DESIGN OF SLIP BASES FOR BREAKAWAY SIGNS

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The object of the study was the design of economical slip bases for breakaway sign supports. The style of baseplate previously used in Kansas, although most economical to fabricate, was too heavy because the baseplate thickness had been based on a theoretical analysis that contained several conservative assumptions. Therefore, full-scale tests were run using experimental stress analysis techniques to determine a more accurate analysis method. A design method was developed based on these results. Application of the method allows the use of flat baseplates that meet the maximum weight recommendations set forth by the Texas Transportation Institute. This is estimated to result in an annual savings of $20,000 in Kansas.

•THIS paper discusses the design of the slip base portion of breakaway sign supports, specifically the baseplates. Recommendations for design of sign supports of this type were developed at the Texas Transportation Institute (TTI) as part of a cooperative highway research project sponsored by several states and the Federal Highway Administration (1-6). The Federal Highway Administration recommended that certain criteria (6) be followed in the design of breakaway supports.

One of the elements specified in the TTI criteria is baseplate weight. The TTI researchers checked the effect of baseplate weight on collision performance using a computer simulation that they developed from actual crash test data. They found that the weight of the baseplate had very little effect on system response within certain practical limitations. The maximum baseplate weights recommended in the criteria are assumed to be these practical limits.

Kansas began using breakaway supports on an experimental basis in 1966. The bases used on the heavier post sections in Kansas differed from those used in Texas and tested at TTI (Fig. 1). The Kansas base is more economical because it requires less labor to fabricate. As part of a project to update all sign supports to match revised AASHO specifications, the Kansas designs were reviewed in 1970. Some baseplates on these designs were found to be significantly heavier than those recommended (6). It was desired to continue use of this type of base in Kansas but to reduce its weight.

In the early Kansas designs the thickness of the baseplates had been based on a theoretical analysis containing several conservative assumptions: that the plates bent in a single curvature about the post flange when the base was subjected to a moment, that the load causing this bending was equal to the bolt load (due to base moment and calculated by statics) applied at the centerline of the bolts, and that the baseplate bending stress should be limited to the same allowable value as the post flange (assuming that the plate and flange are of the same material). This method overestimates thickness requirements because plasticity effects and the reinforcement provided by the weld are not considered. Also, the bolts and washers provide some bending restraint and provide a load application point that is closer to the post flange than the bolt centerline.

Because thicknesses determined by the old design method generally result in baseplate weights that are greater than those recommended (6), full-scale tests were run to determine a more accurate method. Most of the test results for one of the four test designs are given in this paper. The full test data are given elsewhere (3). A design method based on these tests is developed, and application of the new design method to bases to be used in Kansas is discussed.

STATIC STRENGTH TESTING

It appears that the static strength of slip bases subjected to a moment as shown in Figure 2 would be dependent on the following geometric variables: S, T1, T2, C, A, E, d, flange width, flange thickness, web thickness, and bolt diameter. In addition, the strength parameters of the materials would be important, but these relations are known. A complete testing program would require a large number of tests. This was not considered necessary because most of the variables have a small range of values for practical designs. It was decided to test specimens of actual proposed designs in which the thickness of the top baseplate was such that its weight would approximately be equal to that recommended (6). This would prove or disprove the adequacy of the designs tested and allow development of a design method for bases of similar proportions.

