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16. Abstract

A study of the practice of supporting formwork with cast-in-place and welded form-hanger anchors in precast, prestressed concrete girders was accomplished. The study included identification of the types of anchors available for use in Texas, identification of the existing procedures for selection and installation of such anchors, and laboratory testing of 60 anchors of various types in a full-scale girder. It is concluded that while the anchors tested did generally meet manufacturers' specified strengths, some anchor systems tend to fail in a brittle mode, without exhibiting significant deformation prior to ultimate failure. Furthermore, the strength of the various anchors is usually strongly dependent on edge distance, and specifications do not exist to satisfactorily ensure the safe placement and inspection of such anchors. Guidelines, in the form of a draft specification and interim recommendations, are submitted for consideration by the Texas State Department of Highways and Public Transportation.

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CAPACITY OF FORM-HANGER ANCHORS IN PRESTRESSED CONCRETE GIRDERS

by

Ray W. James

Research Report No. 1171-1F

Research Study 2-5-88-1171
Capacity of Form-Hanger Anchors

Conducted for the

Texas State Department of Highways and Public Transportation

In Cooperation with the

Department of Transportation
Federal Highway Administration

by the

Texas Transportation Institute
Texas A&M University System
College Station, Texas

October 1989

METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in²	square inches	645.2	millimetres squared	mm²
ft²	square feet	0.0929	metres squared	m²
yd²	square yards	0.836	metres squared	m²
mi²	square miles	2.59	kilometres squared	km²
ac	acres	0.395	hectares	ha
		MASS (weight)		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		VOLUME		
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft³	cubic feet	0.0328	metres cubed	m³
yd³	cubic yards	0.0765	metres cubed	m³

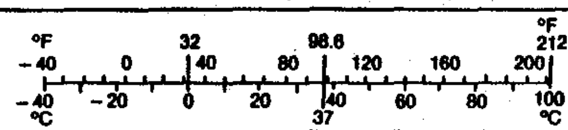
NOTE: Volumes greater than 1000 L shall be shown in m³.

Symbol	When You Know	Multiply By	To Find	Symbol
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

* SI is the symbol for the International System of Measurements

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm²	millimetres squared	0.0016	square inches	in²
m²	metres squared	10.764	square feet	ft²
km²	kilometres squared	0.39	square miles	mi²
ha	hectores (10 000 m²)	2.53	acres	ac
		MASS (weight)		
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m³	metres cubed	35.315	cubic feet	ft³
m³	metres cubed	1.308	cubic yards	yd³
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

SUMMARY

A study of the practice of supporting formwork with cast-in-place and welded form-hanger anchors in precast, prestressed concrete girders was accomplished. The study included identification of the types of anchors available for use in Texas, identification of the existing procedures for selection and installation of such anchors, and laboratory testing of 60 anchors of various types in a full-scale girder. It is concluded that while the anchors tested did generally meet manufacturers' specified strengths, some anchor systems tend to fail in a brittle mode, without exhibiting significant deformation prior to ultimate failure. Furthermore, the strength of the various anchors is usually strongly dependent on edge distance, and specifications do not exist to satisfactorily ensure the safe placement and inspection of such anchors. Guidelines, in the form of a draft specification and interim recommendations, are submitted for consideration by the Texas State Department of Highways and Public Transportation.

SUMMARY STATEMENT ON RESEARCH IMPLEMENTATION

The findings of this study indicate that specifications or standards are needed to cover the use of form-hanger anchors used with precast prestressed concrete bridge girders. At the present time, existing specifications do not address requirements for such anchors, and the lack of such specifications has allowed the use of anchor components and methods which are not regulated or inspected, simply because such anchors are "temporary" components of the structure, for use only during deck placement. A draft specification is offered for consideration by the Department. Recommendations for use of such anchors in the interim are also provided. These recommendations should be reviewed by bridge construction engineers, bridge design engineers, and inspectors at precast plants or construction sites, or other individuals responsible for approving anchor selection, installation and use, or responsible for approving formwork design.

DISCLAIMER

The contents of this report reflect the views of the author, who is responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration or the State Department of Highways and Public Transportation. This report does not constitute a standard, specification or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

ACKNOWLEDGEMENTS

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Technical support and advice was provided by the Technical Coordinator, Berry English (D-5), the Technical Advisor, K. J. Janak (D-9), and Leroy A. Surtees, Jr., (District 15). Contact with Berry English and Leroy Surtees was maintained by telephone and occasional meetings to assure effective communication of findings and problems.

James H. Loper and Heping Zhang assisted with the field work, the experimental work, and the analytical work.

The precast concrete girder was very generously donated to the study by Manco Prestress Company of San Antonio. The cooperation and enthusiasm exhibited by all of Manco's personnel, from President Carlos Cerna right down to the fabrication crew, was remarkable.

All the anchor specimens tested were donated to the study by the manufacturers, who were also helpful in providing specifications, technical information, and insight into the problem.

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Capacity of Form-Hanger Anchors in Prestressed Concrete Girders

INTRODUCTION

Form-hanger anchors are used in the construction of cast-in-place reinforced concrete bridge decks to support the formwork during placement of the plastic concrete. In the case of precast concrete girders, the form-hanger anchors may be cast-in-place in the top of the girder at the time of fabrication or secured to the reinforcing steel projecting from the top surface of the girder by welding or other means. The cast-in-place anchors are typically of fabricated steel or cast steel or iron construction, supplied to the girder fabricator by the bridge construction contractor. Anchors designed for connection to the projecting reinforcing steel are usually installed at the construction site after the prestressed girder is erected. Anchors are also available for use with steel girders, however the study discussed here is limited to those anchors for use with prestressed concrete girders.

Recent accidents in which cast-in-place exterior form-hanger anchors have failed, causing loss of the plastic concrete deck and reinforcing steel, have served to focus attention on the structural integrity of the form-hanger anchors. Since the anchors serve only as an aid to construction and do not serve a purpose in the completed bridge structure, specifications covering the design, manufacture, testing, selection, installation and inspection of form-hanger anchors are lacking. Because the anchors support the formwork which is used as a working surface by contractor's personnel and SDHPT inspectors, and because of the possible presence of traffic beneath the bridge during deck placement, it is desirable to develop sufficient guidelines for the use of form-hanger anchors to ensure the safety of personnel and the travelling public.

Anchors may be used for supporting both exterior formwork cantilevered past the outside girder and for supporting interior formwork between two girders. The use of precast, stay-in-place concrete panels for interior formwork has recently become common practice. Because of this fact this study does not address interior form-hanger anchors.

SCOPE OF THE STUDY

The following tasks were accomplished as a part of this study:

1. Identify the different types of exterior form-hanger anchors commonly used in bridge construction in Texas.

2. Identify parameters which might affect the installed integrity of the form-hanger anchors.

3. Design and conduct tests to determine the effects of the various parameters on the installed structural integrity of the form-hanger anchors.

COMMONLY USED FORM-HANGER ANCHORS

By interviewing various SDHPT engineers and precast girder fabricators, the anchors listed in the following table were determined to be available to and, with one exception, specified by contractors in Texas.

TABLE 1. COMMERCIALLY AVAILABLE FORM-HANGER ANCHORS

Manufacturer	Model No.	Description	Rated Safe Load (lbs)
Dayton	1-A	Fab. Steel, CIP	3500
Dayton	1-AP	Fab. Steel, CIP	2000
Richmond	HFR-HPA	Fab. Steel, CIP	4500
Texas Found.	FH15C	Cast Steel, CIP	3200
Texas Found.	FH45A	Cast Steel, CIP	3200
Dayton	1-AC	Fab. Steel, Welded	3000
Dayton	4-AC	Fab. Steel, Welded	6000
Richmond	HFR-HWA	Fab. Steel, Welded	4500

The Texas Foundries FH45A is the only listed anchor not presently available for use in Texas. This anchor is a prototype which will be marketed in Texas in the near future, according to the manufacturer. Because of this, the anchor was included in the test matrix. The Texas Foundries FH15C model has seen only limited use in Texas but is reportedly more widely used in other states, particularly Louisiana.

DESCRIPTION OF EXPERIMENT

A 60-ft, Type C prestressed concrete girder was fabricated as a test girder. Sixty anchors were placed in the plastic girder in a predetermined random order, as described in Table 2. The random order, selected with the aid of random number tables, was used to ensure that local variations in concrete strength or time of placement did not bias the measured anchor strengths. As shown in Table 2, most of the anchors were placed in sequence, beginning immediately after the placement and consolidation of the plastic concrete. Some anchors, labeled "late" in Table 2, were not placed immediately, but were delayed 70 minutes to evaluate the significance of delayed placement. The girder was cast on March 4, 1988, beginning at approximately 3:00 pm, in San Antonio, Texas. Climatic conditions were very favorable for placement of concrete, with cool temperatures and overcast skies. Placement of the majority of the anchors took approximately 40 minutes, and placement of the "late" anchors took approximately 10 minutes. Because of the favorable climatic conditions, the 30-70 minute delay in placement of the "late" anchors was thought to be a relatively insignificant factor in the ultimate performance of the anchors. The difficulty of installation of the "late" anchors was not noticably greater than that of the normally placed anchors, and it is felt that installation might be delayed even longer in some circumstances.

