P_{yy} with rather light diaphragm bracing.

c) P_{yy} inelastic, P_{xx} inelastic - In this case, only experimental results may be relied upon at present. (Columns CQQ-1, CQQ-2, CKK-1, CKK-2). Results of these tests have been entirely consistent with those of the inelastic tests of case (b).

Column tests performed since September 1963 and those projected for the near future are discussed under Section 4 and are correlated with all previous tests.

4. Additional Column Tests

Two recently conducted double-column tests are reported in Tables II and III. Both tests, for slenderness values not previously covered, utilized diaphragm bracing on one flange face only. Specimen CIQ (L/r_y = 120) was designed to fail in the elastic range and with the purpose, in addition to provision of new data, of examining the influence of upper head motion on test results, as explained below. Specimen CMM-1 (L/r_y = 50) was designed so that P_{yy} as well as P_{max}, was inelastic. Both tests were in excellent agreement with previous tests.

In earlier experiments, the upper head of the testing machine, carried on the main vertical screws of the machine, was able to move laterally within the limits of restraint afforded by bending stiffness of the screws. While the loads on the individual column specimens can be made concentric to close tolerance, the entire specimen assembly may be slightly eccentric in the machine, that is relative to the screw positions. It is this slight total eccentricity which causes upper head sway. Such sway normally does not affect a single column specimen, but may superpose a shear force on a double-column-and-diaphragm assembly in addition to that caused by buckling deflections, hence the danger that premature connector failure may occur at one or the other end of the diaphragm. In order to prevent head sway, the upper head was braced laterally against the main vertical columns of the machine, an entirely feasible procedure since the head does not move vertically during the test except for negligible elastic deflections of the machine. Axial deflection of the test columns is taken up by the loading jacks.

Inasmuch as the results of the braced-head tests agreed quite well with those of earlier tests, it has been concluded at this point that lateral upper head motion is not a significantly harmful factor in the earlier tests. However, future column tests will utilize a braced upper head as a preferable test procedure.

For further confirmation of the conclusions reached to date, two additional double-column specimens as given below are to be tested, after which the column testing phase of the program will be considered completed. The additional tests are:

$$L/r_y = 160, \quad Q = 70 \text{ approximately}$$

$$L/r_y = 160, \quad Q = 120 \text{ approximately}$$

5. Prediction of Maximum Column Load, Especially in the Inelastic Range

In the absence of a suitable theory, prediction of column failure loads in the inelastic domain must be based on experimental results. In the Third Progress Report, a tentative expression for predicted load of the form

$$P_{pred} = A\,\sigma_{e.\ell.} + 3\sqrt[3]{Q-\Delta P} \qquad (5\text{-}1)$$

was fitted to the test results and used chiefly to provide some assurance of consistency among the results of the column tests. It was not regarded as a final expression and subsequently has been discarded in view of some obvious shortcomings such as lack of smooth transition from the elastic curves, no relationship to P_{xx} as an upper limit, and unsatisfactory prediction for specimens having $P_{yy} > A\,\sigma_{e.\ell.}$.

Other types of empirical expressions may be fitted to the test data, all of which require some compromise in order to preserve simplicity. The more accurate the fit, the more complex in form is the expression. Curves of exponential form are one obvious possibility and are presently being studied.

Another possibility, one which is used in this report for comparative purposes, is a hyperbola of the form

$$y = \frac{x}{\alpha+\beta x} \qquad (5\text{-}2)$$

which has an initial slope of $1/\alpha$ and an asymptotic limit of $1/\beta$.

On a P-Q plot as in Fig. 3, this expression, for an initial unit slope and limit R_{max}, becomes

$$R = \frac{Q}{1+ Q/R_{max}} \tag{5-3}$$

provided the origin is always at Q=0. But the origin should be at ΔP, so that the expression is more properly

$$R = \frac{Q - \Delta P}{1+ (Q-\Delta P)/R_{max}} \tag{5-4}$$

where $\Delta P = A\ \sigma_{e.\ell.} - P_{yy}$ = the elastic load range

$R_{max} = P_{xx} - A\ \sigma_{e.\ell.}$ = the inelastic load range

and by definition the predicted inelastic failure load is

$$P_{pred} = A\ \sigma_{e.\ell.} + R \tag{5-5}$$

However, a compromise is necessary at this point. Clearly, at Q = 0 (no bracing) the failure load should be P_{yy}, requiring that the expression be changed to

$$R = \frac{Q-\Delta P}{1+ Q/R_{max}}^{*} \tag{5-6}$$

But then the initial slope is no longer unity as it should be to provide smooth transition from the corresponding elastic curve. Either a discontinuous transition (initial slope $\neq 1$) or lack of coincidence with P_{yy} must be accepted. It has been elected for this report to accept discontinuity and to use expression (5-6). While not entirely satisfactory, this expression is far better than that which was used in the Third Progress Report.

