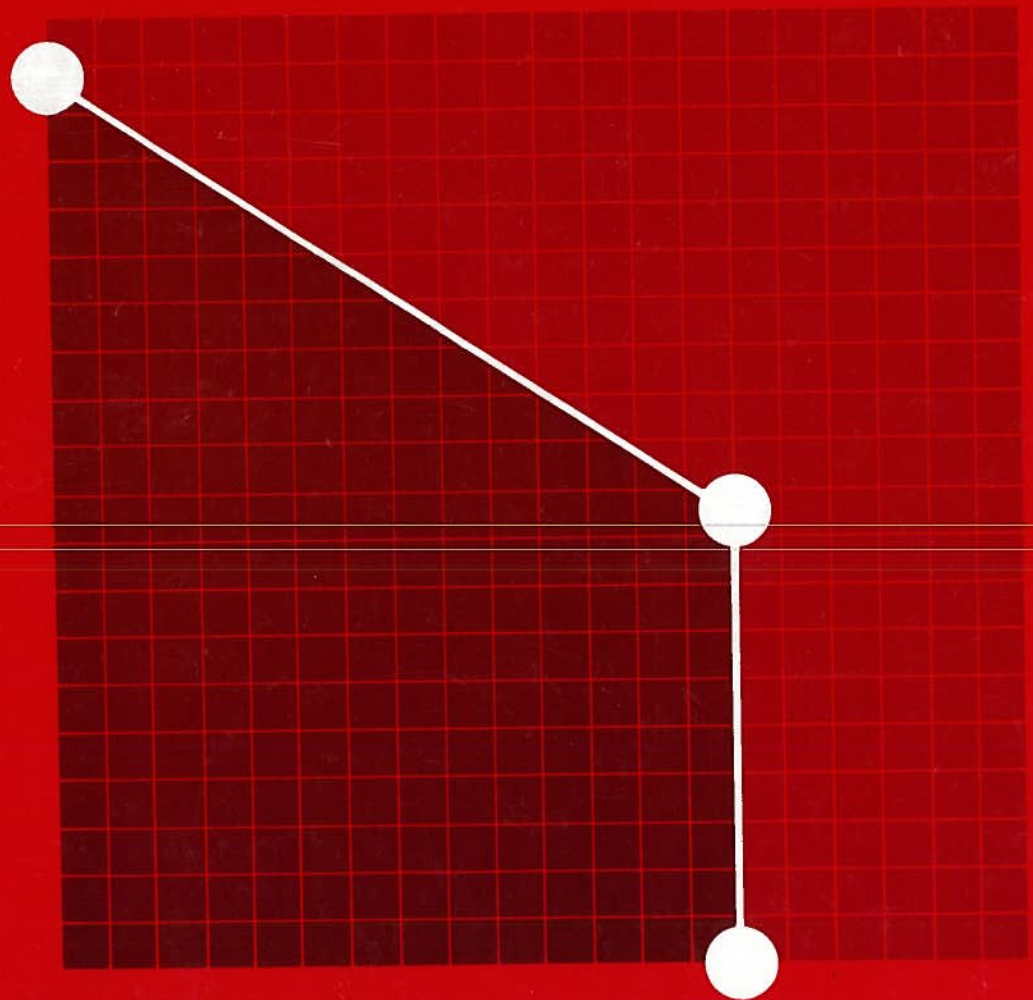




**Rectangular
Concentric and Eccentric
Un-reinforced
WEB PENETRATIONS
IN STEEL BEAMS**

A Design Aid



Rectangular Concentric and Eccentric Un-reinforced WEB PENETRATIONS IN STEEL BEAMS

A Design Aid

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To achieve good operating efficiency for heating and cooling air supply systems, it is necessary to provide ductwork layouts that have optimal airflow characteristics. Therefore, it is desirable to avoid bending the ducts around beams and girders. One of the ways to accomplish this—perhaps the single most commonly used method—is to place the horizontal ductwork in a different plane from that of the beams or girders.

An obvious alternative is to pass the ductwork *through* the beam and girder webs wherever necessary. When this procedure is followed, questions arise as to whether the penetrations can be left without local reinforcement of the web, or whether it is necessary to provide reinforcing such as : web doubler plates, angles welded to the web, or bars welded to the periphery of the hole. (Design of such reinforcement is not within the scope of this discussion; such information is available elsewhere. Moreover, this publication does not consider “castellated” beams.)

The purpose of this publication is to assist designers in the process of determining whether a beam, with either a concentric or eccentric rectangular web hole, can perform satisfactorily *without* reinforcing. Since penetrations through many beams may be required and floor systems tend to be similar throughout a building, an over-design at a typical hole would be repeated a great number of times, thereby leading to the use of unnecessary reinforcing material and added fabrication, thus creating an unwarranted additional cost.

This publication focuses on beam web penetrations where good design practice may eliminate the need for reinforcement, and it provides information on locating holes so that they produce minimum deleterious effects.

Unlike pure tension and compression members that have a constant “design force,” beams and girders in buildings have distributions of “design forces” (shear and bending moments) that vary along their length. Consequently, there are regions where reserve capacity in shear and moment exist. Thus, while the presence of a hole reduces the ultimate shear capacity and ultimate moment capacity *at the hole*, the beam may still perform satisfactorily if the hole is not located at a critical point.

Unfortunately, shear and moment combinations that can be tolerated at a beam section containing a hole are not easily described mathematically. Nonetheless, it has been shown that points on a relatively simple interaction diagram can be derived from formulas given in Figures 3 & 4. While, at first glance, these formulas may seem formidable, the parameters fall into three categories:

- 1) ultimate bending moment and shear resistance of the gross cross-section (M_p and V_p),*
- 2) beam section geometry (A_w , A_f , d), and
- 3) hole description (a , h , e).

Additionally, the tables following the text permit solution of many practical problems without substituting numbers in the formulas.

Interaction Diagram

Frequently, interaction diagrams are used as graphic representations of safe combinations of two—or more—parameters. On each axis, the outermost point is the limiting value of one parameter in the absence of the other. The “interaction curve,” the line connecting these points, (established by theory, laboratory experiments, or other means) defines the maximum combined values that can be tolerated. All points in the region below the interaction curve and between the axes represent “safe” combinations. Often, the parameters plotted on the axes are made non-dimensional; usually this is done by dividing by the largest value that the parameter can reach (i.e., the ultimate value) such that all numerical values are less than unity.

Usually, structural steel designers have their earliest exposure to the interaction method when studying beam-columns. For example, one formula for axial load and bending is frequently expressed as:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1$$

and is a straight line interaction diagram (Fig. 1).

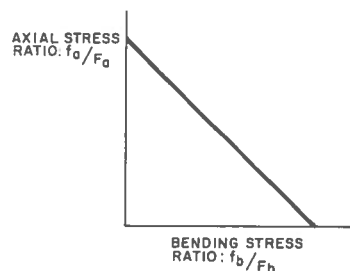


Figure 1

The interaction diagram based on the formulas for beams with web holes is only slightly more complicated than the foregoing; it has a *shear ratio axis*, a *moment ratio axis*, and an *interaction curve* consisting of two straight lines (Fig. 2).

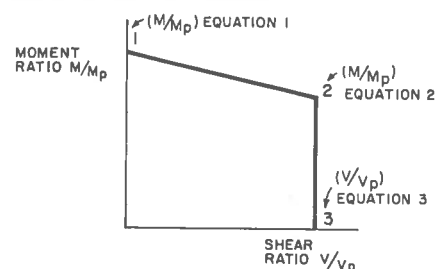


Figure 2.

Equations (1, 2, and 3) define the locations of the three corresponding points in the diagram. Observe that point 1 is on the *moment ratio axis*, point 3 is on the *shear ratio axis*, and point 2 is on a vertical line through point 3.

Strength Considerations

Provisions in Part 2 of the AISC Specifications, allowing certain structures to be proportioned on the basis of plastic design, can be extended to beams with web holes. The rolled shapes, that will be used as beams, are generally designed conventionally—using either allowable stress or plastic analysis. The design aids presented in the following pages are based on the *ultimate strength* of beams with unreinforced rectangular holes. Therefore, for design purposes, the design loads must be amplified by a suitable factor before comparing shear and moment values with the ultimate strength values provided in this publication. It will be observed in subsequent examples that the design loads are increased by “load factors” in accordance with current plastic design practice.

* Definitions of the symbols are given on page 4.

Some of the earliest research in this field was performed on compact beam webs penetrated with a rectangular, concentrically located hole, i.e., with the hole's horizontal centerline coincident with the centerline of the beam. A collapse mechanism with four plastic hinges in the web—one each at the four corners of the hole—gave satisfactory predictions of the ultimate carrying capacity, provided that instabilities did not occur. Subsequent investigations extended the theory to include eccentric holes whose centerlines are parallel to but not coincident with the centerline of the beam. Interaction diagrams, based on these later studies, predicted ultimate capacities that were in conservative agreement with those determined by laboratory experiments.

Stability Considerations*

It is important that the following forms of instability be checked:

Local buckling of the compression flange. The width-thickness ratio of the flange must satisfy the relationship

$$\frac{b_f}{2t_f} < \frac{65}{\sqrt{F_y}}$$

Where

b_f = flange width
 t_f = flange thickness
 F_y is expressed in ksi

Since this is the usual requirement for compact sections, it is not ordinarily a critical factor.

