Design of Lightly Loaded Steel Column Base Plates

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One of the least-studied structural elements are column base plates. Even less attention has been paid to lightly loaded base plates, defined here as relatively flexible plates of approximately the same size as the outside dimensions of the column supported by the plate. Such base plates are found in low-rise conventional construction and in pre-engineered metal building applications. In this class of construction, relatively small dead loads and low slope roofs commonly result in uplift loads which must be transferred to the foundation through the base plates. At present, there is no generally accepted practice for the design of such base plates that is not unduly conservative.

From results of yield-line analyses and limited supporting experimental data, design procedures for lightly loaded base plates supporting H-shaped columns are proposed. Both concentric axial compression (gravity loading) and concentric axial tension (uplift loading) are considered. Base plate strength for erection safety is also discussed.

GRAVITY LOADING

Background—Technical literature concerned with the compressive strength of steel column base plates may be treated broadly in two categories: (*1*) the bearing strength of concrete, and (*2*) the study of various rigid and flexible plates on elastic foundations.[1] The bearing strength of concrete and rock was investigated as early as 1876 by Bauschinger, presented later by Withey and Aston.[2] Bauschinger was concerned about the ability of various classes of rock and concrete to support steel bearing piles; his work with concrete was for him less significant. Meyerhof[3] found the bearing strength of concrete increases in direct proportion to the ratio of concrete thickness to footing (plate) width when splitting is present. He demonstrated that when the mass of the concrete is confined to inhibit splitting, the capacity is in agreement with the theory of bearing capacity presented by Terzaghi. Shelson[4] studied experimentally the behavior of base plates when the ratio of the concrete surface area is large compared to that of the plate bearing area. Apparently, the American Concrete Institute's *Building Code Requirements for Reinforced Concrete*[5] were based on Shelson's work.

Hawkins[6] reported results of 230 tests using rigid plates. He determined that concrete bearing strength is a function of the plate dimensions and concrete cylinder strength, and showed that Shelson's work is conservative. The ACI Code was subsequently liberalized,[7] with an increase in allowable stress from 40–87%, depending on the concrete area to plate area ratio. The current AISC *Specification*[8] is based on ACI Code requirements, but in allowable stress design format.

More recently DeWolf[9] presented the results of 19 tests with plates placed on unreinforced concrete cubes and loaded only in the central portion. He has shown the current AISC *Specification* requirements[8] to be conservative. However, he limits his results to base plates which extend beyond the column perimeter.

Elastic rectangular plates on elastic foundations have been extensively studied by many authors (Refs. 10–16). However, specific applications to column base plate design are rare. The design procedure found in the 7th and earlier editions of the AISC *Manual of Steel Construction*[17] for determining plate thickness if the base plate extends beyond the column perimeter is based on the conservative allowable bearing stress permitted by the ACI code at that time and by the assumption of uniform pressure under the base plate.

Fling[18] has proposed that in addition to the strength requirements suggested by AISC, a relative upward deflection limitation of 0.01 in. be imposed. He also presents yield-line and elastic plate bending solutions for determining plate thickness of lightly loaded base plates. A deflection limitation is suggested, with deflection calculated as the maximum free edge deflection of an elastic plate which is fixed on the opposite edge and supported on the other two edges. The elastic strength solution is based on one point at the maximum stress, which occurs at the

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middle of the fixed edge, set equal to the yield stress of the steel. Fling concedes his requirements are conservative because of the assumptions involved.

The elastic plate bending solution has been adopted by AISC[19] as one criterion for determining column base plate thickness. It is noted that all of the above solutions assume the base plate remains in contact with the concrete substrate during loading, and the resulting pressure is uniform over the plate. Stockwell[20] presented a design method assuming only an H-shaped area under the column flanges and web to be effective in bearing. His reasoning is based on recognition of the fact that uniform bearing pressure is unrealistic and that maximum pressure would logically follow the profile shape. Resultant stress "redistribution" provides reserve capacity as larger areas of the base plate become effective. Details of Stockwell's proposed method will be discussed subsequently.

The analyses made of the bearing strength of concrete are primarily based on observations of concrete specimens, while the analyses made of the behavior of steel plates on elastic substrates are primarily theoretical in nature dealing with various modelings but without experimental evidence. Of the authors discussed, only Shelson,[4] DeWolf,[9] Fling,[18] and Stockwell[20] treat the interaction of a steel plate and concrete substrate. Only Fling and Stockwell treat the topic of column and base plate of like dimensions. Finally, only Stockwell considers the possibility of uplift at the free edge of a base plate.

Finite Element Study—The finite element analysis method permits a relatively easy way of analyzing a base plate, particularly if the plate is subjected to uplift at the boundaries. If the plate is modeled using a bending element supported by elastic springs representing the substrate, the springs can simply be released when uplift occurs. Although the method is iterative, results can be obtained fairly quickly.

To study the elastic behavior of lightly loaded base plates, the finite element capabilities of the widely accepted computer program STRUDL[21] were utilized. Because of double symmetry, only one-quarter of a base plate was analyzed, with support releases used to ensure zero slope at the plate centerlines. Each node was supported by a linear spring in the Z-direction. The plate elements under the web and flange of the column were assigned a greater flexural rigidity than the remaining plate elements by using a modulus of elasticity one-thousand times greater. The STRUDL CPT plate bending element was used exclusively. In each analysis, the plates were first analyzed assuming no uplift; if uplift occurred, the uplifting nodes were released from the substrate and the plate reanalyzed. After several analysis cycles, all uplifting nodes were correctly released. For each plate, two analyses were performed using both a coarse mesh and a fine mesh to ensure convergence.