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* e.g. if $P_{yy} = A\ \sigma_{e.\ell.} + Y$ (inelastic), then $\Delta P = A\ \sigma_{e.\ell.} - P_{yy}$ $= - Y$; then $R = Y/(1+0) = Y$ and $P_{pred} = A\ \sigma_{e.\ell.} + R = P_{yy}$(always).

The expression is shown schematically in Fig. 2 for various values of L/r_y and specifically in relation to test results in Fig. 3. It is further plotted linearized in Fig. 4 in relation to the test results for the inelastic columns. The test results will be seen to be very consistent among themselves and also, as a group, consistently about ten percent under the predicted values.

The prediction perhaps is actually closer than appears in Figs. 3 and 4, since the upper limit P_{xx} has been calculated in all cases for fixed-fixed end-conditions in the strong direction of the column. The actual test set-up probably does not quite provide full fixity, so that the actual P_{xx} would be somewhat lower than assumed and the prediction therefore better. The actual end-restraint of the test set-up eventually will be determined experimentally and an adjustment made for the actual virtual buckling length. Meanwhile, this slight difference between the assumed and actual P_{xx} has no influence on the test loads reached or on the general efficacy of the proposed expressions (5-5) and (5-6).

Within the range of these experiments, then, it is now possible to predict within ten percent the maximum load, either elastic or inelastic, which can be reached by a diaphragm-braced concentric column. The elastic range is supported by theory; the inelastic range is supported only by experiment. One of the objectives of the program, presently being pursued, is to develop a theoretical solution for the inelastic range.

For reasons not yet known, it appears that diaphragm-braced columns always fail in the weak direction and are never forced into the strong-axis failure mode (P_{xx} never quite reached), even when large bracing capacity is provided.

6. Diaphragm-Braced Beam Program

An approximate solution in the elastic range has been obtained for the critical moment at which lateral buckling occurs for a uniformly bent beam laterally supported by diaphragms. From Progress Report No. 3, the applicable expression, assuming that twist of the beam is negligible, is

$$M = \sqrt{\left(\frac{d^2 P_{yy}}{4} + GK\right)(P_{yy} + Q} \qquad (6\text{-}1)$$

If twist is not negligible, the following equation must be used:

$$M = \sqrt{\left(\frac{d^2}{4} P_{yy} + GK + H + \frac{Qd^2}{4}\right)(P_{yy} + Q)} - \frac{Qd}{2} \qquad (6\text{-}2)$$

in which H is a measure of the flexural rigidity of the diaphragm.

Calculations indicate that the increased carrying capacity due to diaphragm support can be substantial. The magnitude of the increase depends on the relative stiffness characteristics of the beam and supporting diaphragm.

To check the theoretical solution in the elastic range, and to provide the basis for an empirical expression in the inelastic range, a minimum of eight tests on diaphragm-braced beams are proposed. As indicated in Table IV and Fig. 5, tests are proposed for two ratios of L/r_y or Ld/Af, (1) for the middle of the elastic range, (2) for the end of the elastic range, (3) for the middle of the plastic range and (4) near the full plastic moment.

The specimens will be fabricated as two parallel beams identical in size, span, and loading, and interconnected by a corrugated steel diaphragm spanning between the beams and attached to their compression flanges with corrugations perpendicular to the beam axes.

Preliminary designs of a suitable test set-up have been developed. It will be important to eliminate accidental and undesirable restraints from the set-up, in order that reliable results be obtained. An arrangement utilizing loading by dead weights seems to offer the best solution.

Further theoretical studies in both the elastic and inelastic range will continue concurrently with the experimental test program.