Overall buckling of the tee-shaped compression zone above or below the hole. This is checked by treating this material as if it were an axially loaded column with an effective length equal to the length of the hole. If, however, the length of the hole is less than 4-times the depth of the tee, this check is unnecessary.

Web buckling. This can occur along any edge of the hole, hence the width-thickness ratio of the web, the aspect ratio of the hole, the amount of web remaining, and the magnitude of the shear force are all inter-related. The following checks must be performed:

a) Calculate the hole parameter

$$\frac{a}{h} + \frac{12h}{d}$$

This must be equal to or less than 5.6 for the design to be acceptable.

(Note: All values listed in the tables given in this publication meet this criterion.)

b) Calculate the web width-thickness ratio

$$\frac{d - 2t_f}{t_w}$$

This must not exceed $\frac{520}{\sqrt{F_y}}$. If this parameter is less than $\frac{420}{\sqrt{F_y}}$, the web qualifies as "stocky," for which the aspect ratio $\frac{a}{h}$ must not be more than 3.0, and V_{max} must not exceed $0.667 V_p$. If the parameter is greater than $\frac{420}{\sqrt{F_y}}$ but less than $\frac{520}{\sqrt{F_y}}$, the web must be regarded as "slender," and the upper limits become 2.2 for $\frac{a}{h}$ and $0.45 V_p$ for V_{max} .

Checks for web buckling given here were derived from results of theoretical and experimental investigations of concentric holes. Recent tests, as yet unpublished, have shown that elastic critical loads can be reduced slightly by eccentricity. Therefore, until further information is developed, it is recommended that $\frac{d - 2t_f}{t_w}$ not exceed $\frac{420}{\sqrt{F_y}}$ for webs having eccentric holes.

The stability checks given above are to be performed in addition to the strength check. The latter is made by constructing an interaction diagram based on values given in the subsequent Tables. The illustrative examples should make the details of the procedure clear.

General Considerations

—Since plastic analysis is the basis of the procedure, only steels having $F_y \leq 65$ ksi are included.

—The plastic moment of the beam without a hole, M_p , shall be taken as $Z F_y$. The ultimate shear force, V_p , shall be taken as $0.55 F_y t_w d$.

—As a practical limit, the hole depth must not exceed 70% of the beam depth.

—A distance of about 0.15d should remain intact between the edge of the hole and the outside of the nearest flange. Consequently:

$$\frac{e}{d} + \frac{h}{d} \leq 0.35$$

(Note: All values listed in the tables given in this publication meet this criterion.)

—Hole corners should be rounded to avoid problems with stress concentrations and fatigue. Corner radii should be at least two times the web thickness or $\frac{5}{16}$ -inch, whichever is larger.

—Concentrated loads or reactions must not be applied to the beam within the length of the hole.

—Beams are assumed to be laterally supported throughout their length.

—While beam deflections produced by shear deformation in the vicinity of the hole are small, they are not negligible. Beams with deflection that is near critical, when calculated in the usual manner, should be investigated further.

* Because there is limited experimental data on the stability of beams with eccentric web holes, the information presented herein should not be used for fabricated girders.

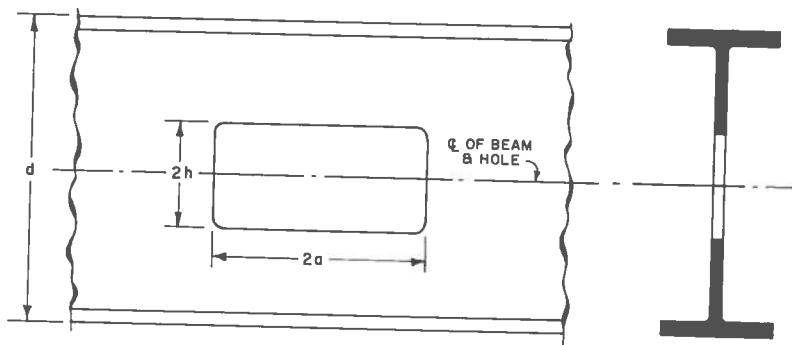


Figure 3. Elevation and cross-section of beam with concentric rectangular web hole.

For a beam with a concentric, rectangular web hole:

Equation 1:

$$\frac{M}{M_p} = \frac{1 + \frac{A_w}{4A_f} \left[1 - \left(\frac{2h}{d} \right)^2 \right]}{1 + \frac{A_w}{4A_f}}$$

Equation 2:

$$\frac{M}{M_p} = \frac{1 - \frac{A_w}{4A_f} \left(1 - \frac{2h}{d} \right) \sqrt{1 + \alpha}}{1 + \frac{A_w}{4A_f}}$$

Equation 3:

$$\frac{V}{V_p} = \left(1 - \frac{2h}{d} \right) \sqrt{\frac{\alpha}{1 + \alpha}}$$

In Equations 2 and 3, $\alpha = \frac{3}{16} \left(\frac{d}{a} \right)^2 \left(1 - \frac{2h}{d} \right)^2$

The Equations

In the equations that follow, the symbols used are defined as:

A_f = cross-sectional area of one flange

A_w = cross-sectional area of the web, the product of beam depth by web thickness.

M/M_p = ratio of bending moment in the beam with a web hole, to ultimate bending moment of the beam without a web hole

V/V_p = ratio of shear force in the beam with a web hole, to the ultimate shear force of the beam without a web hole

a = half-length of web hole

d = depth of beam

e = eccentricity of web hole (transverse distance from mid-depth of beam to mid-depth of web hole; always positive)

h = half-depth of web hole

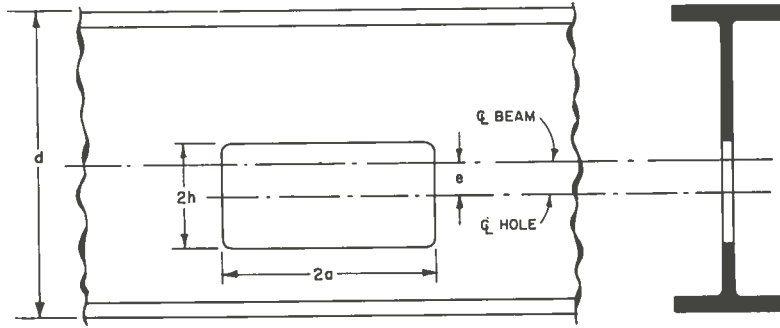


Figure 4. Elevation and cross-section of beam with eccentric rectangular web hole.

For a beam with an eccentric rectangular web hole:

$$\text{Equation 1: } \frac{M}{M_p} = 1 - \frac{\left(\frac{h}{d}\right)^2 \left(1 + \frac{2e}{d} \frac{d}{h}\right) \frac{A_w}{A_f}}{1 + \frac{A_w}{4A_f}}$$

$$\text{Equation 2: } \frac{M}{M_p} = \frac{1 - \frac{A_w}{4A_f} \left(1 - \frac{2h}{d} + \frac{2e}{d}\right) \sqrt{1 + \alpha_B}}{1 + \frac{A_w}{4A_f}}$$

Equation 3:

$$\frac{V}{V_p} = \left[\frac{1}{2} \left(1 - \frac{2h}{d} - \frac{2e}{d}\right) \sqrt{\frac{\alpha_T}{1 + \alpha_T}} \right] + \left[\frac{1}{2} \left(1 - \frac{2h}{d} + \frac{2e}{d}\right) \sqrt{\frac{\alpha_B}{1 + \alpha_B}} \right]$$

In Equations 2 and 3,

$$\alpha_T = \frac{3}{16} \left(\frac{d}{a}\right)^2 \left(1 - \frac{2h}{d} - \frac{2e}{d}\right)^2$$

$$\alpha_B = \frac{3}{16} \left(\frac{d}{a}\right)^2 \left(1 - \frac{2h}{d} + \frac{2e}{d}\right)^2$$

To assist the designer in plotting ultimate-strength moment-shear interaction diagrams, the preceding equations were solved for a range of geometrical parameters. The resulting Tables, included here, employ the following parameters:

Area Ratios:*

$$\frac{A_f}{A_w} = 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0$$

Aspect Ratios:

$$\frac{a}{h} = 1.0, 1.5, 2.0$$

Eccentricity Ratios:

$$\frac{e}{d} = 0.0, 0.05, 0.10, 0.15, 0.20, 0.25$$

Depth Ratios:

$$\frac{h}{d} = 0.10, 0.15, 0.20, 0.25, 0.30, 0.35$$

Close examination of the Tables will show that as the eccentricity increases, the ultimate moment strength *decreases*, and the ultimate shear strength usually *increases* but may, at times, decrease slightly. Moment and shear strength both decrease with increasing aspect ratio. These tendencies have been observed experimentally.

* Although the formulas for $\frac{M}{M_p}$ contain the area ratio $\frac{A_w}{A_f}$, note that the more conventional reciprocal, $\frac{A_f}{A_w}$, appears in the Tables.

TABLES

A Preliminary Note Regarding the Tables

When using these Tables to obtain values for Equations 1 (M/M_p), 2 (M/M_p), and 3 (V/V_p), it may appear that four-way interpolations are needed. Fortunately, this is not the case.

To avoid interpolation, one method is to adjust the dimensions so that the parameters closely match those listed in the Tables. Of course, this may not always be convenient.

A second method has been found — conservative values for plotting the interaction diagrams can be obtained by following this procedure:

- 1) Always round the value of A_f/A_w to the next *lower* value of those listed, i.e., 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0.
- 2) Always round the value of a/h to the next *higher* value of those listed, i.e., 1.0, 1.5, 2.0.