Figure 1a shows a typical fine mesh used in the study. For the particular mesh shown, the base plate is 8 in. by 12 in. by 0.375 in. thick. Column flanges are 8 in. by 0.25 in. and the web thickness is 0.25 in. A substrate modulus of 355.4 kips/in.^{2}/in, as determined by experiment, was used in the analysis. Results of the finite element analyses are shown (along with experimental data to be discussed later) in Fig. 1b. The area of the uplifting nodes is shaded for clarity and the deflection values shown have been normalized with respect to the nodes at the tip of the column flange. Approximately 28% of the contact area uplifted.

Experimental Study—Two gravity load tests were conducted to verify the finite element analysis results. The test specimens consisted of a steel column and base plate section, a reinforced concrete pedestal base, and a layer of expansive grout placed between the steel and concrete to provide a uniform bearing surface. Grout was used to obtain better corroboration with the finite element model, but is not considered necessary for lightly loaded column base plates.

The steel column and base plate sections were fabricated from A572 Gr. 50 steel. Specimen BP1 had a 6 in. by 8 in. by $\frac{3}{8}$ in. base plate and specimen BP2 a 8 in. by 12 in. by $\frac{3}{8}$ in. base plate. Table 1 lists the measured plate thicknesses for each specimen.

The reinforced concrete pedestals were cast to a size to provide a 3 in. wide edge distance and the depth was 18 in. Reinforcement consisted of four No. 4 vertical reinforcing bars and three equally spaced No. 3 ties. Two $\frac{3}{4}$-in. diameter A307 anchor bolts were cast in the specimens. Test cylinder data for concrete and grout are given in Table 2.

The assembled specimens were placed in a 200-kip capacity universal type testing machine. Six dial gages were suspended between the column flanges above the base plate on one side of the specimens. The gages were at the quarter points along the length of the plate in two rows, one row at the free edge and one row midway between the web and the free edge. These gages measured the relative upward displacement of the base plate with respect to a horizontal plane through the column approximately 6 in. above the base plate. A seventh dial gage was used to measure the relative penetration of one column flange into the grout and concrete body. Sixteen electrical resistance strain gages were attached to one quarter of the base plate upper surface of specimen BP2. Dial and strain gage placement is shown in Fig. 2.

After an initial load/unload cycle, loading proceeded in increments until failure, with gage readings recorded at each increment. Specimen BP1 failed by fracture of the grout at the perimeter of the plate along the flanges of the column section at a load of 187 kips. Specimen BP2 had a capacity exceeding that of the testing machine.

Maximum base plate deflections for either specimen did not exceed 0.05 in. Normalized measured deflections at maximum load for Specimen BP2 are shown in Fig. 1b. Reasonable agreement is found between measured and predicted deflections and the uplift area is verified. Results for Specimen BP1 are similar. Stresses calculated from