7. Summary and Conclusions

a) Effective shear modulus of corrugated sheets has been found to be the same for (standard or similar) corrugated sheets used in Subprojects A and B. Results of radically different shear tests have been almost exactly correlated and an empirical expression relating effective shear modulus to sheet width and edge-connector spacing is suggested.

b) Results of all tests on diaphragm-braced columns are consistent among themselves and are consistently about ten percent under predicted values. Two additional column tests have been performed, and two others are proposed to complete the column investigation.

c) A new empirical expression is suggested for prediction of failure loads of diaphragm-braced columns failing in the inelastic range.

d) The strength of slender columns may be increased several fold over the weak-axis buckling load if a modest amount of properly-connected shear rigid diaphragm is used as lateral bracing. The increase in strength of columns as more bracing is provided is relatively less after the onset of inelasticity in the column; or in another sense the addition of diaphragm bracing is increasingly less efficient in supporting columns, the higher the inelastic failure load of the column.

TABLE I

DOUBLE BEAM-SHEAR TEST SUMMARY

(including tests reported in Third Progress Report)

Material	Area(in^2)	Width(in)	(1) N	$(\frac{P}{d})$(k/in) test	Q(k)	G_{eff}(ksi)
0.024 x $1\frac{1}{4}$						
Aluminum	0.336	14	8	0.670	12.5	37.2
"	"	"	4	1.087	34.9	103.8
"	"	"	2	2.230	95.9	285.0
"	"	"	1	5.050	246.7	733.0
26 Gage "Plenum"						
Galv. Steel	0.504	28	8	1.210	41.4	82.2
" "	"	"	6	1.610	62.9	124.8
" "	"	"	4	3.020	138.2	275.0
" "	"	"	2	7.390	372.0	738.0
" "	"	"	3	4.670	226.4	450.0
" "	"	"	1	16.30	849.0	1686.0
" "	0.319	17 3/4	8	0.665	12.3	38.5
" "	"	"	4	1.272	44.8	140.3
" "	"	"	2	3.310	153.7	482.0
" "	"	"	1	10.860	559.8	1752.0
" "	0.252	14	6	1.930	13.7	54.4
" "	"	"	3	2.440	41.1	163.
" "	"	"	2	3.286	86.3	343.
" "	"	"	1	11.670	534.8	2120.

(1) Pins placed at every Nth corrugation.

TABLE II - DESCRIPTION OF DOUBLE-COLUMN TEST SPECIMENS

An Extension of Table II, Third Progress Report

Column Section: 4 1 7.7; $r_x = 1.64$, $r_y = 0.59$, $A = 2.21$

Specimen No.	Diaphragm Support (a)	Connectors (d)	Length Between Knife-edges, in.	L/r_y	$kL/r_x (k = 0.5)$
CIQ	26g x 17 3/4" Granco "Plenum" (c)	Every third valley	71.2	120	21.6
CMM-1	"	Every other valley	29.5	50	9.0

Notes: (a) Width is overall, not between lines of connectors. (c) On one flange face only.

(d) Symmetrical about mid-length.

TABLE III SUMMARY OF DOUBLE-COLUMN TESTS

PREDICTED AND ACTUAL FAILURE LOADS

(being a revision and extension of Table III, Third Progress Report)

	Specimen No.	P_{yy} (1)	P_{xx} (1)	a/L	L_2/L_1	K_1	K_2	Q_1	Q_2	Q	P_{pred} (3)	P_{test}	P_{test}/P_{pred}	P_{CRC} (2)
Elastic	COO	13.3	105.8	.0478	4.07	.502	.304	73.5	20.8	43.2	56.5	49.5	0.876	104.2
	CPP	13.3	105.8	.0485	4.07	.501	.303	73.5	20.8	43.1	56.4	48.5	0.860	104.2
	CII	13.3	105.8	.0493	0	.804	.000	36.8	0	29.6	42.9	39.5	0.921	104.2
	CIQ	44.6	110.0	.0820	0	.670	.000	35.9	0	24.0	68.6	62.7	0.914	109.5
Inelastic	CBB	8.2	102.0	.0406	0	.837	.000	95.9	0	80.3	82.1	77.5	.944	99.4
	CFF	8.2	102.0	.0375	8.00	.347	.504	535	86.3	229.1	93.9	83.0	.884	99.5
	CNO	25.1	108.7	.0637	2.46	.622	.125	560	154	367.6	102.8	86.0	.837	107.8
	CNN	13.3	105.8	.0548	4.95	.437	.343	560	154	297.5	98.5	98.8	1.012	104.2
	CPQ	25.1	108.7	.0626	0	.752	.000	280	0	210	98.9	93.5	.946	107.8
	CQQ-1	83.0	111.0	.1000	0	.612	.000	14.4	0	8.78	89.2	81.0	.909	110.7
	CQQ-2	83.0	111.0	.1080	0	.584	.000	35.9	0	21.0	94.7	89.8	.948	110.7
	CKK-1	83.0	111.0	.1100	0	.577	.000	76.8	0	44.3	99.6	91.8	.922	110.7
	CKK-2	83.0	111.0	.0910	0	.646	.000	280	0	181	106.9	100.6	.942	110.7
	CMM-1	94.0	111.5	.157	0	.450	.000	76.8	0	34.5	103.2	102.5	.993	111.2