TABLE I ULTIMATE STRENGTH VALUES Beams With Un-reinforced Rectangular Web Penetrations $\frac{A_f}{A_w} = .40$

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
1	.40	1.0	0.00	.10	.985	.445	.769
2	.40	1.0	0.00	.15	.965	.377	.627
3	.40	1.0	0.00	.20	.938	.334	.475
4	.40	1.0	0.00	.25	.904	.325	.327
5	.40	1.0	0.00	.30	.862	.349	.200
6	.40	1.0	0.00	.35	.812	.399	.104
7	.40	1.0	.05	.10	.969	.443	.768
8	.40	1.0	.05	.15	.942	.371	.627
9	.40	1.0	.05	.20	.908	.319	.476
10	.40	1.0	.05	.25	.865	.295	.330
11	.40	1.0	.05	.30	.815	.304	.206
12	.40	1.0	.10	.10	.954	.442	.767
13	.40	1.0	.10	.15	.919	.367	.625
14	.40	1.0	.10	.20	.877	.308	.477
15	.40	1.0	.10	.25	.827	.273	.339
16	.40	1.0	.15	.10	.938	.442	.765
17	.40	1.0	.15	.15	.896	.364	.624
18	.40	1.0	.15	.20	.846	.299	.482
19	.40	1.0	.20	.10	.923	.441	.762
20	.40	1.0	.20	.15	.873	.361	.623
21	.40	1.0	.25	.10	.908	.440	.759
22	.40	1.5	0.00	.10	.985	.371	.734
23	.40	1.5	0.00	.15	.965	.294	.562
24	.40	1.5	0.00	.20	.938	.266	.393
25	.40	1.5	0.00	.25	.904	.282	.250
26	.40	1.5	0.00	.30	.862	.328	.144
27	.40	1.5	0.00	.35	.812	.391	.072
28	.40	1.5	.05	.10	.969	.367	.734
29	.40	1.5	.05	.15	.942	.280	.562
30	.40	1.5	.05	.20	.908	.237	.395
31	.40	1.5	.05	.25	.865	.236	.255
32	.40	1.5	.05	.30	.815	.269	.150

3) Round the value of e/d to the next *higher* value when determining 1 (M/M_p) and 2 (M/M_p), but round to the next *lower* value when determining V/V_p . Listed values are: 0.0, 0.05, 0.10, 0.15, 0.20, 0.25.

4) Interpolate for the value of h/d between those listed, i.e., 0.10, 0.15, 0.20, 0.25, 0.30, 0.35.

TABLE I (cont'd)

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
33	.40	1.5	.10	.10	.954	.364	.732
34	.40	1.5	.10	.15	.919	.269	.563
35	.40	1.5	.10	.20	.877	.213	.402
36	.40	1.5	.10	.25	.827	.197	.269
37	.40	1.5	.15	.10	.938	.361	.730
38	.40	1.5	.15	.15	.896	.261	.566
39	.40	1.5	.15	.20	.846	.193	.416
40	.40	1.5	.20	.10	.923	.359	.728
41	.40	1.5	.20	.15	.873	.254	.572
42	.40	1.5	.25	.10	.908	.358	.726
43	.40	2.0	0.00	.10	.985	.308	.693
44	.40	2.0	0.00	.15	.965	.237	.498
45	.40	2.0	0.00	.20	.938	.228	.327
46	.40	2.0	0.00	.25	.904	.262	.199
47	.40	2.0	0.00	.30	.862	.320	.111
48	.40	2.0	.05	.10	.969	.299	.692
49	.40	2.0	.05	.15	.942	.213	.499
50	.40	2.0	.05	.20	.908	.186	.330
51	.40	2.0	.05	.25	.865	.206	.204
52	.40	2.0	.05	.30	.815	.254	.117
53	.40	2.0	.10	.10	.954	.293	.692
54	.40	2.0	.10	.15	.919	.193	.503
55	.40	2.0	.10	.20	.877	.150	.341
56	.40	2.0	.10	.25	.827	.155	.219
57	.40	2.0	.15	.10	.938	.288	.691
58	.40	2.0	.15	.15	.896	.177	.511
59	.40	2.0	.15	.20	.846	.120	.360
60	.40	2.0	.20	.10	.923	.284	.691
61	.40	2.0	.20	.15	.873	.164	.525
62	.40	2.0	.25	.10	.908	.281	.694

TABLE II ULTIMATE STRENGTH VALUES Beams With Un-reinforced Rectangular Web Penetrations $\frac{A_f}{A_w} = .50$

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
1	.50	1.0	0.00	.10	.987	.519	.769
2	.50	1.0	0.00	.15	.970	.460	.627
3	.50	1.0	0.00	.20	.947	.423	.475
4	.50	1.0	0.00	.25	.917	.415	.327
5	.50	1.0	0.00	.30	.880	.436	.200
6	.50	1.0	0.00	.35	.837	.479	.104
7	.50	1.0	.05	.10	.973	.518	.768
8	.50	1.0	.05	.15	.950	.455	.627
9	.50	1.0	.05	.20	.920	.410	.476
10	.50	1.0	.05	.25	.883	.389	.330
11	.50	1.0	.05	.30	.840	.396	.206
12	.50	1.0	.10	.10	.960	.517	.767
13	.50	1.0	.10	.15	.930	.451	.625
14	.50	1.0	.10	.20	.893	.400	.477
15	.50	1.0	.10	.25	.850	.370	.339
16	.50	1.0	.15	.10	.947	.516	.765
17	.50	1.0	.15	.15	.910	.448	.624
18	.50	1.0	.15	.20	.867	.393	.482
19	.50	1.0	.20	.10	.933	.515	.762
20	.50	1.0	.20	.15	.890	.446	.623
21	.50	1.0	.25	.10	.920	.515	.759
22	.50	1.5	0.00	.10	.987	.455	.734
23	.50	1.5	0.00	.15	.970	.389	.562
24	.50	1.5	0.00	.20	.947	.364	.393
25	.50	1.5	0.00	.25	.917	.378	.250
26	.50	1.5	0.00	.30	.880	.418	.144
27	.50	1.5	0.00	.35	.837	.473	.072
28	.50	1.5	.05	.10	.973	.451	.734
29	.50	1.5	.05	.15	.950	.376	.562
30	.50	1.5	.05	.20	.920	.338	.395
31	.50	1.5	.05	.25	.883	.338	.255
32	.50	1.5	.05	.30	.840	.366	.150

TABLE II (cont'd)

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
33	.50	1.5	.10	.10	.960	.448	.732
34	.50	1.5	.10	.15	.930	.367	.563
35	.50	1.5	.10	.20	.893	.318	.402
36	.50	1.5	.10	.25	.850	.304	.269
37	.50	1.5	.15	.10	.947	.446	.730
38	.50	1.5	.15	.15	.910	.359	.566
39	.50	1.5	.15	.20	.867	.301	.416
40	.50	1.5	.20	.10	.933	.445	.728
41	.50	1.5	.20	.15	.890	.353	.572
42	.50	1.5	.25	.10	.920	.444	.726
43	.50	2.0	0.00	.10	.987	.400	.693
44	.50	2.0	0.00	.15	.970	.338	.498
45	.50	2.0	0.00	.20	.947	.331	.327
46	.50	2.0	0.00	.25	.917	.361	.199
47	.50	2.0	0.00	.30	.880	.410	.111
48	.50	2.0	.05	.10	.973	.393	.692
49	.50	2.0	.05	.15	.950	.318	.499
50	.50	2.0	.05	.20	.920	.295	.330
51	.50	2.0	.05	.25	.883	.312	.204
52	.50	2.0	.05	.30	.840	.353	.117
53	.50	2.0	.10	.10	.960	.387	.692
54	.50	2.0	.10	.15	.930	.301	.503
55	.50	2.0	.10	.20	.893	.264	.341
56	.50	2.0	.10	.25	.850	.268	.219
57	.50	2.0	.15	.10	.947	.383	.691
58	.50	2.0	.15	.15	.910	.287	.511
59	.50	2.0	.15	.20	.867	.237	.360
60	.50	2.0	.20	.10	.933	.379	.691
61	.50	2.0	.20	.15	.890	.276	.525
62	.50	2.0	.25	.10	.920	.377	.694

TABLE III ULTIMATE STRENGTH VALUES Beams With Un-reinforced Rectangular Web Penetrations $\frac{A_f}{A_w} = .60$