Test Procedure and Equipment

Tests were run on the four bases with dimensions as given in Table 1. In all tests the loading setup was arranged to produce moment at the slip base as shown in Figure 2. The load was applied at a slow rate. Work was stopped at incremental stages to observe the strain measuring devices. Loading was done with a 450,000-lb universal testing machine and a special mounting frame (Fig. 3). The W 12×19 test was run first to determine if further tests were necessary and to try out the testing procedure. Calculations indicated that, if the W 12×19 base tested could develop the full moment resistance of the post, a $^{5}/_{8}$-in. thick plate could be used for smaller post sections, and further testing would not be required. This was found not to be the case, so tests on three more bases were performed. It was assumed that the worst loading condition for bases with the same top and bottom plates would be with the load close to the base because for a given moment the shear would be higher. For this reason, the first tests were run with the load at the point nearest the base for which slipping of the baseplates would not occur. The location of this point was determined by the value of the coefficient of friction. In the W 12×19 test, the coefficient-of-friction value was taken as 0.35 in calculating the load position. Problems with slippage occurred, so the value used was lowered to 0.20 for the final set of tests. Problems with slippage still occurred, so the load was moved out to a point 60 in. from the base; this provided for a coefficient of friction of about 0.10.

Data from which to develop the design method were obtained by using strain measuring devices in the tests. These included brittle coatings, photoelastic coatings, and electric resistance strain gauges. Figure 4 shows the location of these devices on the bases tested. In the W 12×19 test, the only strain gauge used was rosette A, which was a paperbacked wire rosette with 0.28-in. gauge lengths. A six-channel bridge balancing unit was used in a half bridge circuit with temperature compensation provided by a matching gauge mounted on a block of steel.

For tests on the other three bases, phenolic glass-backed foil-stacked rosettes, with 0.12-in. gauge lengths, were used. The shorter gauge lengths and stacked arrangement provide better results because sharp strain gradients were present. Gauges E and F were phenoloc glass-backed foil gauges with 0.06- and 0.12-in. gauge lengths, and gauge G was purchased preassembled in the bolt. A 20-channel bridge balancing unit was used in half bridge circuit with a dummy precision resistor in the inactive bridge arm. The gauges were temperature-compensated.

Contact cement was used for installation of gauges in all tests, and a null balance strain indicator was used to read out data in all tests.

Aerosol application-type brittle lacquers were used in all tests along with a special calibration device.

Photoelastic coatings were used on all but the W 12×19 base. The coating used had a strain optical coefficient of 0.15, a thickness of 0.125 in., and a fringe constant of 605 μ strain/fringe. A reflection polariscope was used to obtain orthochromatic fringe patterns and isoclinal lines. These were recorded on color slides using a 35-mm camera.

Figure 1. Comparison of Texas breakaway base and Kansas breakaway base.

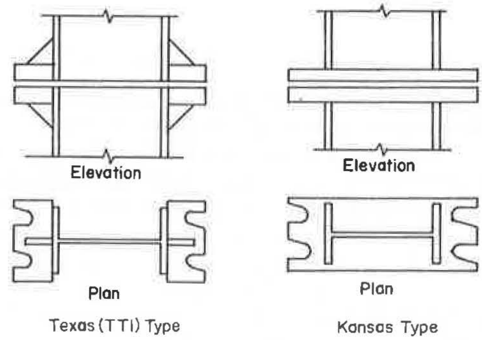


Figure 2. Kansas base.