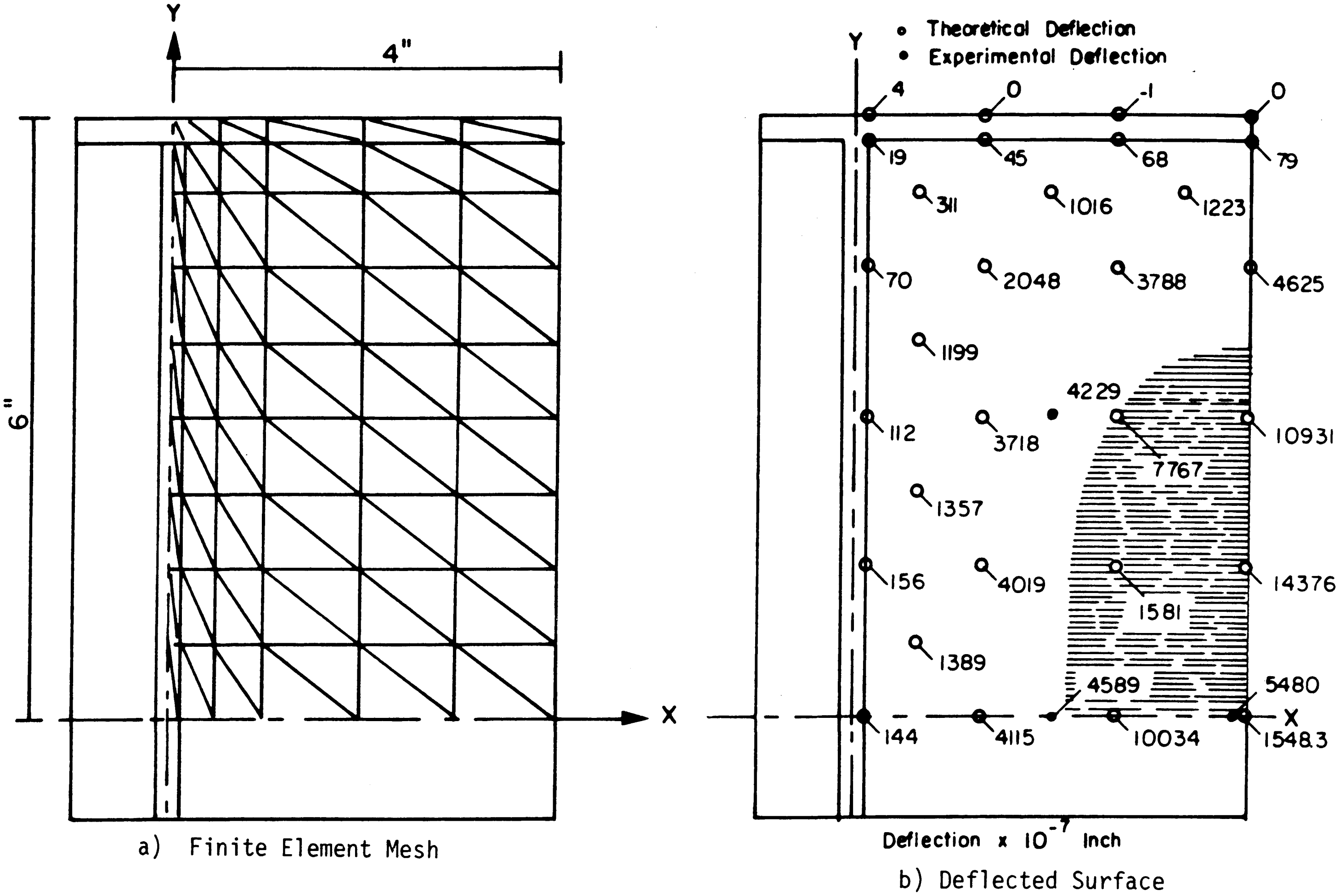

Fig. 1. Gravity loading analytical and experimental results

measured strains and stresses predicted by the finite element analysis were found to be quite small, less than 15 ksi, and are not considered sighificant.

An additional test was made of the pedestal/grout assembly to determine the composite stiffness. A rigid steel plate two inches square was placed on the grout bed and loaded in the universal testing machine to determine values for elastic stiffness and onset of inelastic strains. The results are shown in Fig. 3. The elastic stiffness was found to be 355.4 kips/in^2/in and the onset of inelastic strains at approximately 12 ksi.

Table 1. Gravity Loading Specimen Plate Thicknesses

Specimen	Flange	Web	Base Plate t (in.) Nominal	Base Plate t (in.) Measured
BP1	0.250	0.1799	0.375	0.367
BP2	0.250	0.250	0.375	0.375

Application to Design—The analyses proposed by Fling[18] seek to satisfy both strength and stiffness criteria. The stiffness criterion limits the relative deflection of the free edge to insure adequate load distribution beneath the plate to avoid overstressing the concrete substrate. However, when the concrete mass is large enough to inhibit splitting, the necessity of preventing overstress in the concrete substrate is eliminated as the failure mode is not due to brittle

Table 2. Concrete and Grout Cylinder Test Results

Cylinder	Location	7 Day Tested Strength (psi)
1	BP1	1379
2	BP1	1655
3	BP2	3798
4	BP2	3742
5	Grout	4725