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
1	.60	1.0	0.00	.10	.988	.575	.769
2	.60	1.0	0.00	.15	.974	.523	.627
3	.60	1.0	0.00	.20	.953	.491	.475
4	.60	1.0	0.00	.25	.926	.484	.327
5	.60	1.0	0.00	.30	.894	.502	.200
6	.60	1.0	0.00	.35	.856	.540	.104
7	.60	1.0	.05	.10	.976	.574	.768
8	.60	1.0	.05	.15	.956	.519	.627
9	.60	1.0	.05	.20	.929	.479	.476
10	.60	1.0	.05	.25	.897	.461	.330
11	.60	1.0	.05	.30	.859	.467	.206
12	.60	1.0	.10	.10	.965	.574	.767
13	.60	1.0	.10	.15	.938	.516	.625
14	.60	1.0	.10	.20	.906	.471	.477
15	.60	1.0	.10	.25	.868	.444	.339
16	.60	1.0	.15	.10	.953	.573	.765
17	.60	1.0	.15	.15	.921	.513	.624
18	.60	1.0	.15	.20	.882	.464	.482
19	.60	1.0	.20	.10	.941	.572	.762
20	.60	1.0	.20	.15	.903	.512	.623
21	.60	1.0	.25	.10	.929	.572	.759
22	.60	1.5	0.00	.10	.988	.519	.734
23	.60	1.5	0.00	.15	.974	.460	.562
24	.60	1.5	0.00	.20	.953	.439	.393
25	.60	1.5	0.00	.25	.926	.451	.250
26	.60	1.5	0.00	.30	.894	.486	.144
27	.60	1.5	0.00	.35	.856	.535	.072
28	.60	1.5	.05	.10	.976	.516	.734
29	.60	1.5	.05	.15	.956	.450	.562
30	.60	1.5	.05	.20	.929	.416	.395
31	.60	1.5	.05	.25	.897	.416	.255
32	.60	1.5	.05	.30	.859	.441	.150

TABLE III (cont'd)

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One M $\frac{M}{M_p}$	Equation Two M $\frac{M}{M_p}$	Equation Three V $\frac{V}{V_p}$
33	.60	1.5	.10	.10	.965	.513	.732
34	.60	1.5	.10	.15	.938	.441	.563
35	.60	1.5	.10	.20	.906	.398	.402
36	.60	1.5	.10	.25	.868	.386	.269
37	.60	1.5	.15	.10	.953	.512	.730
38	.60	1.5	.15	.15	.921	.435	.566
39	.60	1.5	.15	.20	.882	.383	.416
40	.60	1.5	.20	.10	.941	.510	.728
41	.60	1.5	.20	.15	.903	.430	.572
42	.60	1.5	.25	.10	.929	.509	.726
43	.60	2.0	0.00	.10	.988	.471	.693
44	.60	2.0	0.00	.15	.974	.416	.498
45	.60	2.0	0.00	.20	.953	.410	.327
46	.60	2.0	0.00	.25	.926	.436	.199
47	.60	2.0	0.00	.30	.894	.480	.111
48	.60	2.0	.05	.10	.976	.464	.692
49	.60	2.0	.05	.15	.956	.398	.499
50	.60	2.0	.05	.20	.929	.378	.330
51	.60	2.0	.05	.25	.897	.393	.204
52	.60	2.0	.05	.30	.859	.429	.117
53	.60	2.0	.10	.10	.965	.459	.692
54	.60	2.0	.10	.15	.938	.383	.503
55	.60	2.0	.10	.20	.906	.350	.341
56	.60	2.0	.10	.25	.868	.354	.219
57	.60	2.0	.15	.10	.953	.455	.691
58	.60	2.0	.15	.15	.921	.371	.511
59	.60	2.0	.15	.20	.882	.327	.360
60	.60	2.0	.20	.10	.941	.452	.691
61	.60	2.0	.20	.15	.903	.361	.525
62	.60	2.0	.25	.10	.929	.450	.694

TABLE IV ULTIMATE STRENGTH VALUES Beams With Un-reinforced Rectangular Web Penetrations $\frac{A_f}{A_w} = .70$

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
1	.70	1.0	0.00	.10	.989	.620	.769
2	.70	1.0	0.00	.15	.976	.573	.627
3	.70	1.0	0.00	.20	.958	.544	.475
4	.70	1.0	0.00	.25	.934	.538	.327
5	.70	1.0	0.00	.30	.905	.555	.200
6	.70	1.0	0.00	.35	.871	.589	.104
7	.70	1.0	.05	.10	.979	.619	.768
8	.70	1.0	.05	.15	.961	.570	.627
9	.70	1.0	.05	.20	.937	.534	.476
10	.70	1.0	.05	.25	.908	.518	.330
11	.70	1.0	.05	.30	.874	.523	.206
12	.70	1.0	.10	.10	.968	.618	.767
13	.70	1.0	.10	.15	.945	.567	.625
14	.70	1.0	.10	.20	.916	.526	.477
15	.70	1.0	.10	.25	.882	.502	.339
16	.70	1.0	.15	.10	.958	.618	.765
17	.70	1.0	.15	.15	.929	.565	.624
18	.70	1.0	.15	.20	.895	.521	.482
19	.70	1.0	.20	.10	.947	.617	.762
20	.70	1.0	.20	.15	.913	.563	.623
21	.70	1.0	.25	.10	.937	.617	.759
22	.70	1.5	0.00	.10	.989	.570	.734
23	.70	1.5	0.00	.15	.976	.517	.562
24	.70	1.5	0.00	.20	.958	.498	.393
25	.70	1.5	0.00	.25	.934	.509	.250
26	.70	1.5	0.00	.30	.905	.540	.144
27	.70	1.5	0.00	.35	.871	.584	.072
28	.70	1.5	.05	.10	.979	.567	.734
29	.70	1.5	.05	.15	.961	.507	.562
30	.70	1.5	.05	.20	.937	.478	.395
31	.70	1.5	.05	.25	.908	.477	.255
32	.70	1.5	.05	.30	.874	.500	.150

TABLE IV (cont'd)

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
33	.70	1.5	.10	.10	.968	.565	.732
34	.70	1.5	.10	.15	.945	.500	.563
35	.70	1.5	.10	.20	.916	.461	.402
36	.70	1.5	.10	.25	.882	.450	.269
37	.70	1.5	.15	.10	.958	.563	.730
38	.70	1.5	.15	.15	.929	.494	.566
39	.70	1.5	.15	.20	.895	.448	.416
40	.70	1.5	.20	.10	.947	.562	.728
41	.70	1.5	.20	.15	.913	.490	.572
42	.70	1.5	.25	.10	.937	.561	.726
43	.70	2.0	0.00	.10	.989	.526	.693
44	.70	2.0	0.00	.15	.976	.478	.498
45	.70	2.0	0.00	.20	.958	.472	.327
46	.70	2.0	0.00	.25	.934	.495	.199
47	.70	2.0	0.00	.30	.905	.535	.111
48	.70	2.0	.05	.10	.979	.521	.692
49	.70	2.0	.05	.15	.961	.461	.499
50	.70	2.0	.05	.20	.937	.443	.330
51	.70	2.0	.05	.25	.908	.457	.204
52	.70	2.0	.05	.30	.874	.489	.117
53	.70	2.0	.10	.10	.968	.516	.692
54	.70	2.0	.10	.15	.945	.448	.503
55	.70	2.0	.10	.20	.916	.419	.341
56	.70	2.0	.10	.25	.882	.422	.219
57	.70	2.0	.15	.10	.958	.513	.691
58	.70	2.0	.15	.15	.929	.437	.511
59	.70	2.0	.15	.20	.895	.398	.360
60	.70	2.0	.20	.10	.947	.510	.691
61	.70	2.0	.20	.15	.913	.428	.525
62	.70	2.0	.25	.10	.937	.508	.694

TABLE V ULTIMATE STRENGTH VALUES Beams With Un-reinforced Rectangular Web Penetrations $\frac{A_f}{A_w} = .80$

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
1	.80	1.0	0.00	.10	.990	.656	.769
2	.80	1.0	0.00	.15	.979	.614	.627
3	.80	1.0	0.00	.20	.962	.588	.475
4	.80	1.0	0.00	.25	.940	.582	.327
5	.80	1.0	0.00	.30	.914	.597	.200
6	.80	1.0	0.00	.35	.883	.628	.104
7	.80	1.0	.05	.10	.981	.655	.768
8	.80	1.0	.05	.15	.964	.611	.627
9	.80	1.0	.05	.20	.943	.578	.476
10	.80	1.0	.05	.25	.917	.564	.330
11	.80	1.0	.05	.30	.886	.569	.206
12	.80	1.0	.10	.10	.971	.655	.767
13	.80	1.0	.10	.15	.950	.608	.625
14	.80	1.0	.10	.20	.924	.571	.477
15	.80	1.0	.10	.25	.893	.550	.339
16	.80	1.0	.15	.10	.962	.654	.765
17	.80	1.0	.15	.15	.936	.606	.624
18	.80	1.0	.15	.20	.905	.566	.482
19	.80	1.0	.20	.10	.952	.654	.762
20	.80	1.0	.20	.15	.921	.605	.623
21	.80	1.0	.25	.10	.943	.654	.759
22	.80	1.5	0.00	.10	.990	.611	.734
23	.80	1.5	0.00	.15	.979	.563	.562
24	.80	1.5	0.00	.20	.962	.546	.393
25	.80	1.5	0.00	.25	.940	.556	.250
26	.80	1.5	0.00	.30	.914	.584	.144
27	.80	1.5	0.00	.35	.883	.623	.072
28	.80	1.5	.05	.10	.981	.608	.734
29	.80	1.5	.05	.15	.964	.554	.562
30	.80	1.5	.05	.20	.943	.527	.395
31	.80	1.5	.05	.25	.917	.527	.255
32	.80	1.5	.05	.30	.886	.547	.150

TABLE V (cont'd)