Observed installation difficulties included some interference with the reinforcing steel, the prestressing steel, and two prestressed strands above the top surface of the girder which were used to align and support the "R" bars. The concrete was stiff enough that a visible void was frequently left around, particularly in back of, the

Table 2. Test Matrix

Anchor Type	Spec. No.	Test Site	Nom. Edge Dist. (in.)	Time of Installation	Comments
1-AP	1	37	1/4 (*)	normal	
1-AP	2	52	1/4 (*)	normal	
1-AP	3	55	1/4 (*)	normal	
1-AP	4	19	0	normal	
1-AP	5	35	0	normal	
1-AP	6	5	-1/4	normal	
1-AP	7	24	-1/4	normal	
1-AP	8	33	1/4 (*)	late	
1-AP	9	49	1/4 (*)	late	
1-AP	10	29	1/4 (*)	late	
1-A	11	51	1/4 (*)	normal	
1-A	12	11	1/4 (*)	normal	
1-A	13	44	1/4 (*)	normal	
1-A	14	7	0	normal	
1-A	15	1	0	normal	
1-A	16	40	-1/4	normal	
1-A	17	47	-1/4	normal	
1-A	18	22	1/4 (*)	late	
1-A	19	32	1/4 (*)	late	
1-A	20	18	1/4 (*)	late	
HFR-HPA	21	12	1/4 (*)	normal	
HFR-HPA	22	43	1/4 (*)	normal	
HFR-HPA	23	50	1/4 (*)	normal	
HFR-HPA	24	53	0	normal	
HFR-HPA	25	38	0	normal	
HFR-HPA	26	20	-1/4	normal	
HFR-HPA	27	16	-1/4	normal	
HFR-HPA	28	26	1/4 (*)	late	
HFR-HPA	29	27	1/4 (*)	late	
HFR-HPA	30	36	1/4 (*)	late	

Table 2. Test Matrix (continued)

Anchor Type	Spec. No.	Test Site	Nom. Edge Dist. (in.)	Time of Installation	Comments
FH15C	31	54	1-1/8 (*)	normal	
FH15C	32	15	1-1/8 (*)	normal	
FH15C	33	17	1-1/8 (*)	normal	
FH15C	34	56	5/8	normal	
FH15C	35	30	5/8	normal	
FH15C	36	8	0	normal	
FH15C	37	59	0	normal	
FH15C	38	34	1-1/8 (*)	late	
FH15C	39	4	1-1/8 (*)	late	
FH15C	40	9	1-1/8 (*)	late	
FH45A	41	60	0 (*)	normal	#3x12" rebar
FH45A	42	21	0 (*)	normal	#3x12" rebar
FH45A	43	45	-1/4	normal	#3x12" rebar
FH45A	44	58	-1/4	normal	#3x12" rebar
FH45A	45	39	-1/2	normal	#3x12" rebar
FH45A	46	2	-1/2	normal	#3x12" rebar
FH45A	47	41	0 (*)	normal	no rebar
FH45A	48	3	0 (*)	normal	no rebar
1-AC	51	46	1/4 (*)		welded
1-AC	52	28	1/4 (*)		welded
1-AC	53	14	1/4 (*)		welded
1-AC	54	13	1/4 (*)		welded
4-AC	61	10	1/4 (*)		welded
4-AC	62	23	1/4 (*)		welded
4-AC	63	25	1/4 (*)		welded
4-AC	64	31	1/4 (*)		welded
HFR-HWA	71	42	1/4 (*)		welded
HFR-HWA	72	6	1/4 (*)		welded
HFR-HWA	73	57	1/4 (*)		welded
HFR-HWA	74	48	1/4 (*)		welded

(* indicates design edge distance)

cast-in-place anchors. Inspection of the finished girder by one anchor manufacturer's representatives led to an observation that the concrete must have been significantly stiffer than his past experience (with precasters in other states). The design slump was 5 in., and the measured slump was 4.75 in. The design concrete strength was 6150 psi, and measured compressive strengths ranged from 6920 psi at release to 10,266 psi at 171 days. Figure 1 shows the measured strengths as a function of age.

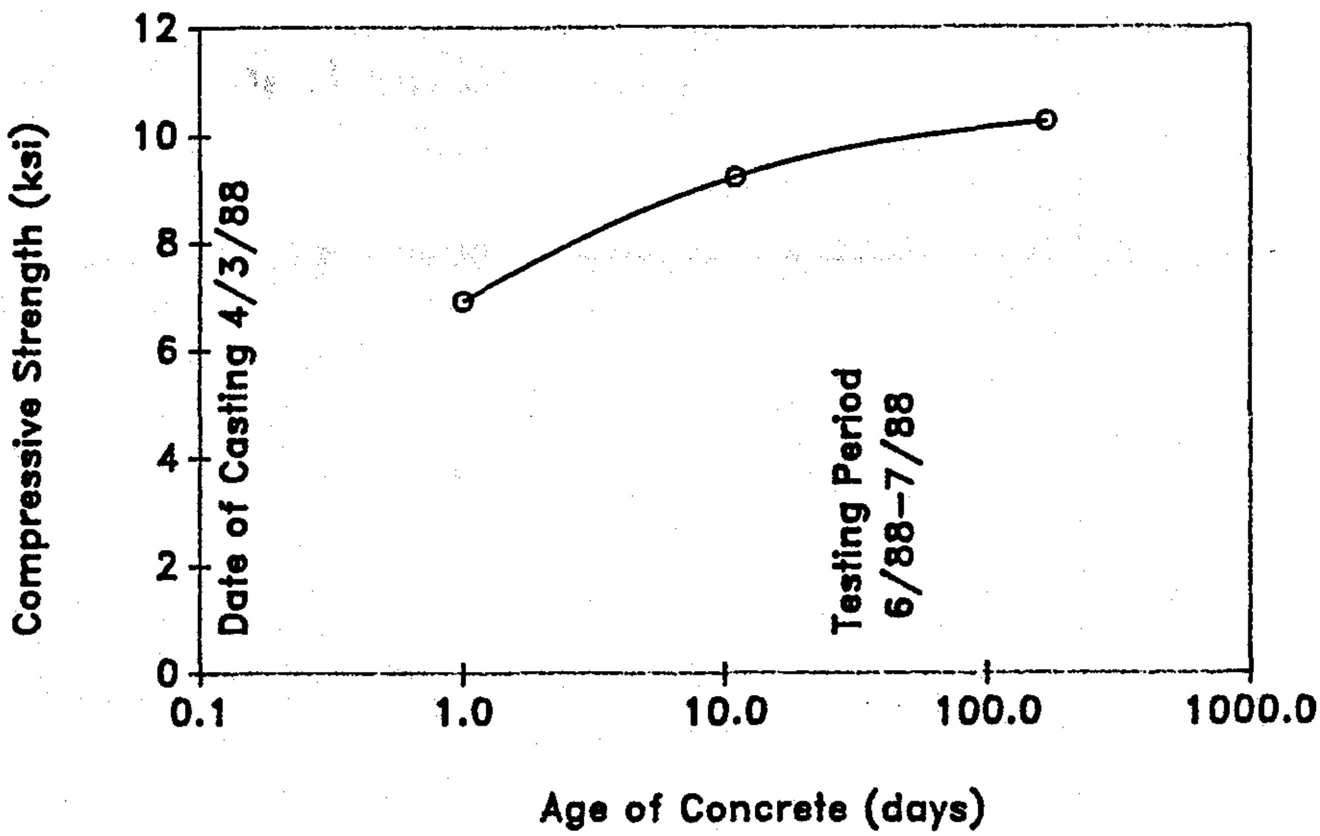


Figure 1.--Strength of Concrete in Test Girder

The girder with cast-in-place anchors was transported to College Station, Texas, where the welded anchors were installed by arc welding to the "R" bars. Manufacturer's instructions for welding the respective anchors, as described in the manufacturer's catalog, were provided to the welder. Specifically, the Richmond HFR-HWA was welded with a fillet weld, and the Dayton 1-AC and 4-AC were welded with flare-V-Groove welds. The welder was instructed to position the anchors for a 1/4 in.-setback from the edge of the prestressed girder, and subsequent inspection indicated that the actual placement was accurate.

Testing of the anchors was carried out in the TEES Civil Engineering Structures Testing Laboratory, on the campus of Texas A&M University. A test fixture was fabricated to position a 22 kip capacity MTS servo-hydraulic actuator at a 45° angle below the horizontal to simulate the load applied to the anchor by the form hanger.

EXPERIMENTAL RESULTS: CAST-IN-PLACE ANCHORS

Figures 2 through 6 present the measured loads for first yield, plotted as a function of actual edge distance. Also included in each figure is a linear regression fit of the data points representing normal installation strength as a function of edge distance. Due to the expected scatter in the results, and especially noticeable in those tests where concrete spalling contributed to the deflection, the data in some cases cannot satisfactorily be modelled by a straight line. Table 3 presents the r^2 value for each regression, a measure of the quality of the fit.