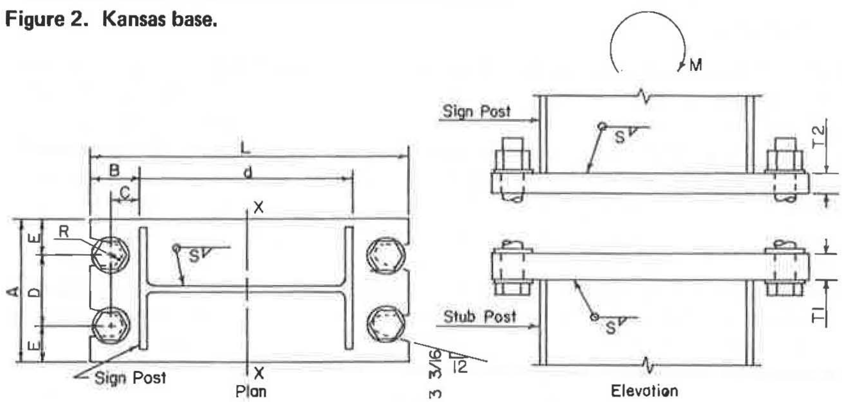
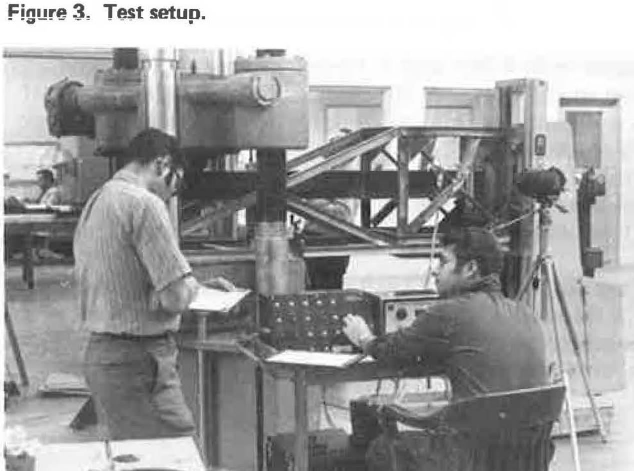


Table 1. Dimensions of bases tested.

Type of Post	Dimension (in.)										Bolt Size (in.)
	S	T1	T2	A	B	C	D	E	L	R	
W 12×19	1/4	5/8	5/8	4 5/8	2	1 1/8	2 1/2	1 1/16	14 1/8	13/32	3/4 by 2 3/4
W 6×8.5	1/4	7/8	5/8	4 5/8	1 7/8	1 1/8	2 1/2	1 1/16	9 5/8	11/32	5/8 by 3
W 10×11.5	1/4	1	3/4	4 5/8	1 7/8	1 1/8	2 1/2	1 1/16	13 5/8	11/32	5/8 by 3 1/2
W 10×21	1/4	1 1/4	7/8	6 3/8	2 1/4	1 1/4	3	1 11/16	14 3/8	15/32	7/8 by 4

Note: Structural shapes and plate according to ASTM A-36 and bolts according to ASTM A-325.

Figure 3. Test setup.



Test Results

Raw strain readings from rosettes were reduced to maximum and minimum principal strains using a calculator. The principal strain at these rosettes was very nearly perpendicular to the post flange. The maximum or minimum principal strain at each rosette location was plotted against base moment. Strain at gauge E was plotted against base moment. Gauges F and G were at locations where the stress could be reliably determined by theory and served mainly as a check. Stress at gauge F and bolt load at gauge G were plotted against moment at gauge F and base moment respectively.

The extent of stress-coat cracking was marked at each increment where it was observed, and photographs showing the marks were taken.

Orthochromatics were recorded at each load increment where the strain gauges were read. Isoclinal line patterns were recorded at only one load increment for 15-deg rotations of the polarizer-analyzer. These were then displayed one at a time and traced on the same sheet of paper.

In the W 12×19 test, bending of the base bottom baseplate was observed at a base moment of 387 kip-in. The test was continued to a moment of 454 kip-in. at which the gap between the baseplates, on the side where the bolts are in tension, had grown to about $^1/_4$ in. In the W 6×8.5 test, first bending of the top baseplate on the tension bolt side was observed at 220 kip-in., and the largest test moment was 252 kip-in. Bending of the tensile bolts was observed at about the same time and rate as the bending of the baseplate. The largest test moment in the W 10×11.5 was 420 kip-in. and in the W 10×21 was 726 kip-in. No significant bending of baseplates was noted in either of these tests. The moment in the W 10×11.5 test was limited to the proof load of the strain-gauge bolt to avoid damaging it. In the W 10×21 test, the moment was limited by the deflection of the testing frame.

After testing of the bases was completed, tensile strength specimens were cut from the baseplates and were tested. The results of these tests are given in Table 2.