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
33	.80	1.5	.10	.10	.971	.606	.732
34	.80	1.5	.10	.15	.950	.548	.563
35	.80	1.5	.10	.20	.924	.513	.402
36	.80	1.5	.10	.25	.893	.503	.269
37	.80	1.5	.15	.10	.962	.605	.730
38	.80	1.5	.15	.15	.936	.542	.566
39	.80	1.5	.15	.20	.905	.500	.416
40	.80	1.5	.20	.10	.952	.603	.728
41	.80	1.5	.20	.15	.921	.538	.572
42	.80	1.5	.25	.10	.943	.603	.726
43	.80	2.0	0.00	.10	.990	.571	.693
44	.80	2.0	0.00	.15	.979	.527	.498
45	.80	2.0	0.00	.20	.962	.522	.327
46	.80	2.0	0.00	.25	.940	.543	.199
47	.80	2.0	0.00	.30	.914	.579	.111
48	.80	2.0	.05	.10	.981	.566	.692
49	.80	2.0	.05	.15	.964	.513	.499
50	.80	2.0	.05	.20	.943	.496	.330
51	.80	2.0	.05	.25	.917	.508	.204
52	.80	2.0	.05	.30	.886	.538	.117
53	.80	2.0	.10	.10	.971	.562	.692
54	.80	2.0	.10	.15	.950	.500	.503
55	.80	2.0	.10	.20	.924	.474	.341
56	.80	2.0	.10	.25	.893	.477	.219
57	.80	2.0	.15	.10	.962	.559	.691
58	.80	2.0	.15	.15	.936	.491	.511
59	.80	2.0	.15	.20	.905	.455	.360
60	.80	2.0	.20	.10	.952	.557	.691
61	.80	2.0	.20	.15	.921	.483	.525
62	.80	2.0	.25	.10	.943	.555	.694

TABLE VI ULTIMATE STRENGTH VALUES Beams With Un-reinforced Rectangular Web Penetrations $\frac{A_f}{A_w} = .90$

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
1	.90	1.0	0.00	.10	.991	.686	.769
2	.90	1.0	0.00	.15	.980	.648	.627
3	.90	1.0	0.00	.20	.965	.623	.475
4	.90	1.0	0.00	.25	.946	.618	.327
5	.90	1.0	0.00	.30	.922	.632	.200
6	.90	1.0	0.00	.35	.893	.660	.104
7	.90	1.0	.05	.10	.983	.685	.768
8	.90	1.0	.05	.15	.967	.644	.627
9	.90	1.0	.05	.20	.948	.615	.476
10	.90	1.0	.05	.25	.924	.602	.330
11	.90	1.0	.05	.30	.896	.606	.206
12	.90	1.0	.10	.10	.974	.685	.767
13	.90	1.0	.10	.15	.954	.642	.625
14	.90	1.0	.10	.20	.930	.609	.477
15	.90	1.0	.10	.25	.902	.589	.339
16	.90	1.0	.15	.10	.965	.684	.765
17	.90	1.0	.15	.15	.941	.640	.624
18	.90	1.0	.15	.20	.913	.604	.482
19	.90	1.0	.20	.10	.957	.684	.762
20	.90	1.0	.20	.15	.928	.639	.623
21	.90	1.0	.25	.10	.948	.684	.759
22	.90	1.5	0.00	.10	.991	.644	.734
23	.90	1.5	0.00	.15	.980	.601	.562
24	.90	1.5	0.00	.20	.965	.585	.393
25	.90	1.5	0.00	.25	.946	.594	.250
26	.90	1.5	0.00	.30	.922	.620	.144
27	.90	1.5	0.00	.35	.893	.656	.072
28	.90	1.5	.05	.10	.983	.642	.734
29	.90	1.5	.05	.15	.967	.593	.562
30	.90	1.5	.05	.20	.948	.569	.395
31	.90	1.5	.05	.25	.924	.568	.255
32	.90	1.5	.05	.30	.896	.587	.150

TABLE VI (cont'd)

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
33	.90	1.5	.10	.10	.974	.640	.732
34	.90	1.5	.10	.15	.954	.587	.563
35	.90	1.5	.10	.20	.930	.555	.402
36	.90	1.5	.10	.25	.902	.546	.269
37	.90	1.5	.15	.10	.965	.639	.730
38	.90	1.5	.15	.15	.941	.582	.566
39	.90	1.5	.15	.20	.913	.544	.416
40	.90	1.5	.20	.10	.957	.638	.728
41	.90	1.5	.20	.15	.928	.578	.572
42	.90	1.5	.25	.10	.948	.637	.726
43	.90	2.0	0.00	.10	.991	.609	.693
44	.90	2.0	0.00	.15	.980	.569	.498
45	.90	2.0	0.00	.20	.965	.564	.327
46	.90	2.0	0.00	.25	.946	.583	.199
47	.90	2.0	0.00	.30	.922	.616	.111
48	.90	2.0	.05	.10	.983	.604	.692
49	.90	2.0	.05	.15	.967	.555	.499
50	.90	2.0	.05	.20	.948	.540	.330
51	.90	2.0	.05	.25	.924	.551	.204
52	.90	2.0	.05	.30	.896	.578	.117
53	.90	2.0	.10	.10	.974	.600	.692
54	.90	2.0	.10	.15	.954	.544	.503
55	.90	2.0	.10	.20	.930	.520	.341
56	.90	2.0	.10	.25	.902	.522	.219
57	.90	2.0	.15	.10	.965	.597	.691
58	.90	2.0	.15	.15	.941	.535	.511
59	.90	2.0	.15	.20	.913	.502	.360
60	.90	2.0	.20	.10	.957	.595	.691
61	.90	2.0	.20	.15	.928	.528	.525
62	.90	2.0	.25	.10	.948	.593	.694

TABLE VII ULTIMATE STRENGTH VALUES Beams With Un-reinforced Rectangular Web Penetrations $\frac{A_f}{A_w} = 1.0$

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
1	1.0	1.0	0.00	.10	.992	.711	.769
2	1.0	1.0	0.00	.15	.982	.676	.627
3	1.0	1.0	0.00	.20	.968	.654	.475
4	1.0	1.0	0.00	.25	.950	.649	.327
5	1.0	1.0	0.00	.30	.928	.661	.200
6	1.0	1.0	0.00	.35	.902	.687	.104
7	1.0	1.0	.05	.10	.984	.711	.768
8	1.0	1.0	.05	.15	.970	.673	.627
9	1.0	1.0	.05	.20	.952	.646	.476
10	1.0	1.0	.05	.25	.930	.634	.330
11	1.0	1.0	.05	.30	.904	.638	.206
12	1.0	1.0	.10	.10	.976	.710	.767
13	1.0	1.0	.10	.15	.958	.671	.625
14	1.0	1.0	.10	.20	.936	.640	.477
15	1.0	1.0	.10	.25	.910	.622	.339
16	1.0	1.0	.15	.10	.968	.710	.765
17	1.0	1.0	.15	.15	.946	.669	.624
18	1.0	1.0	.15	.20	.920	.636	.482
19	1.0	1.0	.20	.10	.960	.709	.762
20	1.0	1.0	.20	.15	.934	.668	.623
21	1.0	1.0	.25	.10	.952	.709	.759
22	1.0	1.5	0.00	.10	.992	.673	.734
23	1.0	1.5	0.00	.15	.982	.633	.562
24	1.0	1.5	0.00	.20	.968	.619	.393
25	1.0	1.5	0.00	.25	.950	.627	.250
26	1.0	1.5	0.00	.30	.928	.651	.144
27	1.0	1.5	0.00	.35	.902	.684	.072
28	1.0	1.5	.05	.10	.984	.671	.734
29	1.0	1.5	.05	.15	.970	.626	.562
30	1.0	1.5	.05	.20	.952	.603	.395
31	1.0	1.5	.05	.25	.930	.603	.255
32	1.0	1.5	.05	.30	.904	.620	.150

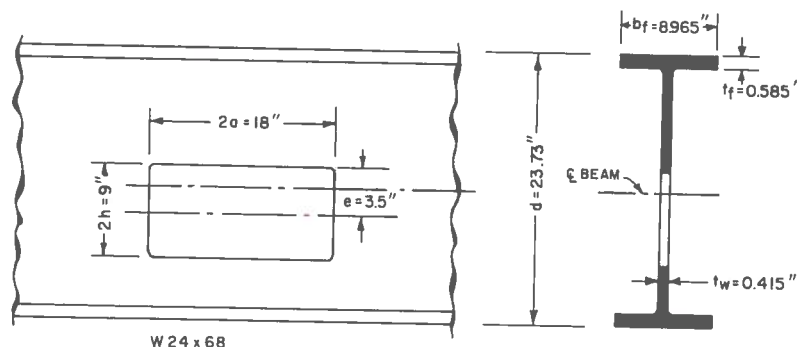
TABLE VII (cont'd)