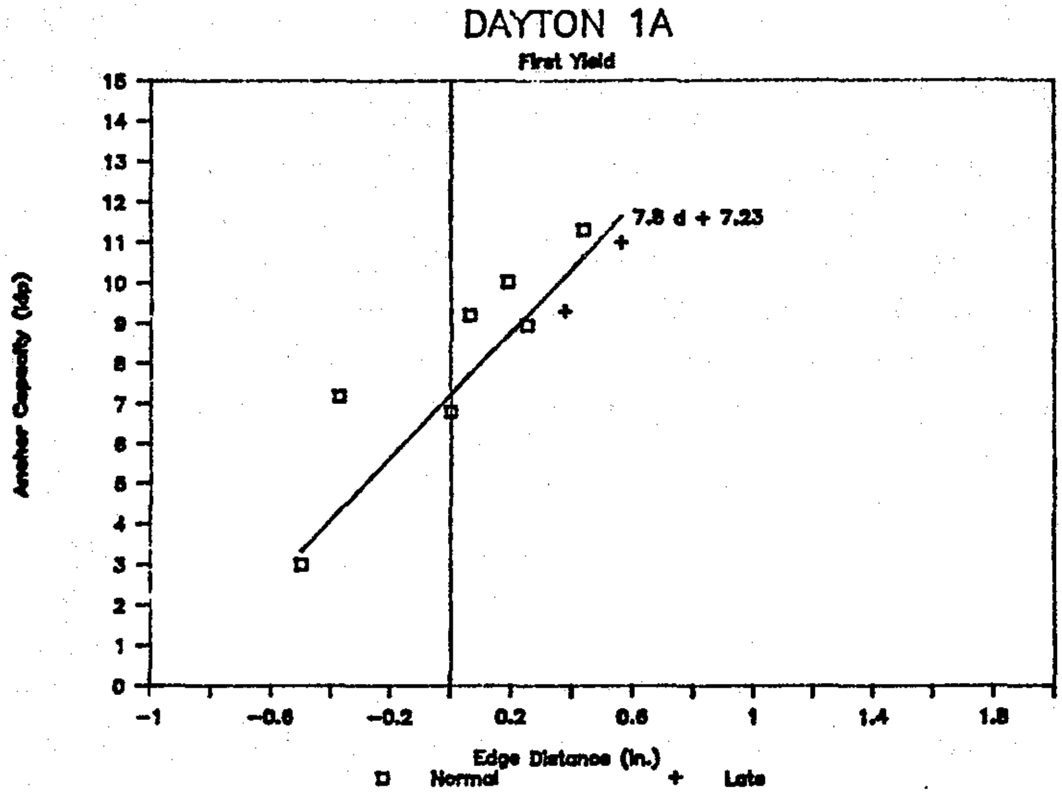


Figure 2.--Observed Yield Loads; Anchor Type 1-A

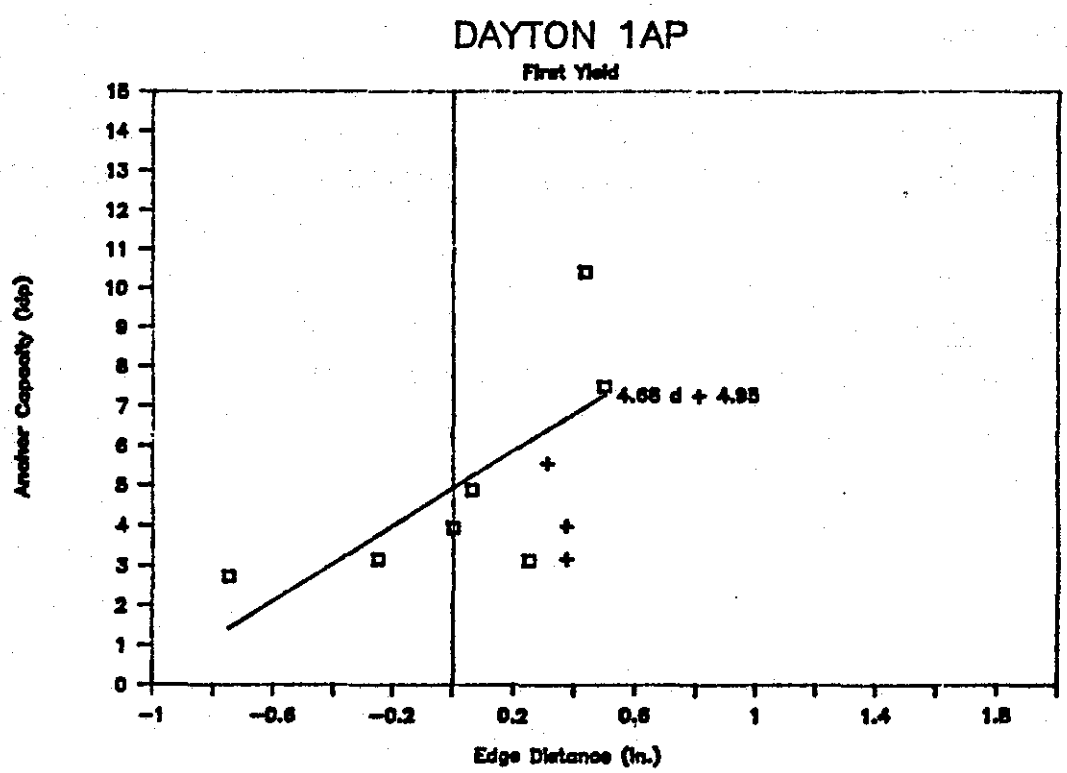


Figure 3.--Observed Yield Loads; Anchor Type 1-AP

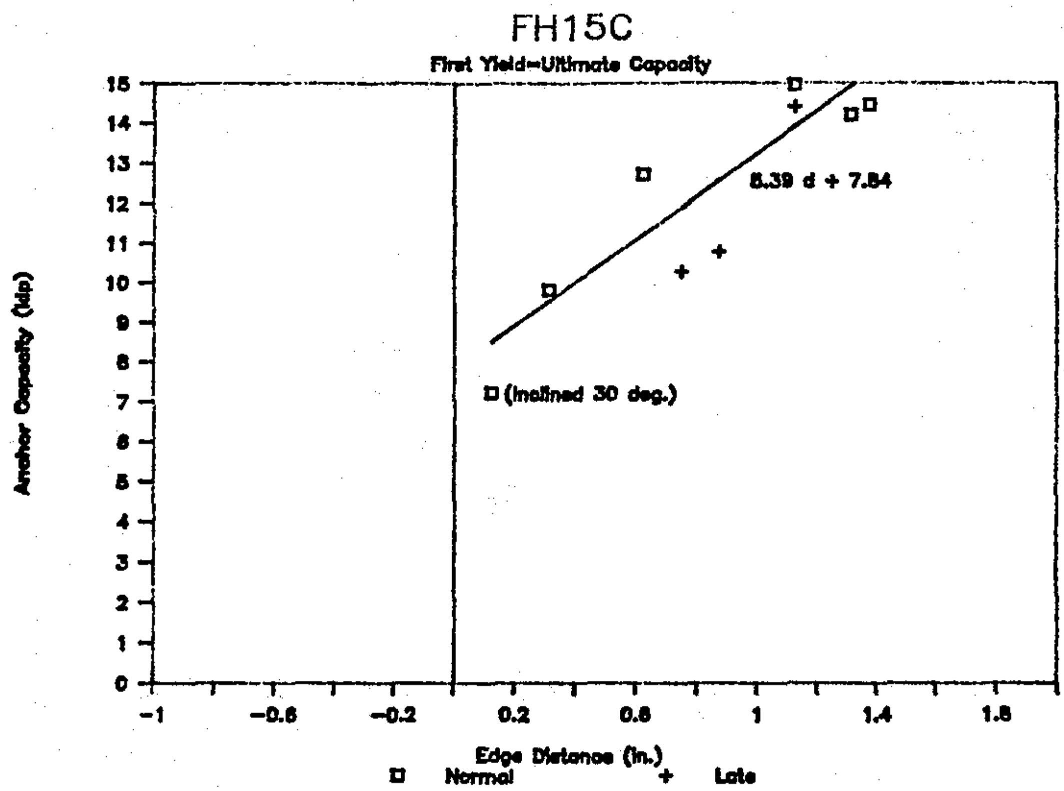


Figure 4.--Observed Yield Loads; Anchor Type FH15C

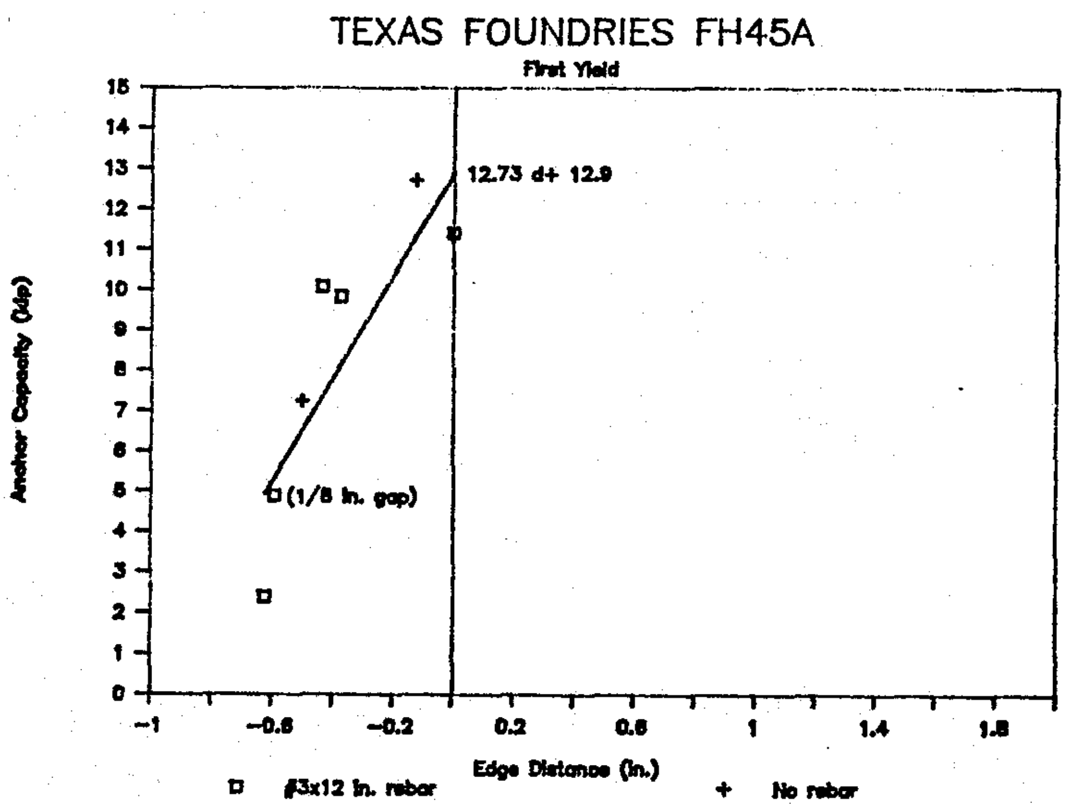


Figure 5.--Observed Yield Loads; Anchor Type FH45A

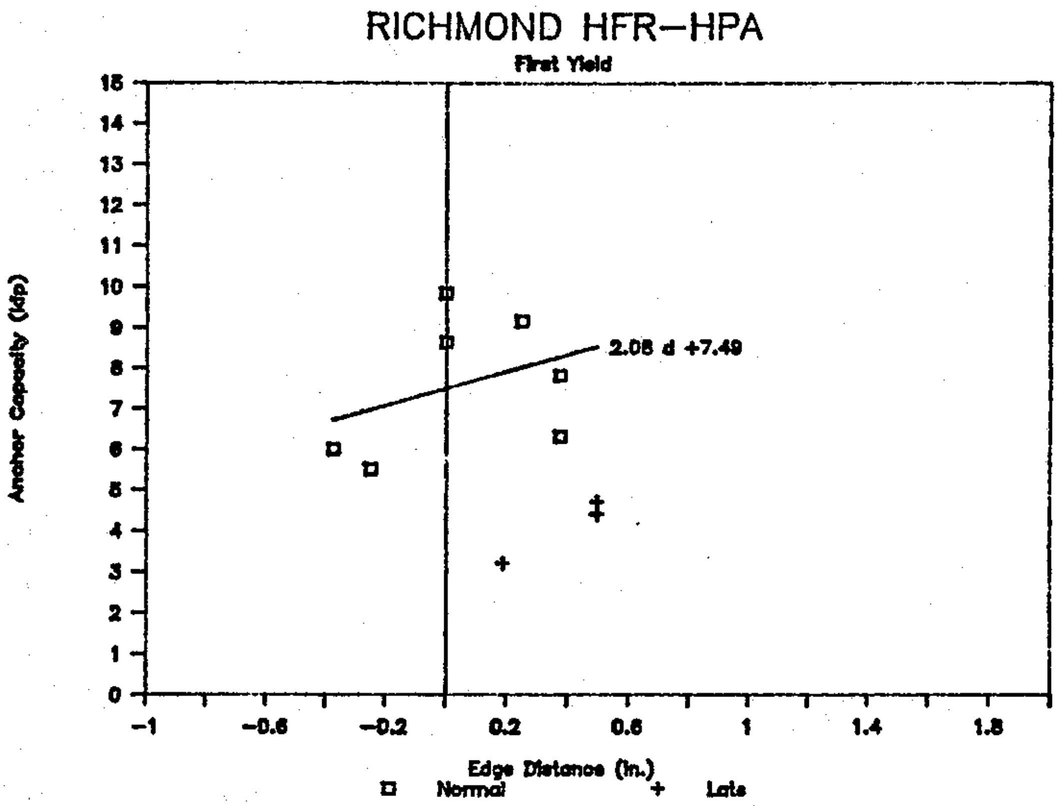


Figure 6.--Observed Yield Loads; Anchor Type HFR-HPA

Table 3. Correlation Coefficients for Yield Strengths

Anchor Type	Dist. Coeff. (kip/in.)	Constant (kip)	r^2
FH15C[1]	5.39	7.84	0.86
1-A	7.80	7.23	0.80
FH45A	12.73	12.9	0.68
1-AP	4.68	4.95	0.51
HFR-HPA	2.08	7.49	0.13

[1]NOTE: FH15C specimens did not exhibit any nonlinear behavior.

Figures 7 through 11 present the measured ultimate loads for the anchors tested, plotted as a function of edge distance. Table 4 presents the correlation coefficients for each regression.