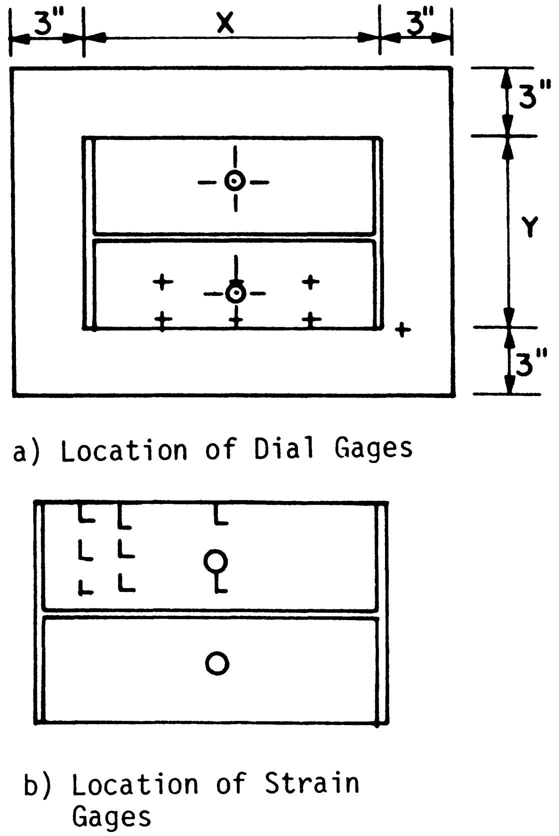

Fig. 2. Dial- and strain-gage placement for gravity loading tests

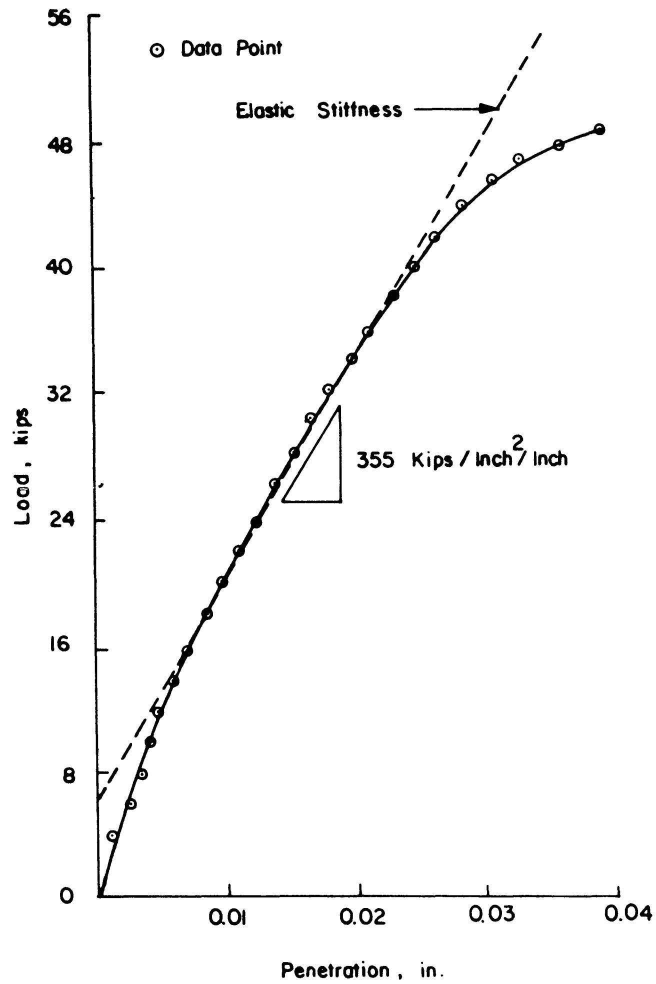

Fig. 3. Steel plate grout penetration

fracture of the concrete base. The experimental evidence does not support the need of a stiffness requirement. Rather the overstress of the substrate encountered with relatively thin plates is highly localized and results in redistribution of bearing stress as localized inelastic strains develop. This indicates that a design method need only recognize the ultimate behavior of the base plate (failure mode) and the stiffness criterion can be neglected.

From experimental and analytical results, thin base plates lift off the substrate during loading. Thus, the assumption of uniform stress distribution at the interface is invalid. Further, consideration of only elastic plate strength greatly underestimates the capacity of thin bearing plates. Unfortunately, the current AISC recommended design procedure[19] uses both assumptions, resulting in unnecessarily thick column base plates. Stockwell[20] has recognized both inadequacies and has recommended the plate be approximated as an H-shaped area under the column as shown in Fig. 4, or in effect cantilevers perpendicular to the column flanges and web. The maximum allowable bearing stress for the substrate is assumed to be uniformly distributed over the H-shaped area, from which the dimensions of the area can be determined. Constant cantilever length for both the flange and web portions is assumed as shown in Fig. 4.

From ACI 318-77,[7] the allowable bearing stress of concrete pedestals is given by

$$f_p = 0.85 \sqrt{A_2/A_1}\, \phi f'_c \leq 1.19 f'_c \qquad (1)$$

in which A_1 = area of the base plate, A_2 = area of the concrete pedestal, ϕ = capacity reduction factor = 0.70, and f'_c = specified compressive strength of concrete. Since the required bearing area for lightly loaded base plates generally will be small compared to the area of the pedestal, the allowable bearing stress using the ACI procedure will approach 1.19 f'_c. Furthermore, the research done by Hawkins[4] and the testing reported herein, demonstrate that stress distribution occurs under the base plate and, therefore, it is recommended that 1.19 f'_c be used for the allowable bearing pressure for lightly loaded column base plates. The resulting proposed design method is then only