Analysis of Results

In some cases the test results indicated the adequacy of the design tested. Because the W 12×19 bottom baseplate began bending at a moment of 387 kip-in. and because it would take a moment of 840 kip-in. to produce the yield point stress (39.3 ksi) in the post flanges, it was concluded that this design was unsatisfactory. A moment of 200 kip-in. would produce yield point stress (38.9 ksi) in the post flange of the W 6×8.5, but this base was subjected to a moment of 220 kip-in. before baseplate bending was noted: It was concluded that this design was satisfactory because the post would fail before the baseplate. In the other two tests the largest moments reached (420 and 726 kip-in.) were less than the post yield moments of 474 kip-in. (at 45.2 ksi) and 970 kip-in. (at 42.2 ksi) respectively.

The results obtained from the strain indicating devices clarify the stress behavior of the Kansas type of breakaway base. The stress-coat crack patterns all indicate concentrations of stress near the bolts. This verifies that the strain rosettes were located in the proper places to detect maximum strains. It also indicates the importance of dimensions C and A (Fig. 2). The measured strains were primarily dependent on base moment and almost independent of base shear. This is shown by the closeness of results from test series done with the load applied at different points on the same specimen.

Maximum principal strain results always occurred at rosette A (Fig. 4) except in the W 12×19. This is contrary to what theory predicts because base moment causes compression at rosette A, whereas base shear causes tension at rosette A (both in the direction of principal strain). At rosette B, on the other hand, theory says that the strains caused by base shear and moment are additive. The author believes this phenomenon is due to the difference in restraint caused by the bolts and washers. The bolts on the tension side (where rosette A is located) were observed to bend in one of the tests. This is logical because they are not as stiff as the baseplate. The orthochromatics do not clearly indicate whether there is any restraint causing double curvature or not. The isoclinics do appear to indicate double curvature of the tension side baseplates. On the compression side (where rosette B is located), the middle washer is in compression. Because this washer cannot deform sufficiently to provide for much rotation, between baseplates, it must offer some restraint. Perhaps the most important results are that maximum measured principal strains ranged from only 59 to 73 percent of those predicted by the old design assumptions and that about 50 percent additional load can be taken, after the yield strain level is reached, before significant bending of the baseplates takes place. This latter fact can be explained by plasticity theory when it is remembered that the shape factor for a rectangular section is 1.5.

Figure 5 shows close agreement between the measured flange stress at gauge F and the calculated flange stress (x - x moment at gauge F divided by post section modulus). Figure 6 shows close agreement between the measured bolt load and the calculated bolt load (x - x base moment divided by 2d).

Based on the results obtained in these tests and general knowledge of structural behavior, a theory was developed to explain the behavior of the bases. First, it was shown by the tests that the top baseplates fail by bending about the tension flange of the post. It was also shown that the critical principal strain was mainly dependent on base moment. Therefore, it was assumed that the principal strain of the baseplate is linearly proportional to the moment in the plate divided by the section modulus of the plate. Second, the bending load is applied to the baseplate by the bolts; therefore, the baseplate bending moment is linearly proportional to the bolt load times the distance from bolt to flange. Third, the bolt load can be calculated by static strength from base moment; this was verified by the measured bolts. All of the preceding assumptions were used in previous theoretical analyses. The lower principal strain values observed in the tests can be accounted for by introducing other parameters. These parameters must account for the following factors: restraint caused by the bolts and washers, load being applied through the washers rather than through the centerline of bolts, reinforcing effect of the weld, and uneven bending stress distribution in the plate. Based on only three tests, it was impossible to account for all of these factors. (The W 12×19 test was not considered at this point because its design was such that the bottom plate failed rather than the top.) The first three factors had about the same effect in the other three tests and would have about the same effect for all practical designs. Observation of stress-coat crack patterns demonstrated that the last factor was dependent on the ratio C/A (Fig. 2). It was decided that, to arrive at one dimensionless parameter (K) to represent all four parameters, this parameter would be dependent on C/A. A solution was desired in the form ϵ-plate = f(M, T2, K), and it was known that $\epsilon\text{-plate} = \frac{6}{29 \times 10^3 \text{ ksi}} \frac{M\,C}{A(T2)^2\,(2C + d)}$ for K = 1, based on the first three assumptions given earlier. Terms used in equations are shown in Figure 2. A plot of C-plate (measured) divided by C-plate (calculated with K = 1) versus C/A was constructed (Fig. 7). It showed good correlation between the two tests with the same C/A ratio and indicated that the stress concentration factor decreases with increasing C/A as was expected. The ϵ-plate (measured) and ϵ-plate (calculated K = 1) were determined for M such that ϵ-plate measured equaled ϵ-yield. From this plot, the equation K = 0.78 - 0.80 (C/A) was determined. Thus, the equation for baseplate stress becomes