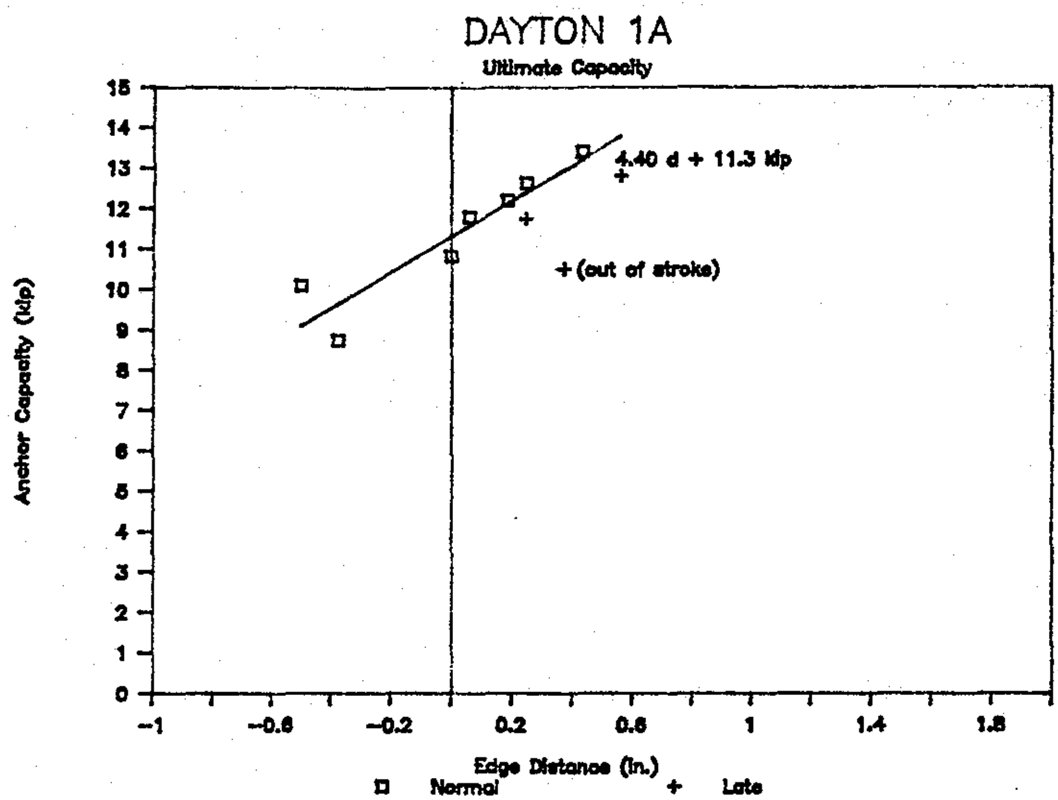


Figure 7.--Observed Ultimate Resistance; Anchor Type 1-A

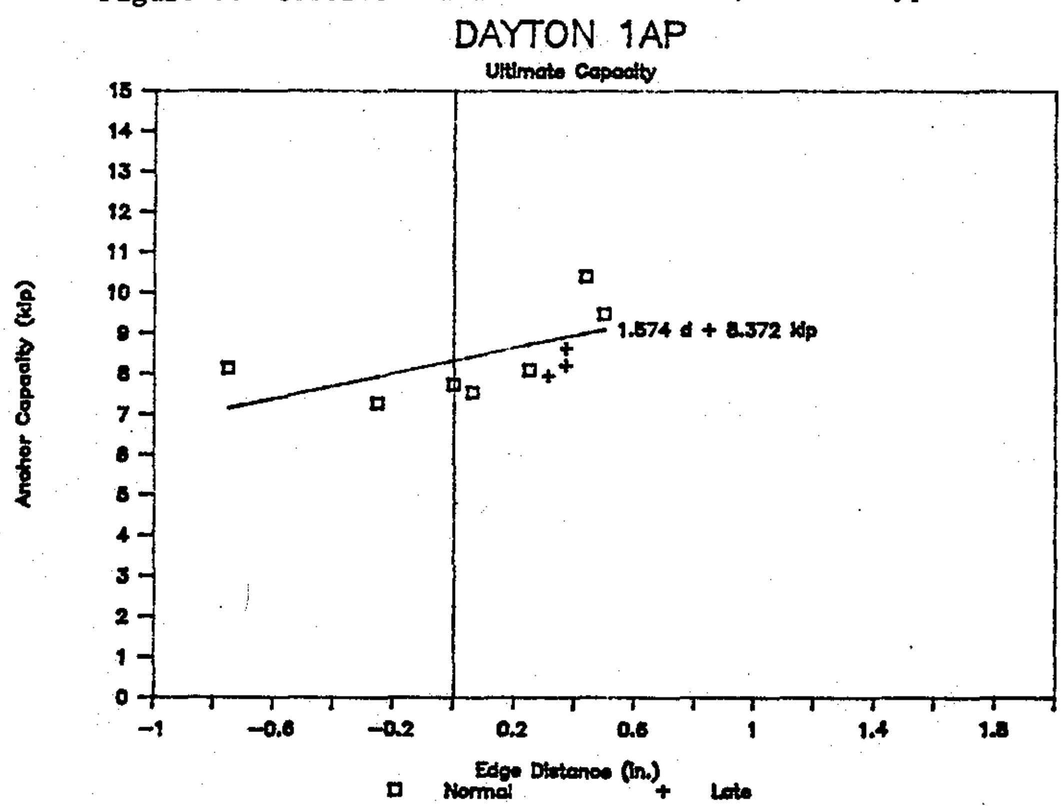


Figure 8.--Observed Ultimate Resistance; Anchor Type 1-AP

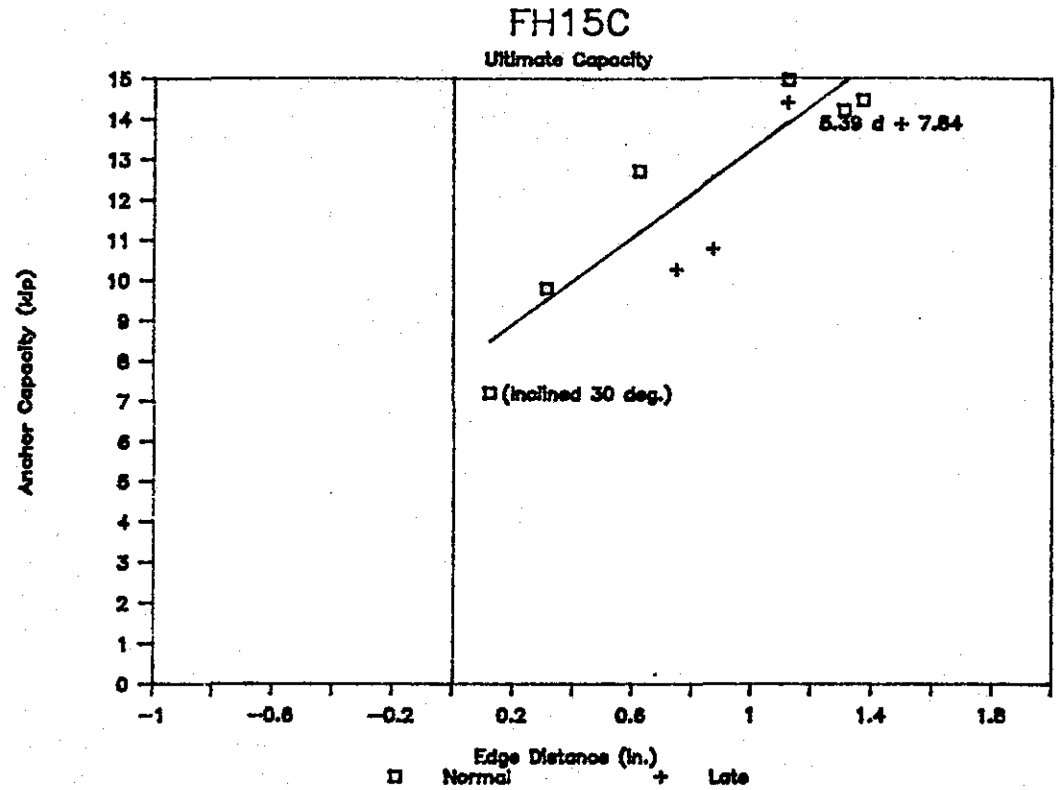


Figure 9.--Observed Ultimate Resistance; Anchor Type FH15C

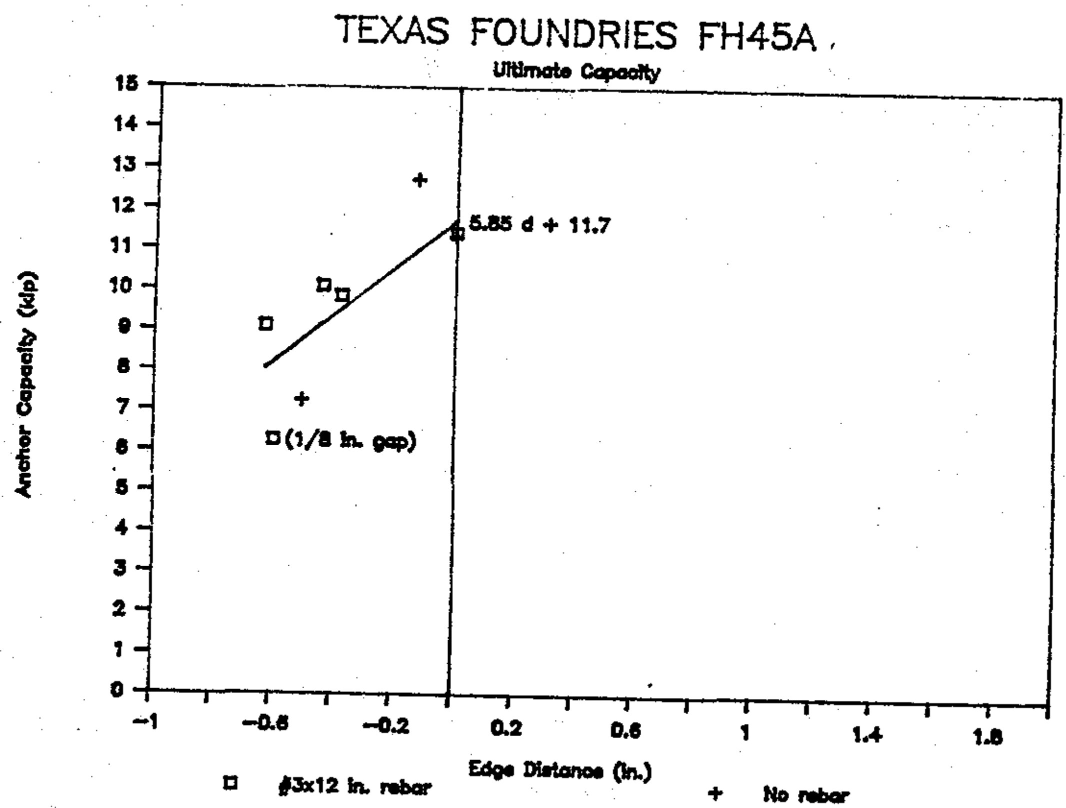


Figure 10.--Observed Ultimate Resistance; Anchor Type FH45A

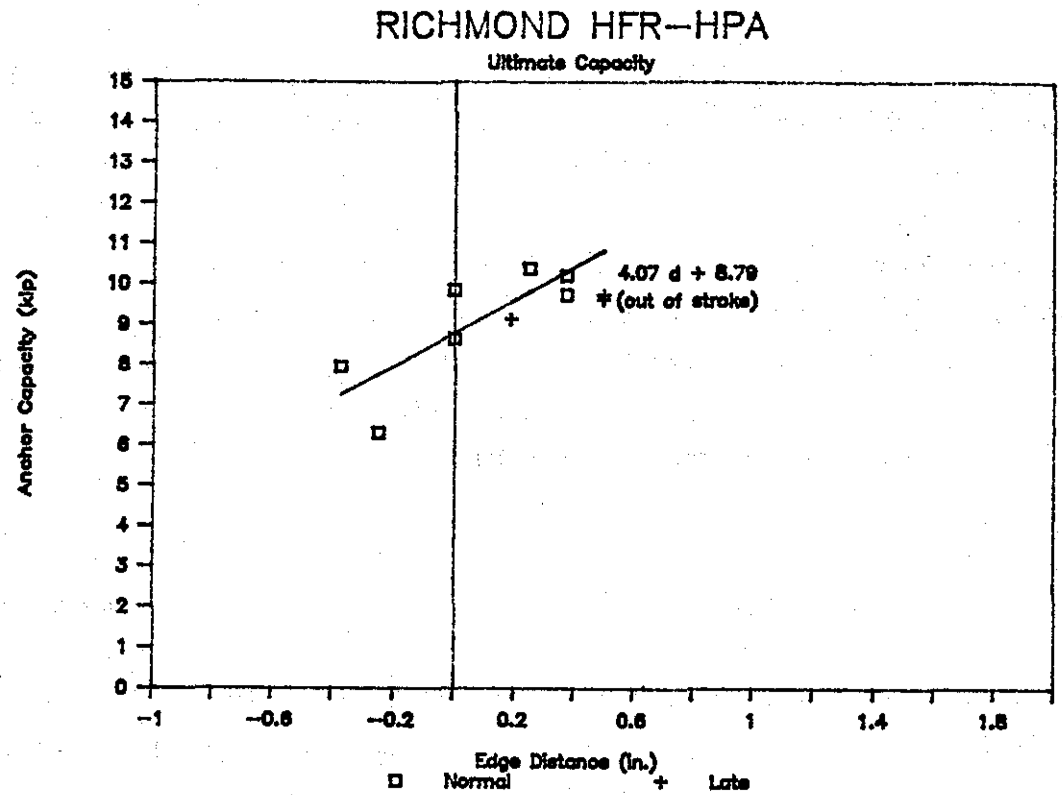


Figure 11.--Observed Ultimate Resistance; Anchor Type HFR-HPA

Table 4. Correlation Coefficients for Ultimate Strengths

Anchor Type	Dist. Coeff. (kip/in.)	Constant (kip)	r^2
FH15C	5.39	7.84	0.86
1-A	4.40	11.3	0.85
HFR-HPA	4.07	8.79	0.66
FH45A	5.85	11.7	0.58
1-AP	1.57	8.37	0.35

All these cast-in-place anchors failed at loads above twice the manufacturer's rated safe load, when installed according to the manufacturer's instructions.

EXPERIMENTAL RESULTS: WELDED ANCHORS

In addition to the tests of cast-in-place anchors, tests of three types of welded anchors were conducted. These anchors, Dayton 1AC, Dayton 4AC, and Richmond HFR-HWA, are designed to be welded to the hairpin bars, or "R" bars as they are called out on the Department's standard drawings. These reinforcing bars are provided on 2-ft centers, except near the ends of the girder, where they are spaced 1 ft apart, to provide shear force transfer between the slab and the girder for composite action. It is common practice for form-hanger anchors of various types to be welded to these bars.

Figures 12, 13 and 14 show the test results for the three types of welded anchors tested. All anchors were installed according to instructions in the manufacturers' catalogs or literature. No attempt was made to investigate the dependence on anchor position, so variations in test edge distance are random installation errors. Observed variations in anchor strength do not correlate to anchor edge distance, however. Two points are plotted in each figure for each test, corresponding to first observed yield or nonlinearity in the load-displacement output and to the ultimate failure load. The Dayton 1AC and the Richmond HFR-HWA anchors both failed at more than twice the manufacturer's rated safe loads, however the Dayton 4AC anchors generally failed at loads below twice the manufacturer's rated safe load of 6000 lb. The average ultimate resistance exhibited by these anchors was 11.2 kips, however, so the actual factor of safety was 1.87, rather than the 2.0 stated by the manufacturer. The strength was clearly not limited by the field weld--these anchors exhibited failures in the rod, in the manufacturer's spot weld, and in the field weld, with the strengths of the three failure modes all being in the range 10.8-11.8 kips.