LINE	$\frac{A_f}{A_w}$	$\frac{a}{h}$	$\frac{e}{d}$	$\frac{h}{d}$	Equation One $\frac{M}{M_p}$	Equation Two $\frac{M}{M_p}$	Equation Three $\frac{V}{V_p}$
33	1.0	1.5	.10	.10	.976	.669	.732
34	1.0	1.5	.10	.15	.958	.620	.563
35	1.0	1.5	.10	.20	.936	.591	.402
36	1.0	1.5	.10	.25	.910	.582	.269
37	1.0	1.5	.15	.10	.968	.668	.730
38	1.0	1.5	.15	.15	.946	.616	.566
39	1.0	1.5	.15	.20	.920	.580	.416
40	1.0	1.5	.20	.10	.960	.667	.728
41	1.0	1.5	.20	.15	.934	.612	.572
42	1.0	1.5	.25	.10	.952	.666	.726
43	1.0	2.0	0.00	.10	.992	.640	.693
44	1.0	2.0	0.00	.15	.982	.603	.498
45	1.0	2.0	0.00	.20	.968	.599	.327
46	1.0	2.0	0.00	.25	.950	.616	.199
47	1.0	2.0	0.00	.30	.928	.646	.111
48	1.0	2.0	.05	.10	.984	.636	.692
49	1.0	2.0	.05	.15	.970	.591	.499
50	1.0	2.0	.05	.20	.952	.577	.330
51	1.0	2.0	.05	.25	.930	.587	.204
52	1.0	2.0	.05	.30	.904	.612	.117
53	1.0	2.0	.10	.10	.976	.632	.692
54	1.0	2.0	.10	.15	.958	.580	.503
55	1.0	2.0	.10	.20	.936	.558	.341
56	1.0	2.0	.10	.25	.910	.561	.219
57	1.0	2.0	.15	.10	.968	.630	.691
58	1.0	2.0	.15	.15	.946	.572	.511
59	1.0	2.0	.15	.20	.920	.542	.360
60	1.0	2.0	.20	.10	.960	.628	.691
61	1.0	2.0	.20	.15	.934	.566	.525
62	1.0	2.0	.25	.10	.952	.626	.694

Examples

Example 1. Beam size: W24 x 68
Material: ASTM A36

This beam is simply supported at the ends of a 36-ft span. It supports concentrated loads of 23 kips at the third points. A rectangular penetration of the beam web is needed. This hole is to be 9 inches high, 18 inches long, and its centerline is to be 3.5 inches below that of the beam. Applicable specifications are those of the American Institute of Steel Construction (AISC).



Questions:

- Can an un-reinforced hole of this size be permitted in this beam shape?
- Can the hole be located between the loads?
- Where can the hole be located?

Hole properties:

$$a = \frac{18}{2} = 9"$$

$$h = \frac{9}{2} = 4.5"$$

$$e = 3.5" \text{ (always positive)}$$

$$\frac{a}{h} = \frac{9}{4.5} = 2.0$$

$$\frac{e}{d} = \frac{3.5}{23.73} = 0.147$$

Beam properties:

$$d = 23.73"$$

$$b_f = 8.965"$$

$$t_f = 0.585"$$

$$A_f = b_f t_f = 8.965 \times 0.585 = 5.24 \text{ sq inches}$$

$$t_w = 0.415"$$

$$A_w = d t_w = 23.73 \times 0.415 = 9.85 \text{ sq inches}$$

$$Z = 177 \text{ inches}^3$$

$$M_p = Z F_y = 177 \times 36 = 6372 \text{ inch-kips} \times \frac{1}{12} = 531 \text{ ft-kips}$$

$$V_p = 0.55 F_y t_w d = 0.55 \times 36 \times 0.415 \times 23.73 = 195 \text{ kips}$$

Stability checks:

Flange

$$\frac{b_f}{2t_f} = 7.7$$

$$\frac{65}{\sqrt{F_y}} = \frac{65}{\sqrt{36}} = 10.83$$

$$7.7 < 10.83 \text{ OK}$$

Compression tee

$$\text{Hole length} = 2a = 2 \times 9 = 18"$$

Depth of top tee

$$\begin{aligned} \text{(subject to compression)} &= \frac{d}{2} - h + e \\ &= \frac{23.73}{2} - 4.5 + 3.5 \\ &= 10.87 \end{aligned}$$

$$18 < 4 \times 10.87 \text{ OK}$$

Web

$$\begin{aligned} \frac{a}{h} + \frac{12h}{d} &= \frac{9}{4.5} + \frac{12 \times 4.5}{23.73} = 4.28 \\ 4.28 &< 5.6 \text{ OK} \end{aligned}$$

$$\frac{d - 2t_f}{t_w} = \frac{23.73 - (2 \times 0.585)}{0.415} = 54$$

$$\frac{420}{\sqrt{F_y}} = \frac{420}{\sqrt{36}} = 70$$

$$54 < 70 \text{ OK}$$

∴ Web qualifies as "stocky."

$$\frac{a}{h} = \frac{9}{4.5} = 2 < 3.0 \quad \text{OK}$$

$$V_{\max} \leq 0.667 V_p = 0.667 \times 195 = 130 \text{ kips}$$

(This shear will be checked against the shear at the hole after its location has been determined. All other stability checks are OK.)

Interaction chart parameters:

$$\frac{A_f}{A_w} = \frac{5.24}{9.85} = 0.53, \text{ round down: use } 0.5$$

$$\frac{a}{h} = \frac{9}{4.5} = 2.0: \text{ use } 2.0$$

$$\frac{e}{d} = \frac{3.5}{23.73} = 0.147: \text{ close enough to use } 0.15$$

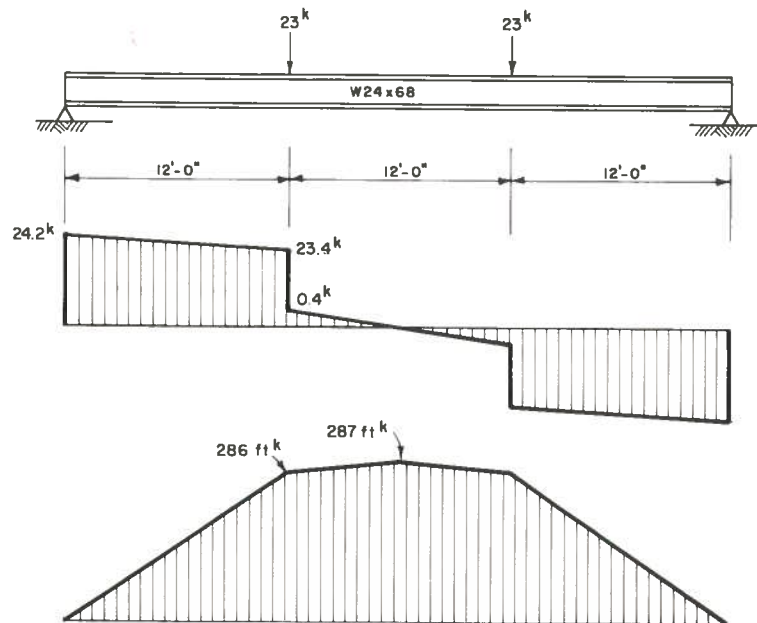
$$\frac{h}{d} = \frac{4.5}{23.73} = 0.19: \text{ interpolate}$$

Line 59 of Table II shows $A_f/A_w = 0.5$, $a/h = 2.0$, $e/d = 0.15$, $h/d = .20$, and gives $(M/M_p)_1 = 0.867$, $(M/M_p)_2 = 0.237$, and $(V/V_p) = 0.360$. Interpolation can be made between these values and those shown on line 58 (where $h/d = 0.15$). This interpolation will give the following values:

$$\begin{aligned} (M/M_p)_1 &= 0.876 \\ (M/M_p)_2 &= 0.247 \\ (V/V_p) &= 0.390 \end{aligned}$$

These values were used to plot the interaction diagram shown in Diagram A.

An *un-reinforced* hole is permissible in this beam shape if a combination of moment and shear defining a point under the interaction line can be found.



Note that all of the work done thus far is independent of the load, and serves only to determine ultimate capacities.

Shear and moment diagrams are constructed in the usual manner.

Since these loads, shears, and moments are the result of actual dead and live loads, they must be multiplied by a load-factor of 1.7 when being compared to ultimate capacities.