Notes: 1 - See Appendix II, Third Progress Report

2 - See Appendix II, Third Progress Report, Eq. II-2 with $\sigma_{RC} = 0.50\,\sigma_y$

3 - Revised values for inelastic specimens according to Eqs. (5-5) and (5-6) herein; values for elastic specimens same as reported in Third Progress Report.

TABLE IV

Proposed Tests on Lateral Buckling of Beams

Order of Ld/A_f	Bracing*	M_{max}	Failure Mode
800, 1050	$< Q_e$	$M < M_{e.\ell.}$	Elastic, lateral buckling
800, 1050	Q_e	$M = M_{e.\ell.}$	" "
800, 1050	$Q_e + \frac{\Delta Q}{2}$	$M_{e.\ell.} < M < M_p$	Inelastic, lateral buckling
800, 1050	Q_p	$M = M_p$	Inelastic, bending (full plastic moment)

* Q_e is the amount of bracing required to reach $M_{e\ell}$, the moment at which outer fibers reach the elastic limit.

Q_p is the amount of bracing required to reach M_p, the full plastic moment capacity.

$\Delta Q = Q_p - Q_e$

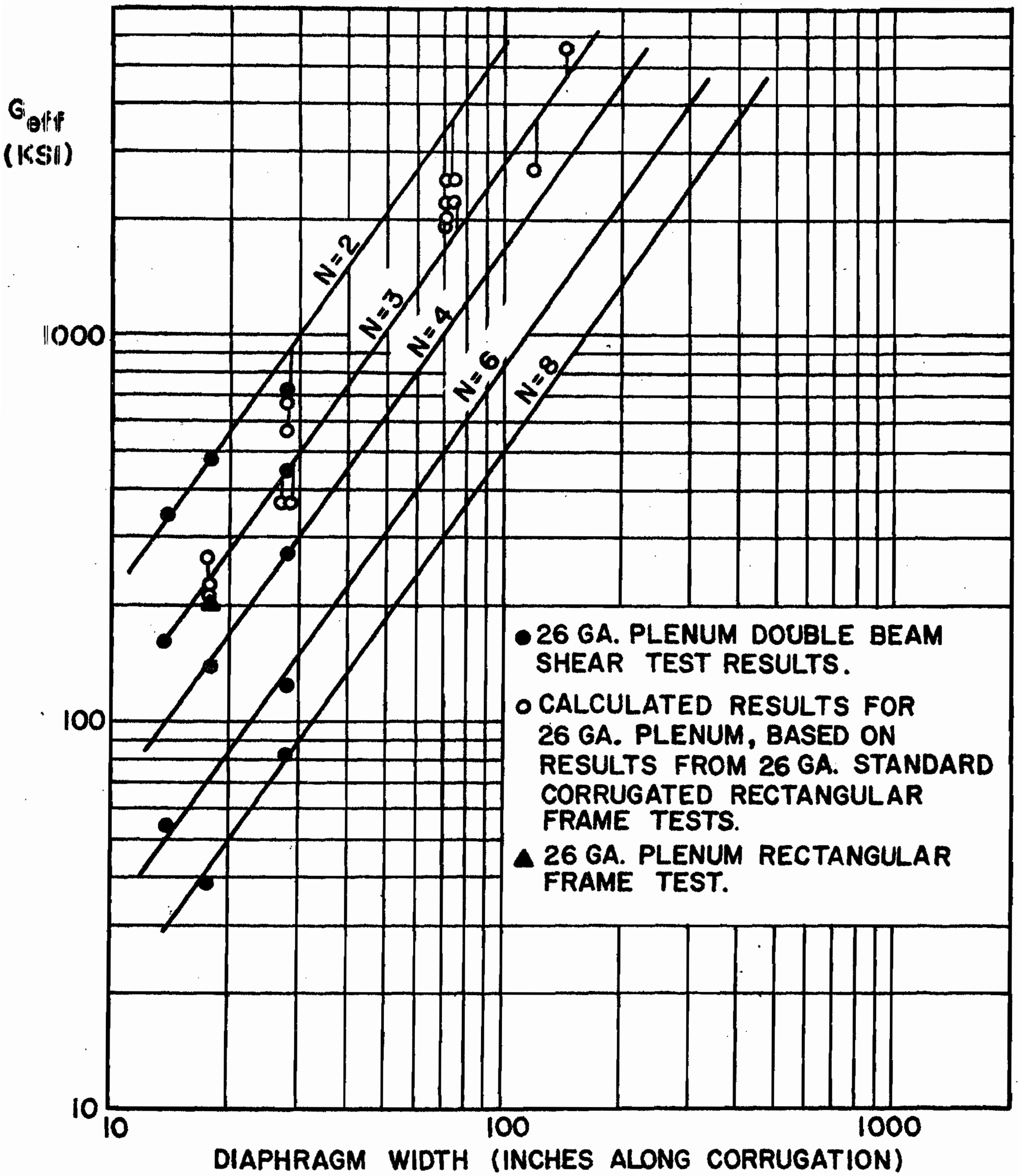

FIG. I EFFECTIVE SHEAR MODULUS OF 26 GAUGE PLENUM STEEL

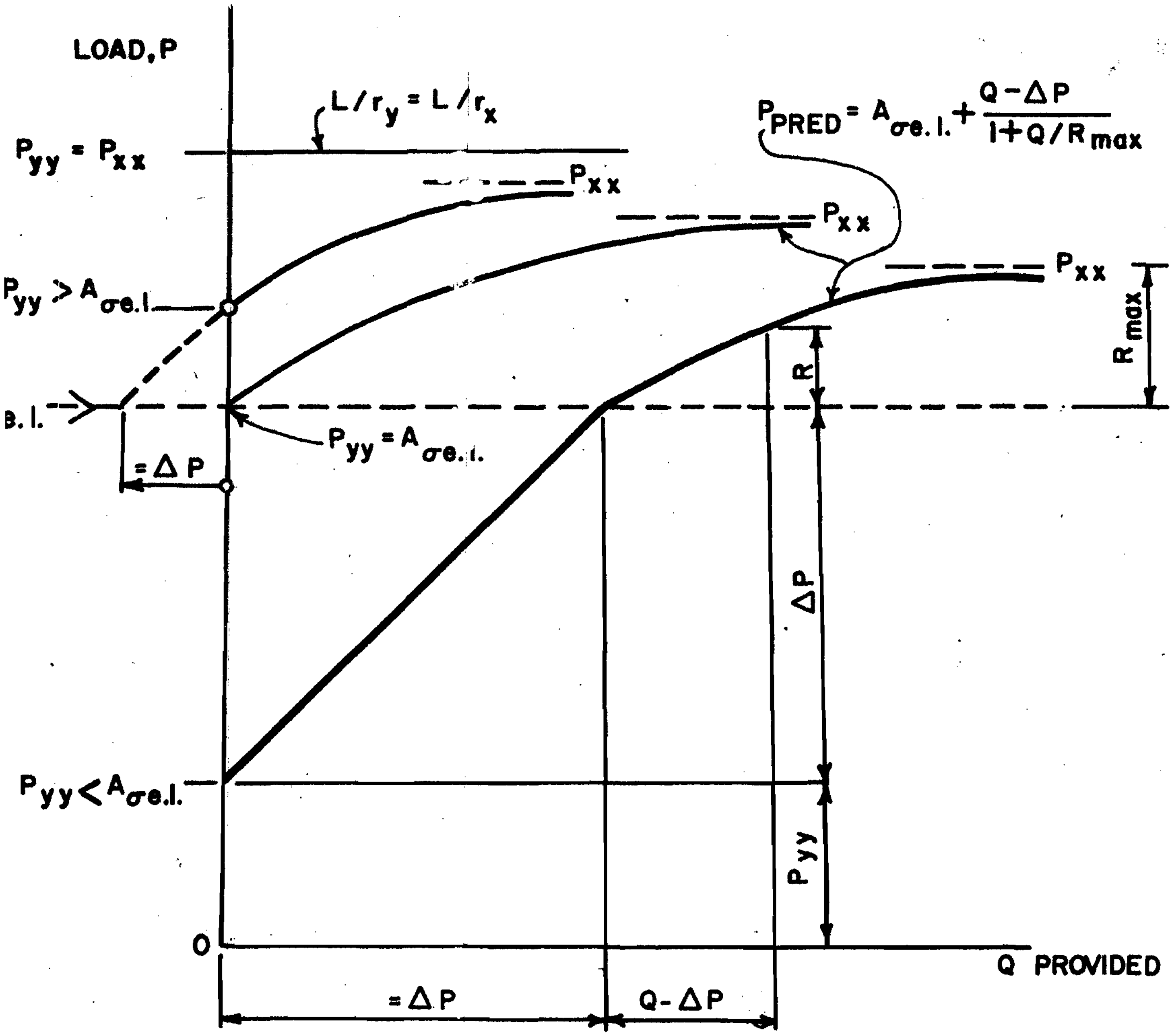

FIG. 2 SCHEMATIC DIAGRAM OF THE INFLUENCE OF DIAPHRAGM BRACING ON THE STRENGTH OF CONCENTRICALLY LOADED COLUMNS OF PARTICULAR CROSS SECTION