DAYTON 1AC

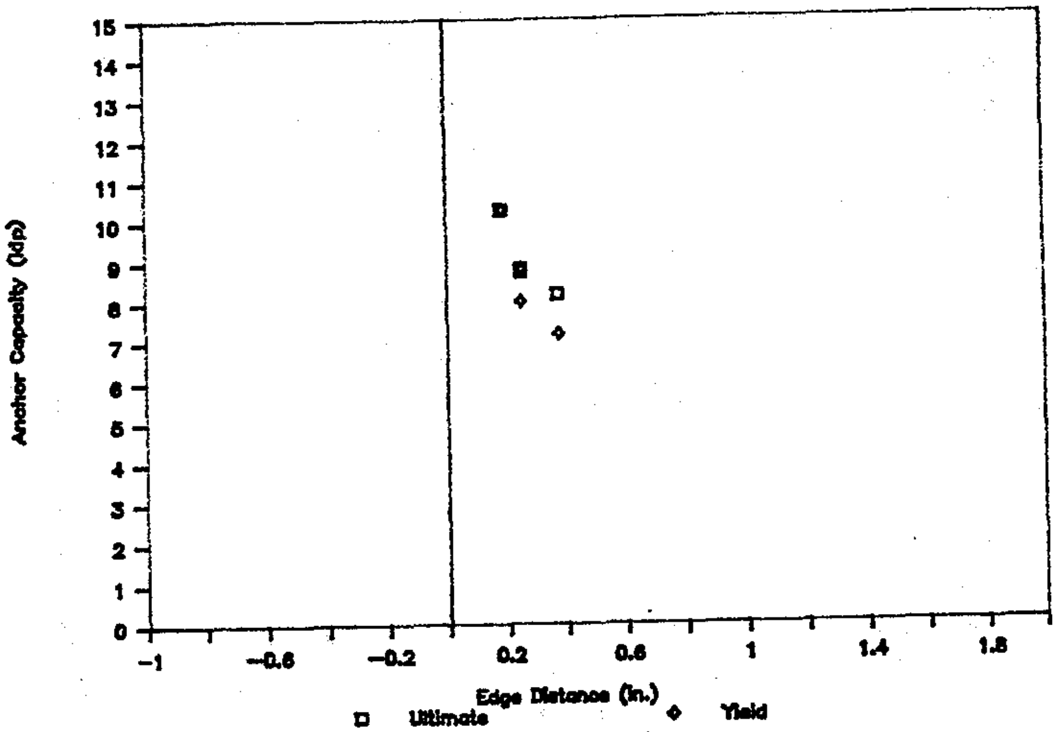


Figure 12.--Observed Resistance--Welded Anchor Type 1AC

DAYTON 4AC

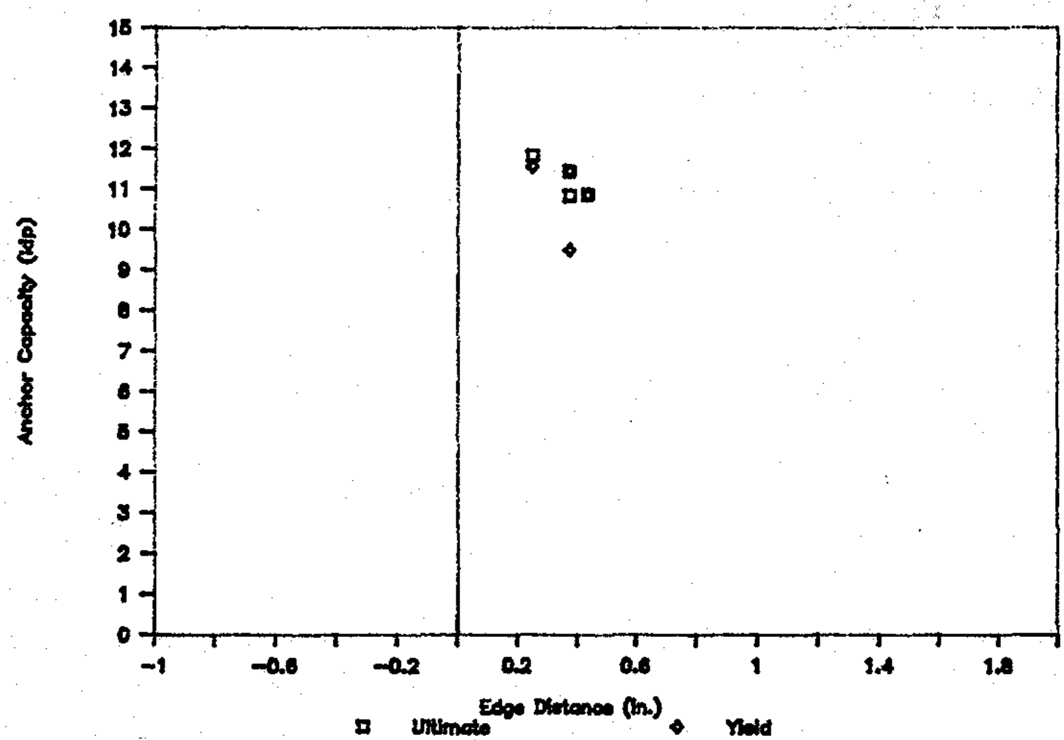


Figure 13.--Observed Resistances--Welded Anchor Type 4AC

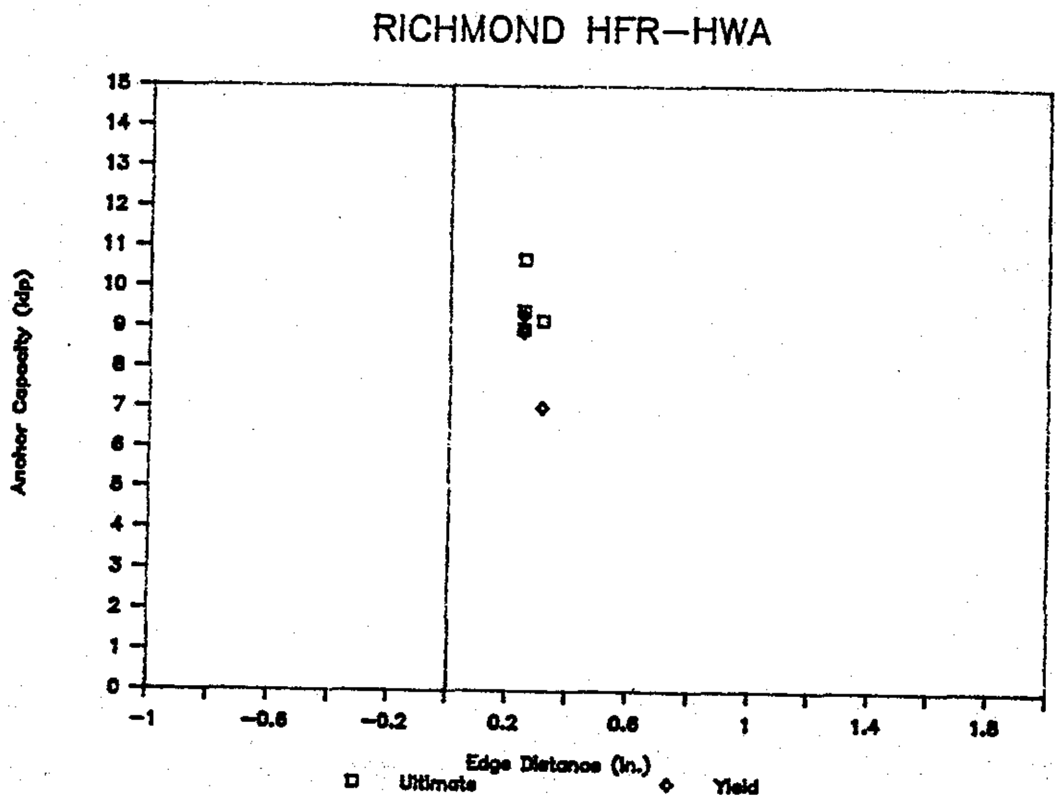


Figure 14.--Observed Resistance--Welded Anchor Type HFR-HWA

OBSERVATIONS FROM EXPERIMENTAL STUDY

Based on the experimental results presented above, the following observations can be made:

1) Failure of the cast-in-place anchors occured in two distinct modes--a relatively brittle rupture of the concrete along a more or less shallow conical failure surface with the apex at the anchor, and a much more ductile failure which included ductile behavior of both the concrete (spalling), the anchor material (yielding and necking), and the anchor geometry (lateral buckling), culminating in rupture of the anchor material, usually at or near a welded connection. The first, more brittle mode was characteristic of the cast steel anchors (FH15C and FH45A), while the fabricated steel anchors (all others tested) exhibited the second, more ductile mode of failure.

2) The data indicates that the cast-in-place anchor capacity is strongly dependent on the quality of the installation; in particular, the edge distance. Improper positioning, with edge distances less than specified by the anchor manufacturer, can result in significantly reduced capacities. For some of the anchors tested, the effects of reduced edge distance can be estimated with reasonable confidence from the results presented, while for others additional data is needed. Time of installation was not observed to have such a serious effect, but the circumstances, ie. ideal climatic conditions, may not have allowed a thorough test of this factor.

3) The cast-in-place anchors all exhibited strengths in excess of twice the manufacturer's stated safe load. Those anchor type exhibiting brittle failure modes failed at nearly four times their manufacturer's rated safe load. The FH15C is rated at 3200 lb, and the linear regression of the data indicates a strength of approximately 13.9 kips, or 4.3 times the rated strength at a 1.125 in. edge distance. The FH45A is designed for flush installation and exhibited an average strength of approximately 11.7 kips, or 3.7 times the rated strength.

4) The welded-in-place anchors exhibited strengths in excess of twice the manufacturers' stated safe load, with the exception of one type, which had a strength only slightly less than this value. Failures of these anchors involved minor spalling of the girder, yielding of the anchors, necking of the anchor rods, and ruptures of the field welds, the rods, and the shop welds. The most significant observation is that considerable buckling and large deformation of the cold-formed sheet components common to these anchors almost always occurred during these failures, which were also accompanied by some minor spalling of concrete.

DESIGN RESISTANCE: EXAMPLE CALCULATIONS

The designer of form-hanger anchors is faced with a task for which there is little guidance in print. While it is beyond the scope of this report to present a detailed discussion of the methods of analysis, some attention to the topic is appropriate, especially since there are several complicating and confusing aspects to the problem of sizing and spacing anchors, which may not be anticipated by designers. It should be pointed out that the design methods addressed in the following examples have been chosen to illustrate some of these complicating aspects.

The AASHTO specifications ("Standard Specifications for Highway Bridges") can be applied in principle to the design of the anchors considered. For example, the anchors failing by rupture of concrete have capacities that are limited by the shear strength of the plain concrete. Application of the AASHTO allowable stress design method requires calculation of the stress distribution in the anchor and in the concrete around the embedded anchor. The AASHTO load factor design method is simpler to apply for these anchors, using the simplifying assumption that the tension in the inclined hanger rod is resisted by a uniformly distributed shearing stress over the inclined failure surface. AASHTO section 8.16.6.2 gives the calculated resistance of plain concrete in shear to be

$$V_c = 2\sqrt{f_c'}\ b_w\ d \tag{1}$$

...where V_c is the shear resistance (lbs), f_c' is the cylinder compressive strength (psi), and (b_w d) may be taken here to be the area of the failure surface. A description of a representative failure surface is given in Figure 15. Analysis of similar data for several anchors indicated that the failure area (b_w d) is in the range of 95-130 in^2. This shear resistance will be reduced by a resistance factor of 0.85. That is,

$$V_u \le \phi\ V_c = 0.85\ V_c$$

...where V_u is the calculated ultimate shear force on the failure surface. The loading on the failure surface is approximately equal to the tension in the 45^o anchor rod, if the failure surface is assumed to be inclined approximately 45°. The actual failure surfaces may be at different angles; failures in the tests conducted as a part of this study exhibited failure angles of approximately 36° from vertical, as is illustrated in Figure 15. The appropriate load factors γ and β must be identified from AASHTO section 3.22,