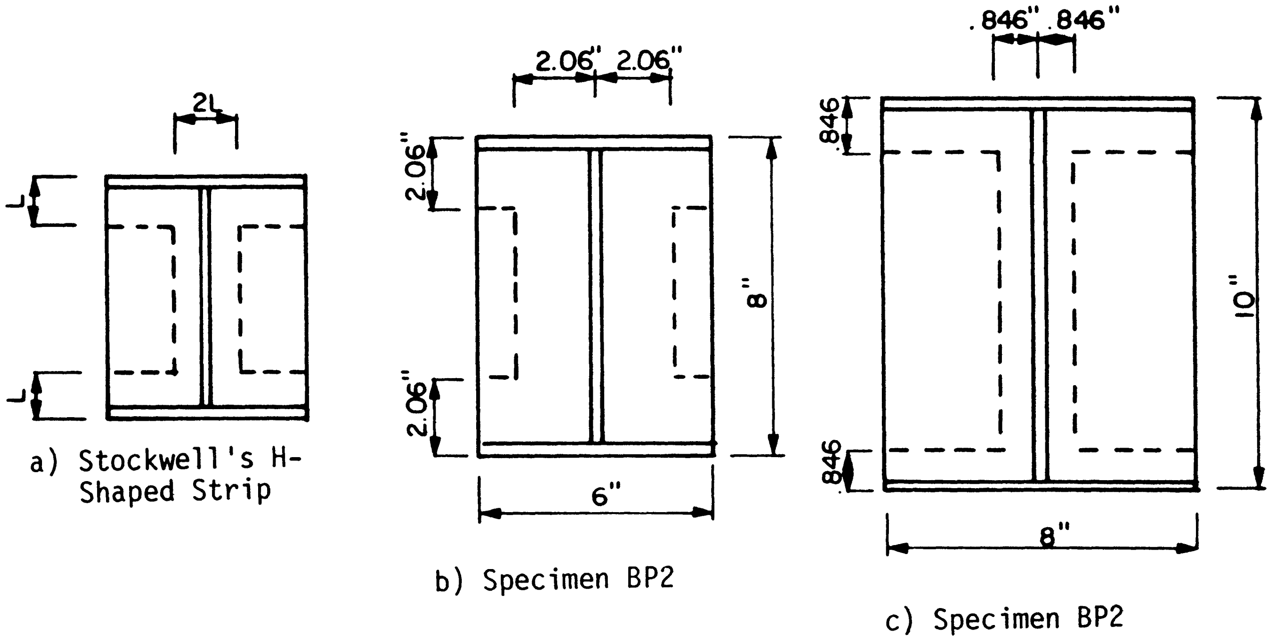

Fig. 4. Effective bearing areas

a variation of the method proposed by Stockwell,[20] where bearing stress for allowable stress design is taken as $0.7 f'_c$.

Using the proposed method with an assumed plate yield stress of 55 ksi (a 10% increase to reflect typical coupon test results) to calculate the plastic moment capacity, m_p, of the base plate and concrete strengths shown in Table 2, the predicted ultimate capacities of the BP1 and BP2 test specimens are 35.6 kips and 148.7 kips, respectively. The H-shaped areas at computed maximum load are shown in Figs. 4b and 4c. The BP1 specimen failed at 187 kips; the strength of the BP2 specimen exceeded the capacity of the testing machine. A complete example is found at the end of this paper.

UPLIFT LOADING

Background—To the writer's knowledge, studies have not been published on the design of lightly loaded column base plate subjected to uplift loading. At least three analysis techniques are available: classical plate bending solutions, the finite element method and the yield-line method. Blodgett[22] and Stockwell[23] have used the yield-line method for certain beam-to-column moment connections where plate elements are subjected to bending from concentrated tensile loads. The method is adopted here.

Yield-Line Analysis—The yield-line method is an energy method requiring the least upper bound of all possible failure mechanisms. Unfortunately, methods are not available to determine directly the least upper bound and trial and error must be used to determine the general yield pattern. However, once the pattern is selected minimization of the internal energy will result in the least upper bound for that pattern.

To determine a possible yield pattern for the typical lightly loaded column base plate shown in Fig. 5, the finite element model for the gravity load specimen BP2, Fig. 1a, was modified for uplift loading applied at the anchor bolt hole location. The base plate was assumed to be fixed along the web and simply supported at the flanges. This analysis indicated the possible yield-line pattern shown in Fig. 5: three lines radiating from the center of the web, one perpendicular and the other two at an angle.

Generally, the internal energy stored by any yield line can be obtained by multiplying the normal moment on the yield line by the normal rotation of the yield line due to an arbitrary virtual displacement at some point on the mechanism. The energy stored in the nth yield line, of length L_n, is then

$$W_{in} = m_p L_n \theta_n \tag{2a}$$

and the total energy stored by all the yield lines is simply the summation of the energies stored by the individual yield lines:

$$W_i = \sum_{n=1}^{K} m_p L_n \theta_n \tag{2b}$$

where K = the number of yield lines in the pattern.

For complicated yield line patterns, it is more convenient to resolve the moments and rotations into orthogonal di-