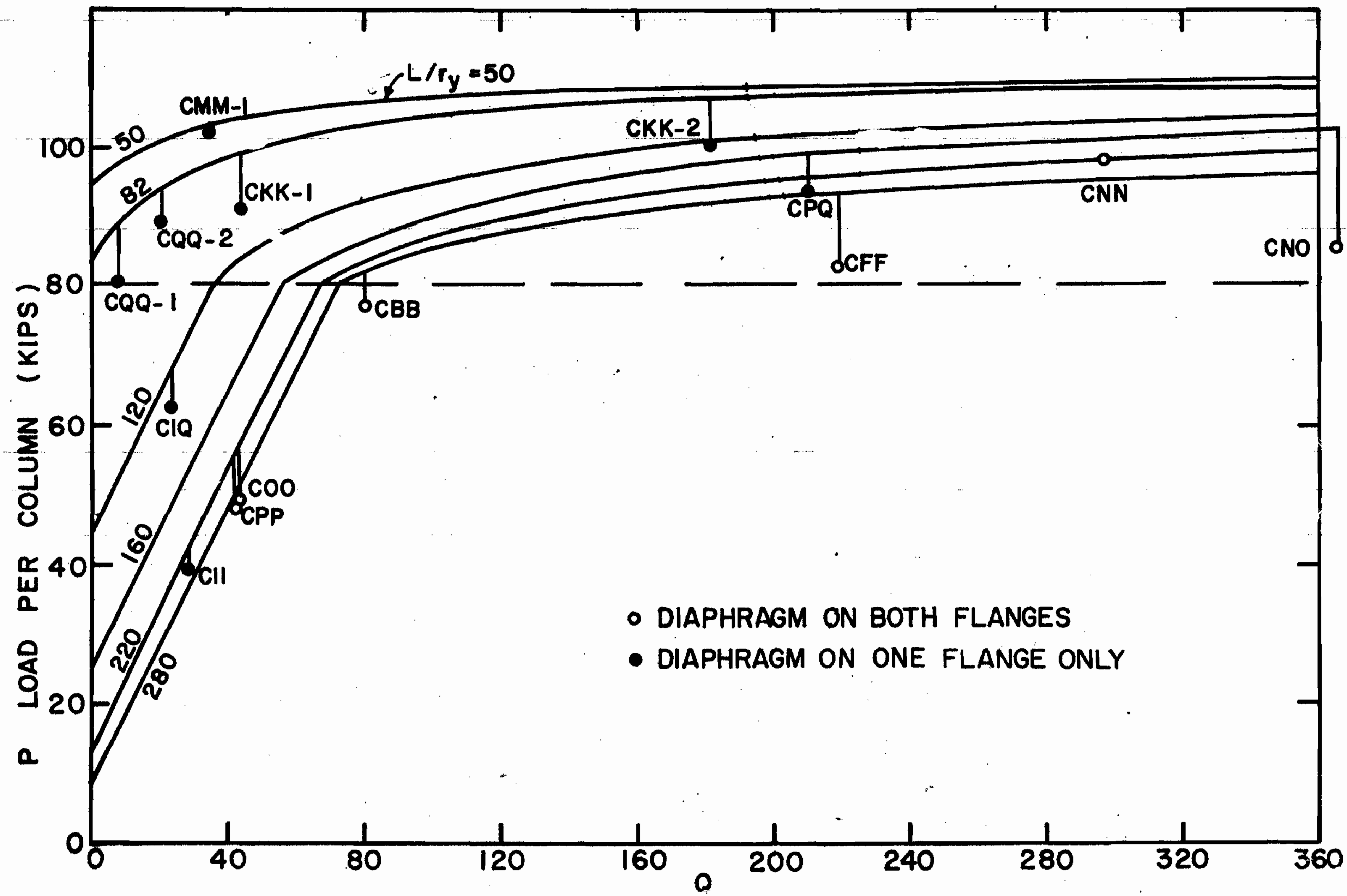

FIG. 3 PREDICTION CURVES AND TEST RESULTS FOR DIAPHRAGM-BRACED COLUMNS