$$f = \frac{6MC\,[0.78 - 0.80\,(C/A)]}{A(T2)^2\,[2C + d]} \tag{1}$$

It is felt that this equation can be used with reasonable confidence for bases similar to the ones tested.

DEVELOPMENT OF DESIGN METHOD

Equation 1 serves as the basis for a new design method. An additional consideration required for design is the correct value of allowable stress (Fb). Most elastic design specifications provide for an increase in allowable stress based on plasticity theory

Figure 4. Location of strain measuring devices.

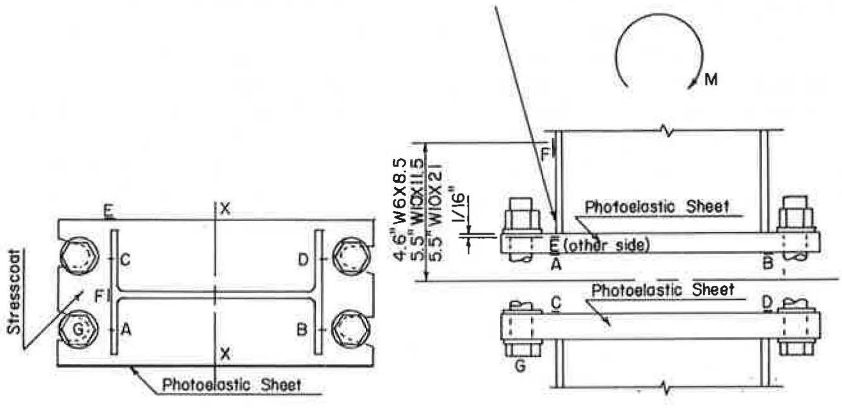
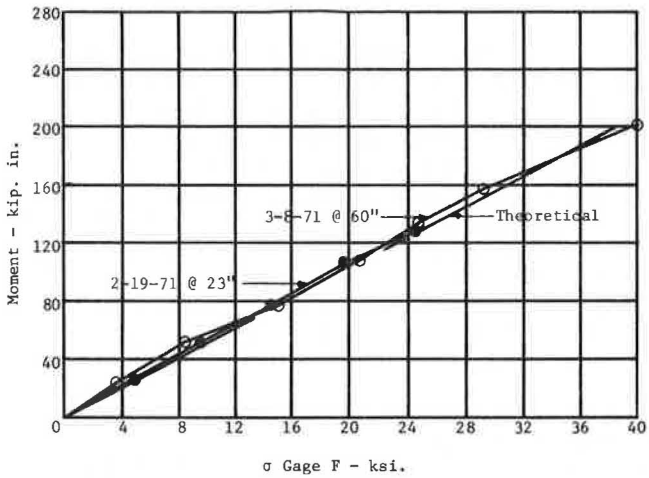


Table 2. Results of tensile tests.