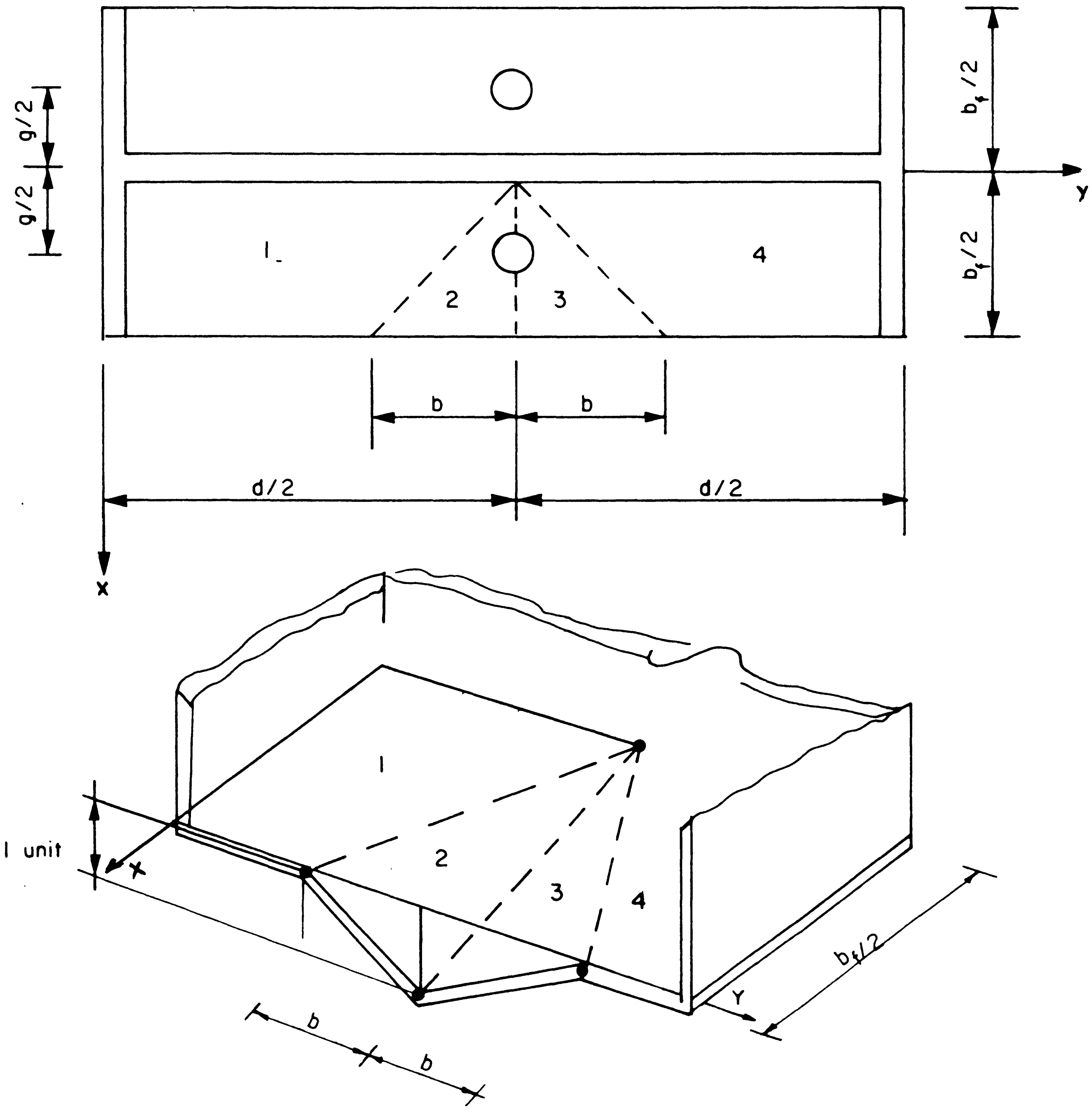

Fig. 5. Uplift yield-line pattern

rections. Thus, the energy stored in the nth yield line can be written as:

$$W_{in} = (m_{px}\theta_{nx}ds_x + m_{py}\theta_{ny}ds_y) \qquad (3)$$

in which m_{px} and m_{py} = the x- and y-components of the normal moment capacity per unit length on yield line n, ds_x and ds_y = the x- and y-components of the yield line elemental length ds, and θ_{nx} and θ_{ny} = the x- and y-components of the relative normal rotation of the yield line n.

Since this equation requires the relative rotations in the x- and y-directions, it is convenient to calculate the rotations of each segment of the pattern shown in Fig. 5 and then determine the relative rotations of the different yield lines. For a one unit virtual displacement at the edge of the centerline of the base plate, the slopes normal to the x- and

y-axes for each of the segments are:

Segment	x-axis	y-axis
1	0	0
2	$-1/b$	$2/b_f$
3	$1/b$	$2/b_f$
4	0	0

The slopes are given as normal to the x- and y-directions. The right hand system is used to define the rotations.

The internal and external work can now be calculated for the virtual displacement. In the derivations that follow, line i/j denotes the line of intersection between the rigid segments I and J. The energy stored in Lines 1/2, 2/3, 3/4 are

$$W_{1/2} = m_p(b_f/2)(1/b) + m_p(b)(2/b_f)$$

$$W_{2/3} = m_p(b_f/2)\{1/b - (-1/b)\} = m_p(b_f/2)(2/b)$$

$$W_{3/4} = m_p(b_f/2)(1/b) + m_p(b)(2/b_f)$$

Adding the energy stored by each yield line, and doubling the quantities derived for half the plate, the total energy stored is

$$W_i = 2m_p\left[\frac{2b_f^2 + 4b^2}{bb_f}\right] \qquad (4)$$

where $m_p = F_y(1)(t^2)/4$ and t = base plate thickness. The internal work is therefore a function of b. Differentiating W_i with respect to b and equating to zero, results in

$$b = \sqrt{2}(b_f/2) \qquad (5)$$

Since b cannot exceed $d/2$, the minimum internal work is obtained for

$$b = \min\left|\begin{matrix}\sqrt{2}(b_f/2)\\ d/2\end{matrix}\right. \qquad (6)$$

The external work is the product of the anchor bolt force and the virtual displacement of its location

$$W_e = \frac{P_u}{2}\frac{g/2}{b_f/2}(1) = \frac{P_u g}{2b_f} \qquad (7)$$

where P_u = the total uplift force on the base plate and g = the anchor bolt gage.

Equating (4) and (7), taking into account (6), results in P_u in terms of m_p. Thus, for a known plate thickness, the ultimate load can be calculated, or, conversely if P_u is known, the required plate thickness can be found.

Experimental Study—Tests were conducted on four base plate specimens. Figure 6 shows the uplift test setup. Each setup consisted of two column/base plate sections, separated by a 1 in. thick layer of expansive grout and bolted with two 1 in. diameter, A36 steel, threaded studs. One section consisted of a short column length and the test base plate. The other section served as a reference section and consisted of a short column section and a 1-in. thick base plate. The same reference section was used for all tests. Dimensions of the test specimen are given in Table 3; all specimens were fabricated from A572 Gr. 50 steel.