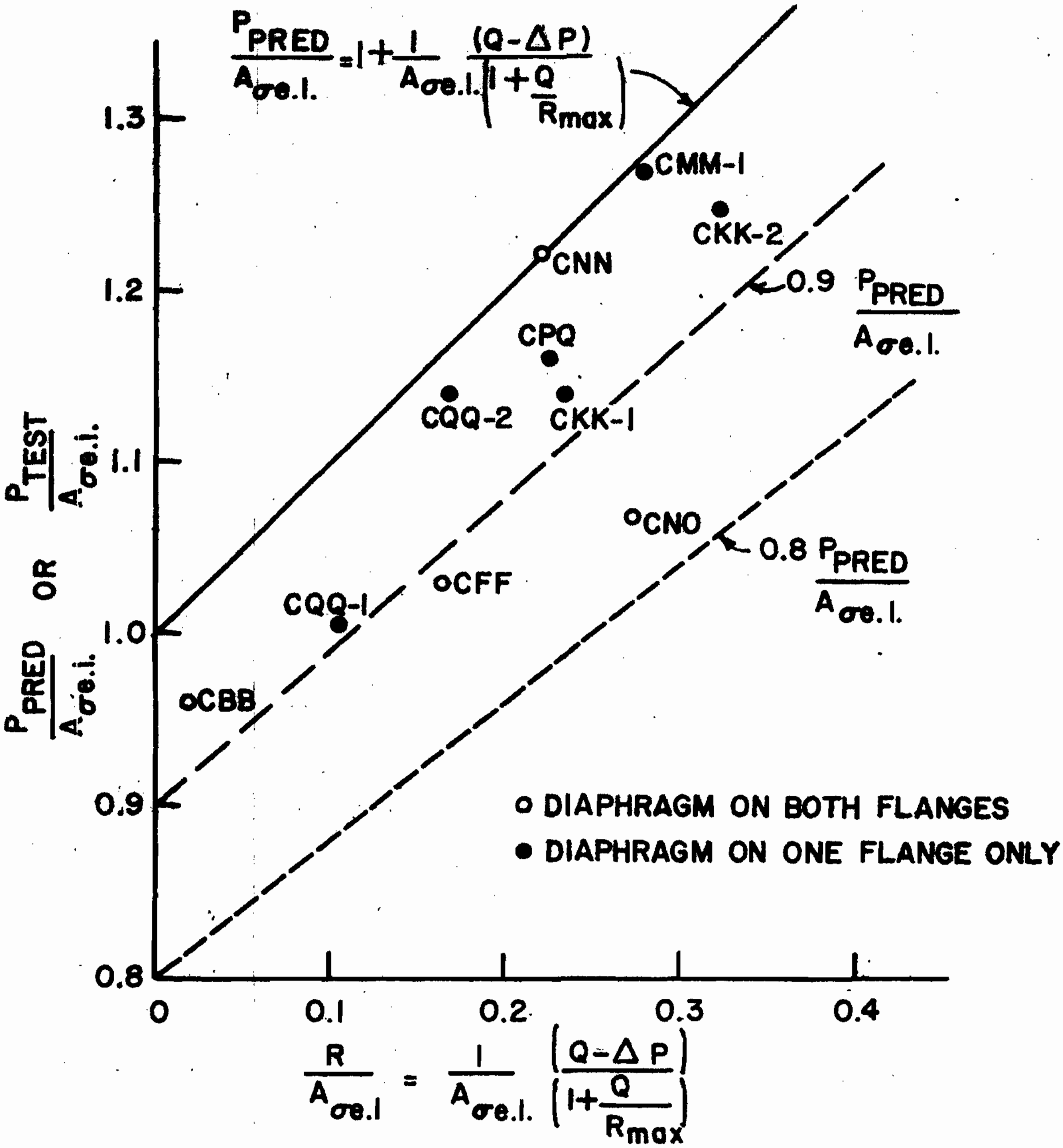

FIG. 4 ACTUAL AND PREDICTED LOADS FOR INELASTIC COLUMNS