Specification No.	Plate Thickness (in.)	Yield Point (psi)	Ultimate Stress (psi)	Elongation of 2-in. Gauge (percent)	Type of Post
1	5/8	38,895	65,888	36.5	W 6×8.5
2	3/4	45,231	72,820	26	W 10×11.5
3	7/8	42,154	73,846	33	W 10×21
4	5/8	39,289	65,990	33	W 12×19

Figure 5. Stress at gauge F as function of moment for W 6x8.5.



when the full or partial plastic moment can be developed by the member. For example, AASHO specifications allow a 9 percent increase for compact W-sections and 20 percent for round or oval tubes. A similar increase should be allowed for the rectangular baseplate. The shape factor for W-sections varies from 1.10 to 1.18, and a 9 percent increase is used with them. Because the shape factor for rectangular sections is 1.5, a 45 percent increase should be allowed for baseplates. For design, it is desirable to solve Eq. 1 for T2 in order to find plate thickness required for a given design moment and for M in order to find the allowable moment for a given design. This was done for an AASHO group II loading that allows an additional 45 percent increase in allowable stress, resulting in the following equations:

$$T2 = \frac{2.86\ MC[0.78 - 0.80(C/A)]}{Fb\ A\ [2C + d]} \tag{2}$$

$$M = \frac{FbA(T2)^2[2C + d]}{2.86\ C\ [0.78 - 0.80(C/A)]} \tag{3}$$

The design moment should be based on AASHO group II loading.

For purposes of comparison with test results, ultimate moments were computed for the bases tested using Eq. 3 with Fb = Fy/1.45. The moments obtained were 210 kip-in. for the W 6×8.5, 522 kip-in. for the W 10×11.5, 790 kip-in. for the W 10×21, and 374 kip-in. for the W 12×19. These compare with test moments where bending was noted of 220 kip-in. for the W 6×8.5 and 387 kip-in. for the W 12×19. Note that the design method is on the conservative side. Moments of 420 kip-in. for the W 10×11.5 and 726 kip-in. for the W 10×21 were the highest obtained in those tests with no bending noted.

Some concern has been voiced regarding the rigidity of the Kansas design baseplates. The concern apparently centers around the following statement (part II, p. 4:123, 6): "It must be pointed out that rigidity of the base plates is very important to the operation of the base and the theory developed to explain it. If significant changes are made in the design of the base, the force-slip characteristics should be re-evaluated by laboratory test."

Regarding this, the author wishes to make the following points. Texas and other states are using the Kansas design for small post sections, and there are many documented cases of their satisfactory performance (2). The removal of the stiffener tends to make the Kansas design base less rigid than the Texas design; however, the continuation of the plate between post flanges tends to make the Kansas design base more rigid than the Texas design. A coefficient of friction of 0.2 was found at small slips for two of the bases tested. The 0.2 figure was calculated by the same method as used by TTI and is within the range of values reported (part II, p. 3:45, Fig. 3.3.1, 6). In comparing coefficients of friction, it should be remembered that the surface condition of the baseplates is important. In this project, the tests were run on ungalvanized bases that had been cleaned of mill scale. Ungalvanized steel surfaces generally have a higher coefficient of friction than galvanized surfaces, but mill scale removal lowers the value somewhat.

It is the author's opinion that the Kansas design does not constitute a significant change that would affect the force-slip characteristics.

The design method herein developed is recommended only for use in design of bases similar to those tested. Use of the method results in allowable moments about 2½ times as large or baseplate weights about two-thirds as much as those designed by previous theoretical methods. Based on discussion with fabricators, who make both types of bases, it is believed that fabrication labor for the Kansas base is about half that for the Texas base. Material, shipping, and installation costs for both bases are believed to be small. Because contract bid prices for breakaway bases in Kansas have averaged about $25, it appears that fabrication labor must average about $20. Because Kansas installs about 1,000 bases per year, it is estimated that a $20,000 annual savings will be realized by the use of bases designed by this method.

Figure 6. Bolt load as function of moment for W 6x8.5.