Table 3. Dimensions of Uplift Test Specimens

Test No.	Base Plate Dimensions Length (in.)	Width (in.)	Thickness (in.)	Column Dimensions Flanges (in.)	Web (in.)
U1	8.0	6.0	0.364	6 × 0.250	0.183
U2	10.0	8.0	0.368	8 × 0.319	0.245
U3	12.0	6.0	0.377	6 × 0.250	0.316
U4	12.0	8.0	0.375	8 × 0.315	0.315

The specimens were tested in a universal type testing machine. Three dial gages were used to measure deformation of the test base plate, located as shown in Fig. 6. One

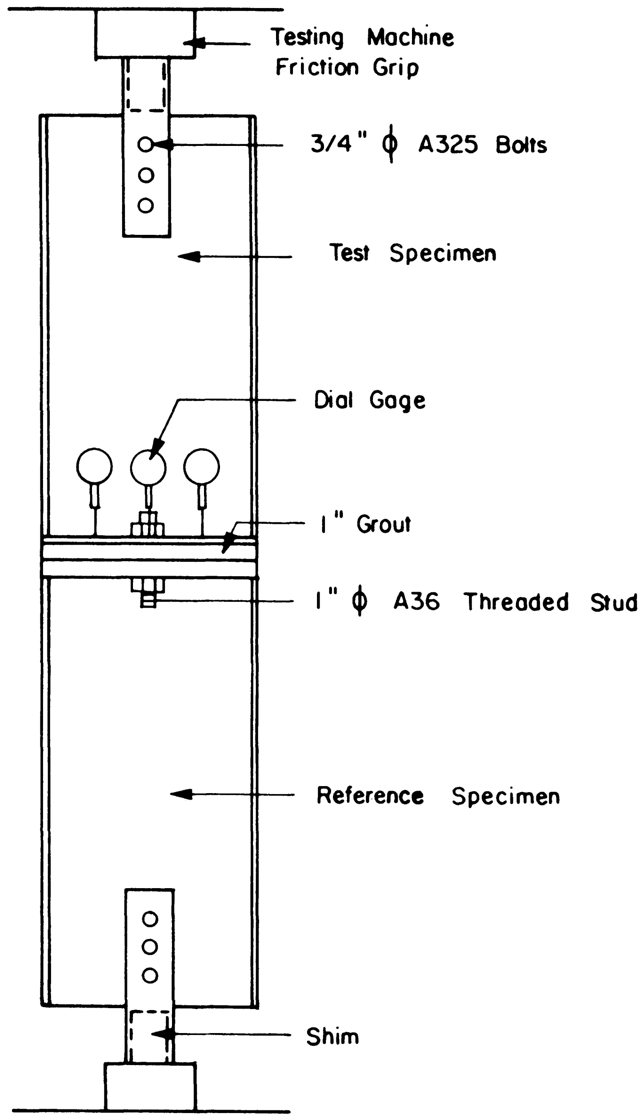

Fig. 6. Uplift test set-up

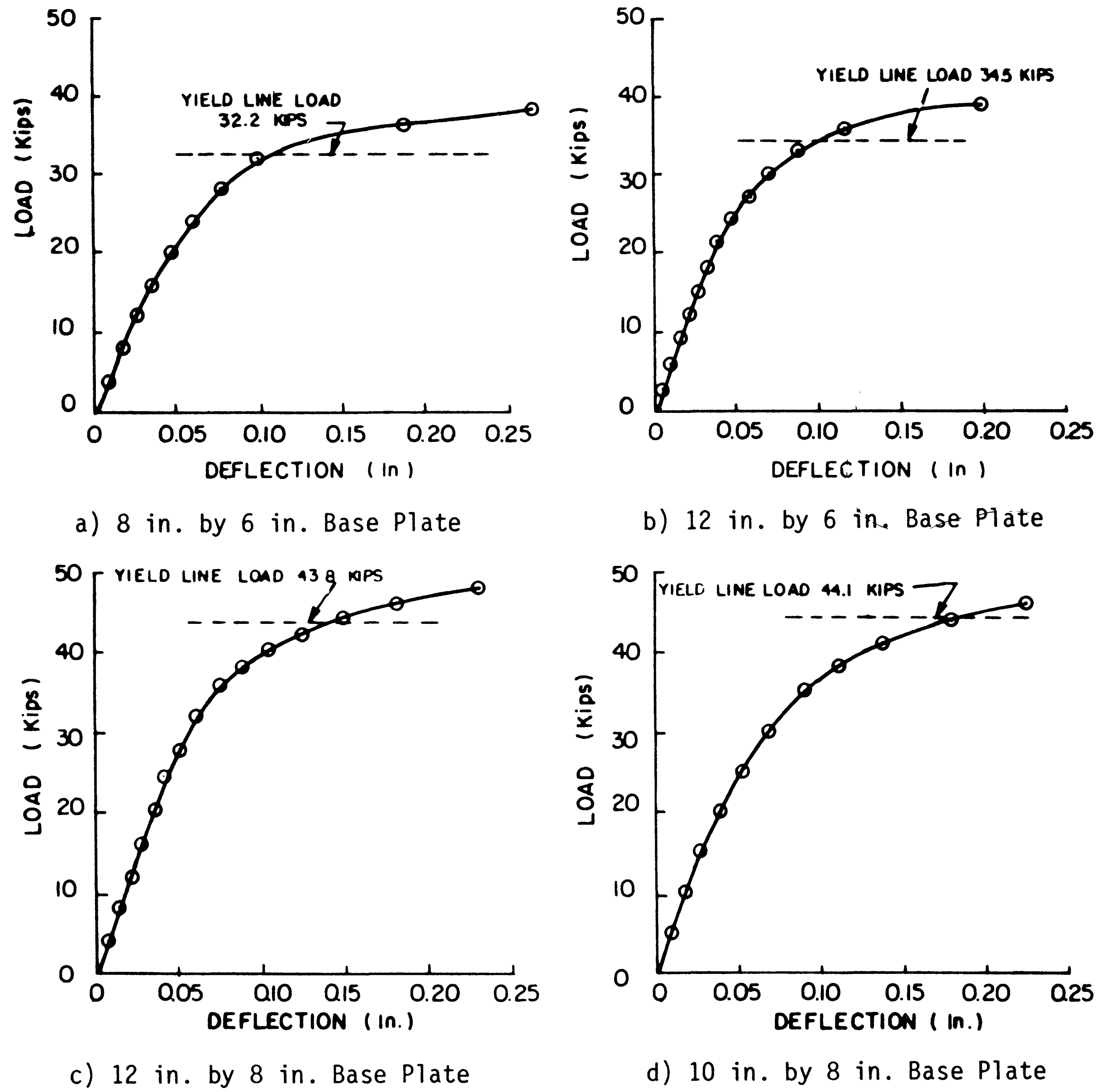

Fig. 7. Uplift load vs. deflection

was placed at the centerline of the plate approximately 0.25 in. from the free edge and the other two were placed at the quarter points of the base plate width and halfway between the free edge and the web centerline. The specimens were coated with whitewash prior to testing.

Deflection at the center dial gage location is shown plotted against applied load for each specimen in Fig. 7. Also shown is the ultimate load predicted by the yield-line analysis. An assumed yield stress of 55 ksi was used since coupon tests were not made. In all tests, the maximum load applied was approximately equal to the predicted ultimate load from the yield-line analysis. Further, from the load-deflection curves, it is obvious that extensive yielding had occurred in the base plate. Observation of the yield patterns, as shown by flaking of the whitewash, showed that the yield-line pattern assumed in the previous section does represent the behavior of thin base plates under uplift loading. Thus, it is concluded that the yield-line solution developed above is an acceptable method for analyzing lightly loaded column base plates subjected to uplift loading.

Application to Design—Equating (4) and (7), using the definition of m_p and the limitations of (6), results in

$$t = \sqrt{\frac{\sqrt{2}P_u g}{4b_f F_y}} \quad \text{for } \sqrt{2}b_f \leq d \tag{8}$$

and

$$t = \sqrt{\frac{P_u g d}{F_y(d^2 + 2b_f^2)}} \quad \text{for } \sqrt{2}b_f > d \tag{9}$$

A design example is found at the end of this paper.