In the center portion between the loads

$$\frac{M}{M_p} = \frac{287 \times 1.7}{531} = 0.92$$

$$\frac{V}{V_p} = \frac{0.4 \times 1.7}{195} = 0.003$$

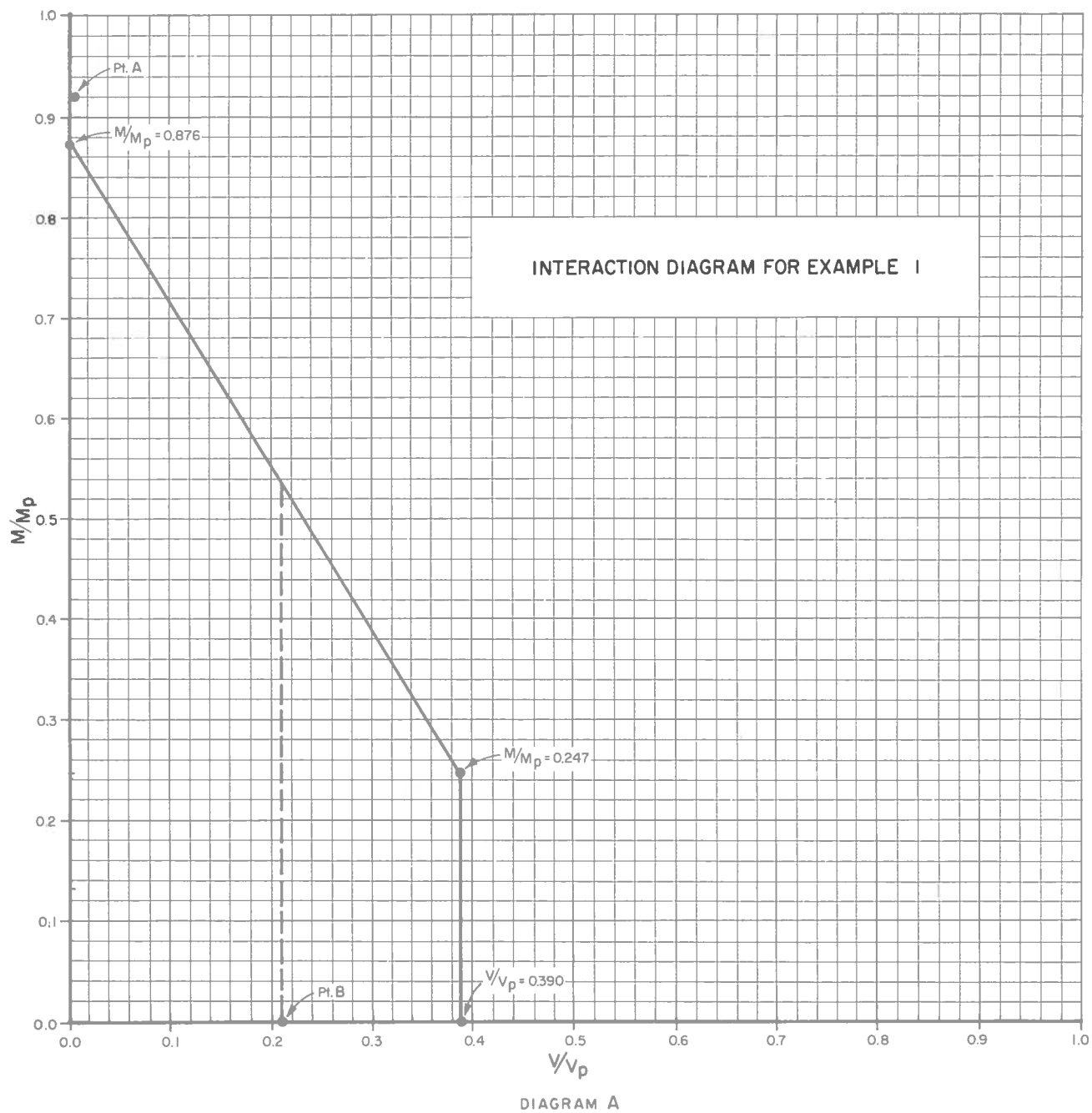
This plots as point "A" which is outside the safe zone and is therefore NO GOOD. Hence, the hole *cannot* be located between the loads.

If the hole is located near the left end

$$\frac{M}{M_p} = 0$$

$$\frac{V}{V_p} = \frac{24.2 \times 1.7}{195} = 0.21$$

This plots as point "B" which is in the safe zone and is therefore OK. (Also note that factored shear (41.1^k) is less than V_{\max} for web stability (130^k .) Since the shear is almost constant (in actuality it decreases slightly) between the end support and the concentrated load, a vertical line through point "B" on the interaction diagram can be drawn to represent conditions of combined moment and shear in this portion of the span. The intersection of this vertical line with the interaction line defines the limiting $M/M_p = 0.54$.

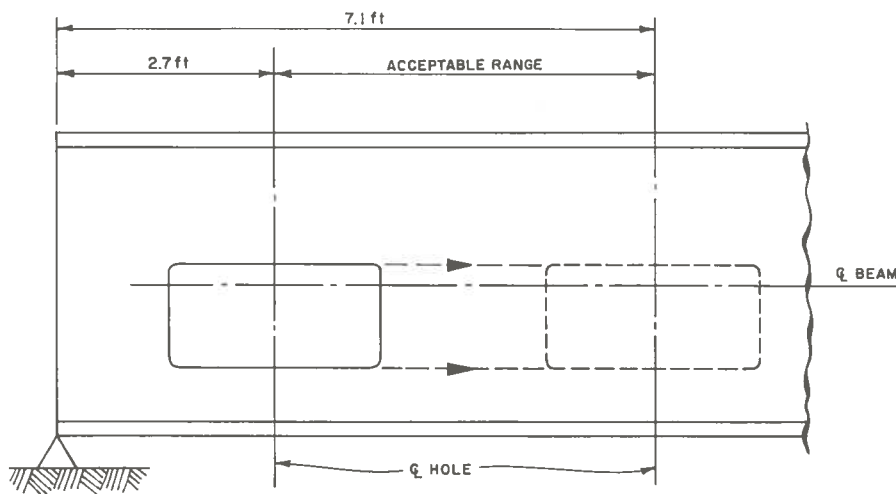


Max. allowable factored moment =
 $0.54 \times 531 = 287 \text{ ft-kips}$

Max. allowable design moment = $\frac{287}{1.7}$
 $= 169 \text{ ft-kips}$

Since the moment diagram lines are almost straight, the distance at which $M = 169 \text{ ft-kips}$ can be found as $\frac{169}{286} \times 12 = 7.1 \text{ ft}$.

The edge of the hole should be kept approximately a distance d from the support so that the hole's centerline will be from 2.7 ft to 7.1 ft from either end of the beam.



Example 2

As in Example 1, a W24x68 beam of A36 steel is to be penetrated by a rectangular opening 9 inches high and 18 inches wide. Again, the horizontal centerline of the hole is to lie 3.5 inches below the beam's centerline. In this case, however, the beam span is 27 ft and must be designed for lateral wind loads plus gravity loads of 48 kips concentrated at the third points, and the beam's weight. The lateral wind forces produce moments of 120 kip-ft at each end of the beam. AISC Specifications apply.

Questions:

- Can an un-reinforced hole of this size be permitted in this beam shape?
- If so, where can this hole be located?

The calculation procedure here is identical to that of Example 1 for *hole properties*, *beam properties*, *stability checks*, and *interaction diagram parameters*. The bottom tee section has a depth of

$$\frac{d}{2} - h - e = \frac{23.73}{2} - 4.5 - 3.5 = 3.87$$

The hole is more than four times longer than this depth. If a hole is to be located in the negative moment region, the compression stress on this tee section must be investigated.

All other numerical results are the same as in Example 1, including the interaction diagram (Diagram B). Once again, an un-reinforced hole in the beam is permissible provided that it is possible to find a combination of moment and shear that defines a point under the interaction line.

Shear and moment diagrams are constructed for gravity and lateral loads as shown below.

The beam must have sufficient strength to support a factored load equal to 1.7 times the gravity load. Moreover, it must also be capable of sustaining a factored load equal to 1.3 times the combination of gravity and lateral load.

Since all lines in the shear and moment diagrams are straight (or nearly so), investigations will be made as follows:

Point 1, at left support:

$$\begin{aligned} \text{Gravity shear, } V_G &= 48.9\text{k} \\ &\times 1.7 = 83.1\text{k} \\ &\times 1.3 = 63.6\text{k} \end{aligned}$$

$$\begin{aligned} \text{Gravity moment, } M_G &= 292\text{ ft k} \\ &\times 1.7 = 496\text{ ft k} \\ &\times 1.3 = 380\text{ ft k} \end{aligned}$$

$$\begin{aligned} \text{Wind shear, } V_W &= 8.9\text{k} \\ &\times 1.3 = 11.6\text{k} \end{aligned}$$

$$\begin{aligned} \text{Wind moment, } M_W &= 120\text{ ft k} \\ &\times 1.3 = 156\text{ ft k} \end{aligned}$$

Gravity only:

$$\frac{V}{V_p} = \frac{83.1}{195} = 0.43, \quad \frac{M}{M_p} = \frac{496}{531} = 0.93$$

This plots as point G1 and is unacceptable.

Gravity plus wind force:

$$\begin{aligned} \frac{V}{V_p} &= \frac{63.6 + 11.6}{195} = 0.39 \\ \frac{M}{M_p} &= \frac{380 + 156}{531} = 1.00 \end{aligned}$$

This plots as point GW1 and is also unacceptable.

It is, of course, not surprising that there is no reserve capability at the supports; shear and moment are both maximum. Additionally, having found this hole location unacceptable for one loading case, calculations for the second loading case are unnecessary since the location must be acceptable for *all* loading cases if the hole is to be permitted.

Nonetheless, considerable insight is gained from this calculation and plot. Clearly, gravity shear between end supports and concentrated loads is far too large to make V/V_p much less than 0.39, and will cause all points in these regions to plot either outside the safe zone of the interaction diagram or so close to its edge as to be questionable. Therefore, the center portion of the beam, just inside the concentrated loads, should be investigated.

$$\begin{aligned} V_G &= 0.3\text{k} \\ &\times 1.7 = .5\text{k} \\ &\times 1.3 = .4\text{k} \end{aligned}$$

$$\begin{aligned} M_G &= 145\text{ ft k} \\ &\times 1.7 = 247\text{ ft k} \\ &\times 1.3 = 189\text{ ft k} \end{aligned}$$

$$\begin{aligned} V_W &= 8.9\text{k} \\ &\times 1.3 = 11.6\text{k} \end{aligned}$$

$$\begin{aligned} M_W &= 40.0\text{ ft k} \\ &\times 1.3 = 52\text{ ft k} \end{aligned}$$

Gravity only:

$$\frac{V}{V_p} = \frac{.5}{195} = 0.003, \quad \frac{M}{M_p} = \frac{247}{531} = 0.47$$

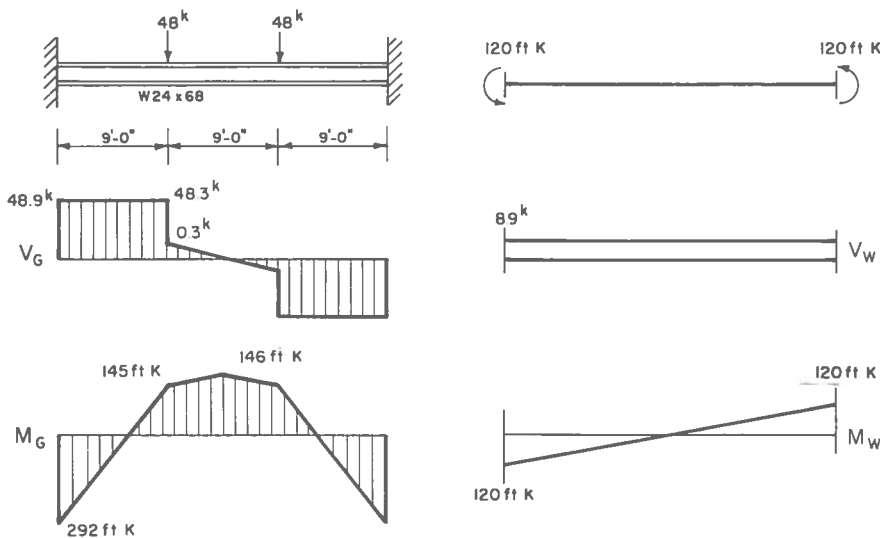
This plots as point G2 and is acceptable.