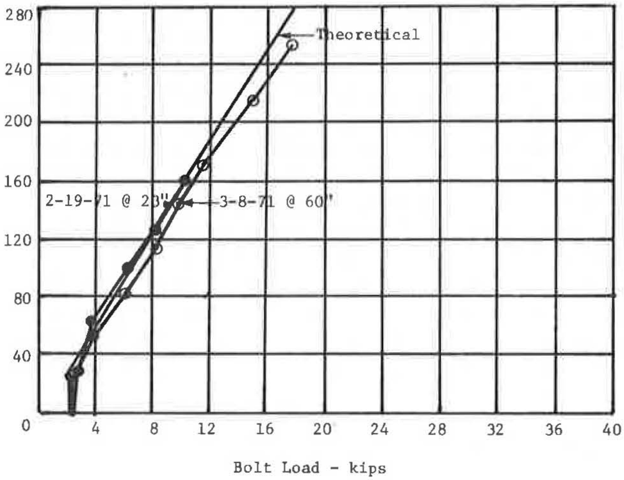


Figure 7. Relative strain as function of C/A.

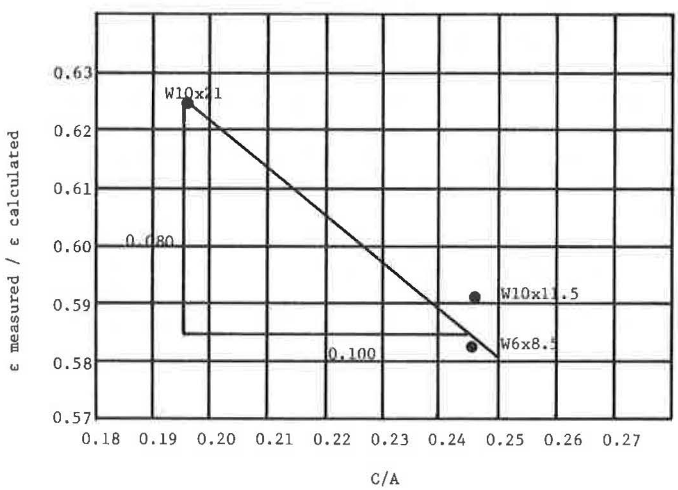


Table 3. Dimensions of Kansas breakaway bases.

Type of Post	Dimension (in.) S	T1	T2	A	B	C	D	E	L	R	Bolt Size (in.)
W 6×8.5	1/4	7/8	5/8	4 5/8	1 7/8	1 1/8	2 1/2	1 1/16	9 5/8	11/32	5/8 by 3
W 10×11.5	1/4	1	3/4	4 5/8	1 7/8	1 1/8	2 1/2	1 1/16	13 5/8	11/32	5/8 by 3 1/4
W 10×21	3/8	1 3/8	1	6 5/8	2 3/8	1 3/8	3	1 11/16	14 5/8	15/32	7/8 by 3 3/4

Note: Structural shapes and plate according to ASTM A 441 and bolts according to ASTM A 325.

IMPLEMENTATION: DESIGN OF BASES FOR USE IN KANSAS

The new design method has been used to design three slip bases for standard break-away supports. The W 6×8.5 and W 10×11.5 are the same as those tested, but the W 10×21 has a larger fillet weld and thicker baseplate. The baseplates are fabricated from ASTM A 441 steel as are the posts. The design moments used in designing the bases were equal to the maximum allowable moment on each post. Size of bolts and welds was determined by AASHO specifications. The dimensions of Kansas standard designs are given in Table 3; they correspond to those shown in Figure 2. The plan dimensions are the minimum ones that will allow sufficient clearance and edge distances for bolts and welds. When the dimensions and design moment were known, Eq. 2 was used to determine the required thickness. In the case of the W 10×21, the required thickness turned out to be such that the plate would weigh slightly more than the maximum value recommended (6). The weight was reduced by taking clips out of the plate between flanges. Although this probably reduces the strength slightly, the thickness provided in going to standard plate thickness is more than the minimum required, and the design is felt to be satisfactory.

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