ERECTION SAFETY

The design procedures proposed here may result in relatively thin base plates. As a result, sufficient strength to maintain column stability during erection may not exist. The critical case is, of course, a long column subjected to lateral forces near the top due to erection procedures or high winds. This situation was not investigated in the study reported here, but it is believed that the procedure proposed for the uplift case can easily be modified by a designer to investigate the strength of a base plate when subjected to an overturning moment.

DESIGN EXAMPLE

Gravity Loading

Given: Select a base plate for a W8x24 column supporting a service load of 55 kips, f'_c = 3000 psi, A36 steel.
Solution: For W8x24, d = 7.93 in. and b_f = 6.495 in. With a load factor of 1.7 and a compressive concrete stress of 1.19 f'_c, the required area is

$$A_r = \frac{(1.7 \times 55)}{(1.19 \times 3.0)} = 26.19 \text{ in.}^2 < 7.93 \text{ in.} \times 6.495 \text{ in.}$$

Try 8 in. by $6\frac{1}{2}$ in. base plate.
Using Fig. 4, the length of the cantilever is determined from $[8 + (6.5 - 2L)]L = 26.19$. Therefore, L = 1.057 in. The resulting moment is then

$$m_p = \frac{(1.19 \times 3.0)(1.057)^2}{2} = 2.00 \text{ in.-kip/in.}$$

The required section modulus is

$$Z_r = \frac{m_p}{F_y} = \frac{2.00}{36} = 0.0555 \text{ in.}^3 = t^2/4$$

or

$$t = 0.471 \text{ in.}$$

Use: $PL\frac{1}{2} \times 6\frac{1}{2} \times 0$ ft 8 in.
(Note, using the procedure found in the 8th ed. AISC *Manual of Steel Construction*, a 1-in. thick plate would be required).

Uplift Loading

Given: Check the above base plate for an uplift load of 20 kips. The anchor bolt gage is 4 in.

Solution: With a load factor of 1.3

$$P_u = 1.3(20) = 26 \text{ kips}$$

Since $\sqrt{2}b_f = \sqrt{2}(6.495) = 9.18$ in. is greater than d = 7.93 in., Eq. 9 applies.

$$t^2 = \frac{P_u g d}{F_y(d^2 + 2b_f^2)} = \frac{26.0 \times 4.0 \times 7.93}{36(7.93^2 + 2 \times 6.495^2)}$$

and

$$t = 0.394 \text{ in.}$$

The plate is satisfactory.

ACKNOWLEDGMENTS

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