Gravity plus lateral force:

$$\begin{aligned} \frac{V}{V_p} &= \frac{.4 + 11.6}{195} = 0.06 \\ \frac{M}{M_p} &= \frac{189 + 52}{531} = 0.45 \end{aligned}$$

This plots as point GW2 and is acceptable.

By inspection, it can be seen that all other points in the beam's center region are acceptable hole locations. However, since the edge of the hole should be kept approximately d from the concentrated load, the hole's centerline should fall within the middle 42 inches of the beam.



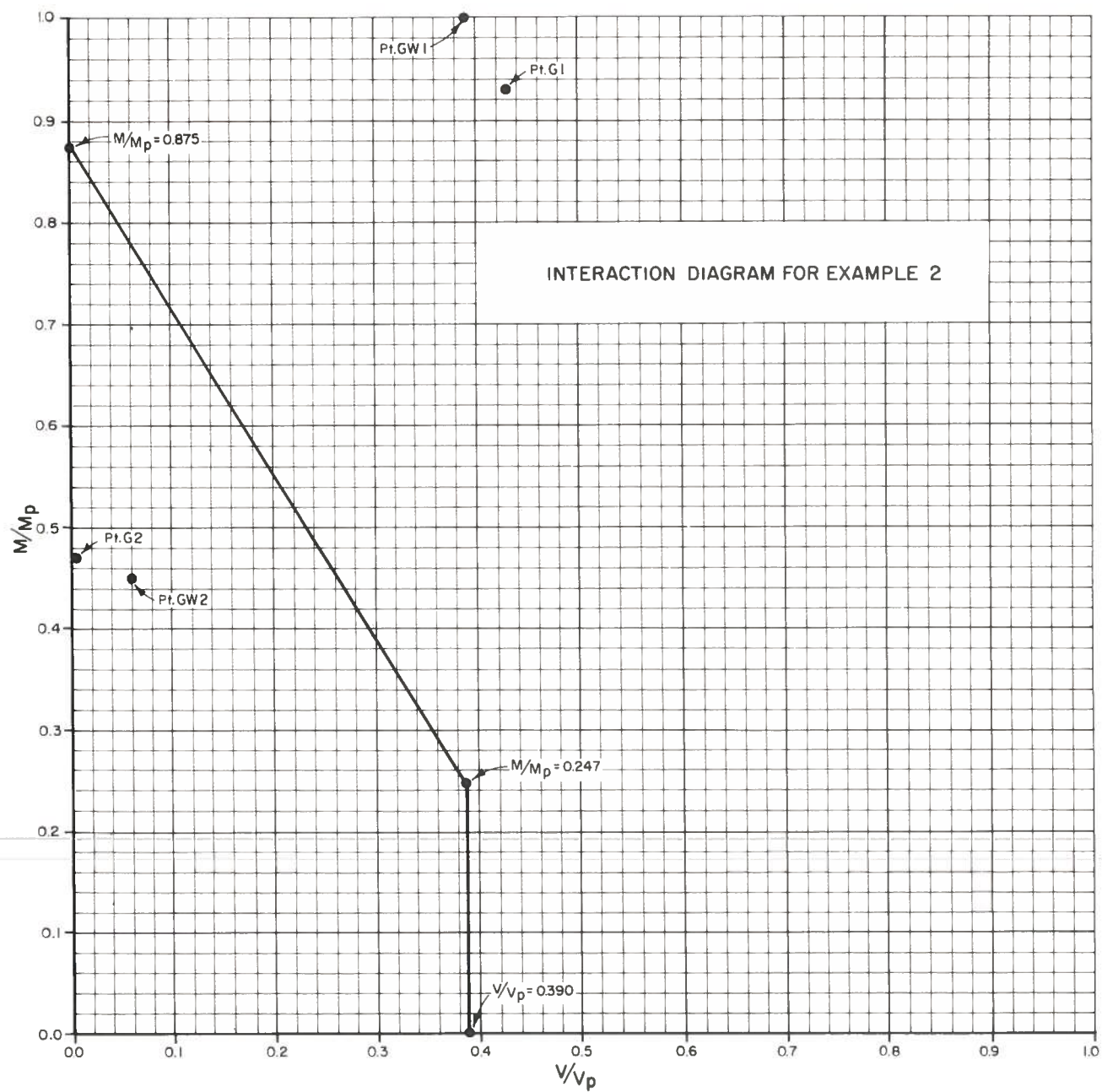


DIAGRAM B

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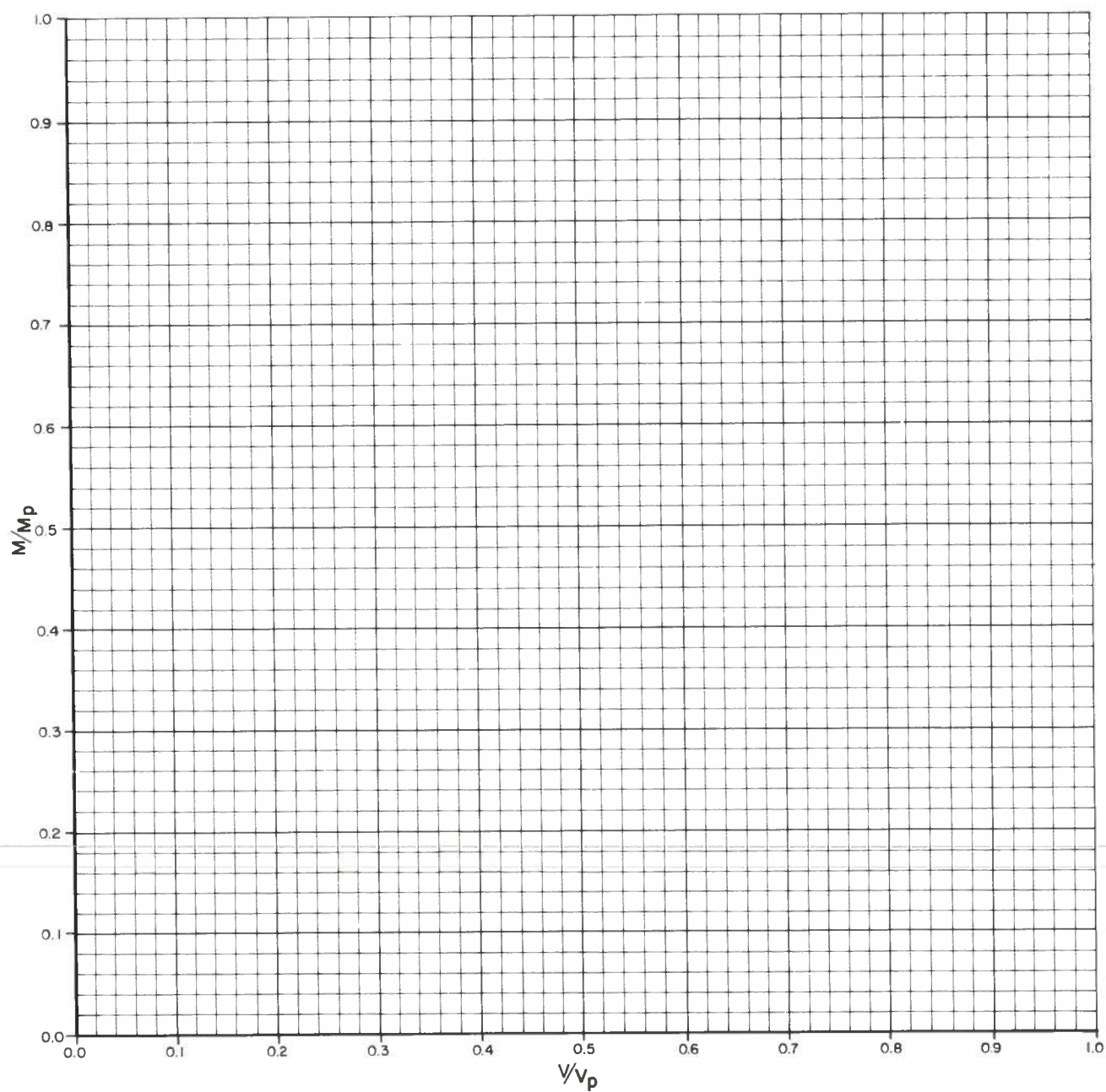
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