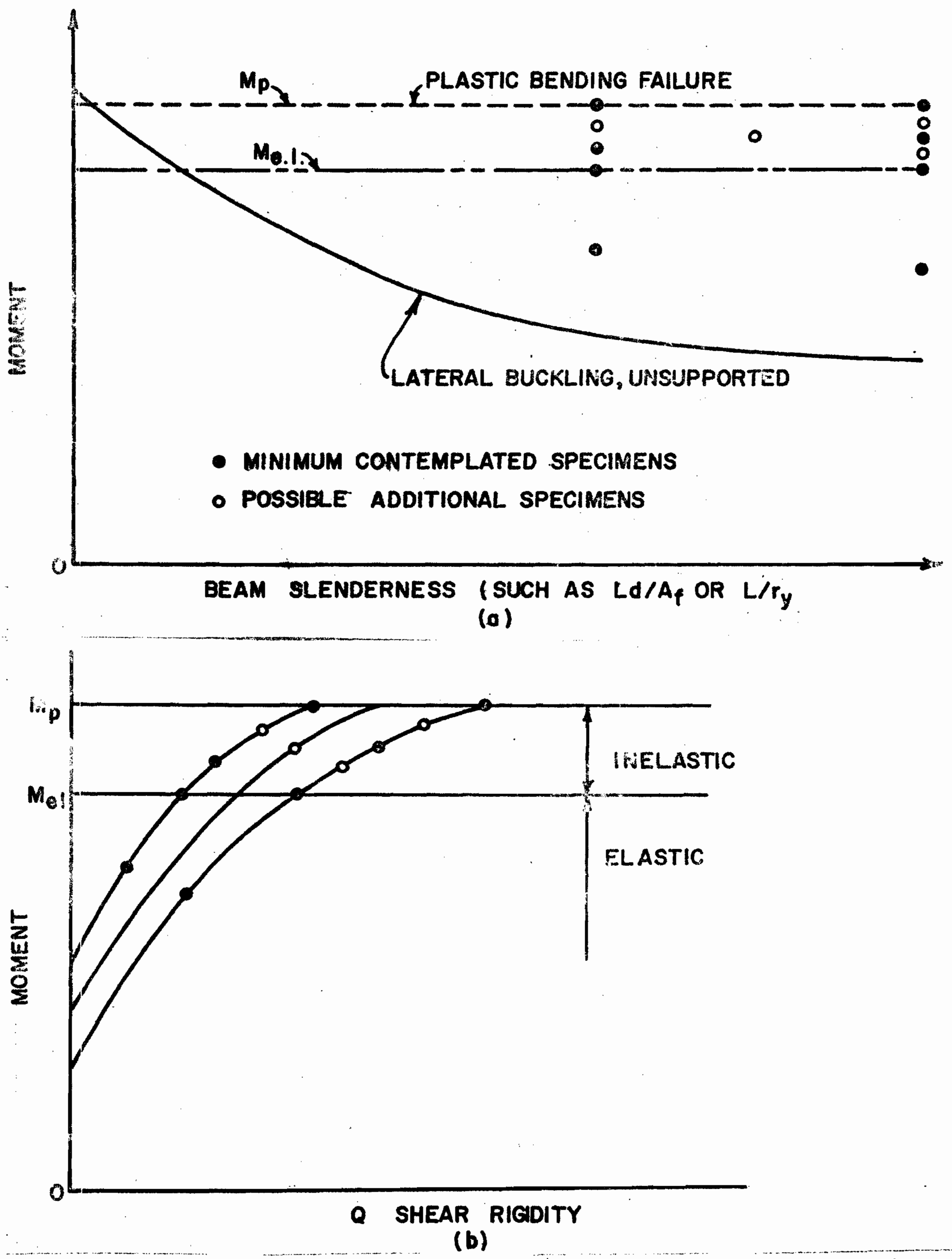

FIG. 5 CONTEMPLATED LATERAL BUCKLING TESTS OF DIAPHRAGM—BRACED BEAMS