$$\gamma = 1.3,$$
$$\beta_D = 1.0, \text{ and}$$
$$\beta_{L+I} = 5/3 = 1.67$$

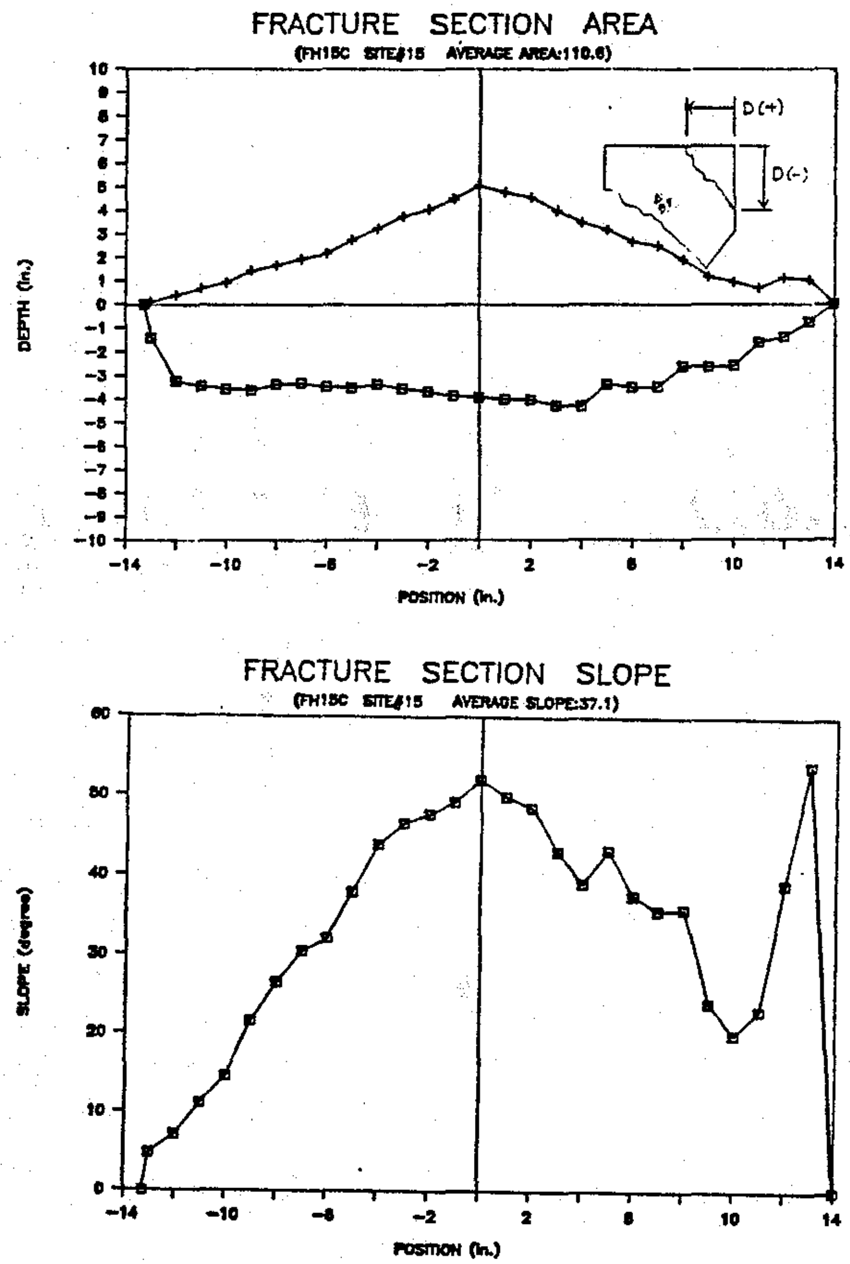


Figure 15.--Description of Concrete Failure Surface

The factored tension in a 45° hanger rod is approximately

$$\sqrt{2}\ \gamma\left[\beta_D DL + \beta_{L+I}(LL+I)\right],$$

...where DL is the dead load, LL is the live load, and I is the impact load. The impact load may be chosen to be some fraction of the live load. If the dead load is taken to be the weight of the formwork, and the weight of the plastic concrete plus the screed and the workers is considered live load, a conservative factored load will result. Probably the weight of the plastic concrete can be estimated better than the truck loads which were the basis for the chosen load factors. A less conservative factored load could be obtained by considering the weight of the plastic concrete to be dead load. Since the plastic concrete is sometimes piled up significantly higher than the final design thickness of the deck, a factored load calculated using the plastic concrete as dead load might be unconservative. The impact fraction clearly must multiply the weight of the plastic concrete, which implies the weight of the plastic concrete should be considered live load. Furthermore, there are no guidelines in the specifications for determining the appropriate impact fraction. In short, the load factors defined in AASHTO section3.22 and impact fraction defined in section 3.8.2 may not be appropriate for calculating the loads on the anchors.

Design procedures typical to the anchor industry are more simplified. As an example, consider the design shown in Fig. 16. The anchors support a 54-in. cantilevered deck of varying thickness which averages approximately 11 in., a formwork plus construction crew load estimated to be 50 psf uniformly distributed over the 6-ft tributary area, and the concentrated loads of the screed wheels. Various screed geometries and weights may be used. The screed in this example has two wheels on either side carrying 1170 lb/wheel. The resultant gravity loading on the bracket, its location, D, and the tension in the 45°inclined anchor hanger rod, are tabulated in the Table 5.

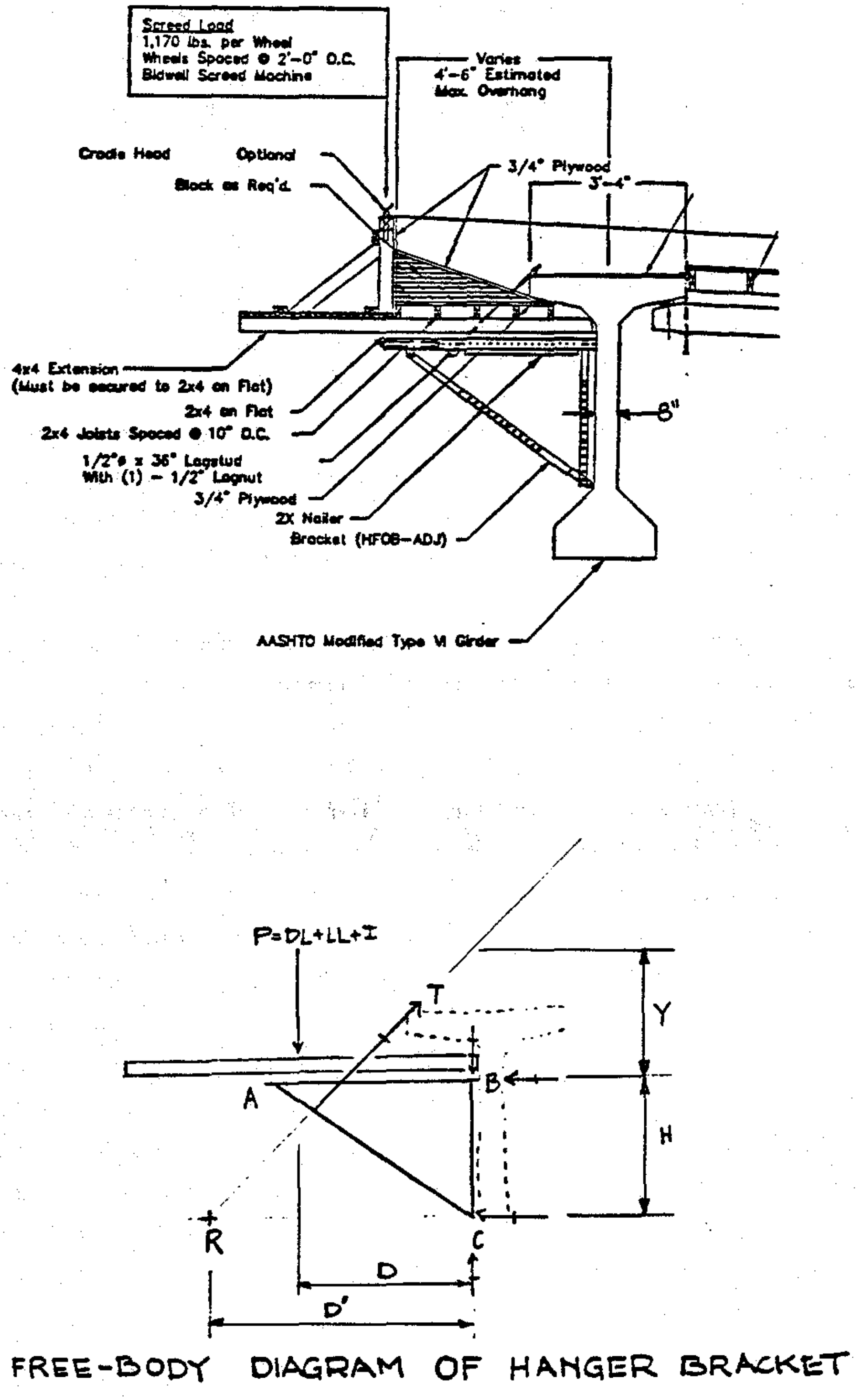


Figure 16.--Illustrative Anchor Design Example

Table 5.--Resultant Gravity Load on Hanger in Example Problem

S (ft)	Resultant, R (lb)	D (in.)	$\sqrt{2}$ R (lb)
2	3008	31.1	4254
3	4317	31.0	6105
4	5431	30.7	7680

Assuming a manufacturer's rated safe working load of 6000 lb in the hanger rod, based on a 12,000 lb ultimate capacity and a factor of safety of 2.0, it can be seen that spacings of 3 ft and greater are not safe for this example; therefore a 2-ft spacing would be selected.

It should be pointed out that the calculations above neglect any vertical force from the reaction on the bracket at points B and C, where the bracket bears on the girder web. In fact such forces will be present and may require consideration. In this example the calculated resultant distance D is less than the horizontal distance D' from C to the line of action of the inclined hanger rod tension T. In this case the reactions required for stability will include an upward reaction on the bracket at point C (or some upward frictional resistance at point B), which will reduce proportionately the vertical component of the tension in the anchor rod. It is conservative in this example to neglect the upward forces at B and C. In the case when D>D', the situation is reversed, and a downward reaction at B is required for stability. Depending on exactly how the reaction at B is provided, a downward reaction may increase the tension in the inclined anchor hanger rod and cannot be neglected when checking the capacity of the anchor or inclined anchor hanger rod in the case D>D'. The reaction at B or at C is sometimes called a "posting" reaction, since it may be provided for by a post above B or below C to the adjacent flange of the girder.

Alternatively, consider the following design philosophy based roughly on the AASHTO design procedure. Assuming dead loads of the timber formwork and brackets equivalent to 15 psf, and treating the weight of the plastic concrete, the screed, and a 50 psf construction crew load on a 3-ft wide tributary area as live loads, assuming an impact load equivalent to 30% of the plastic concrete plus the crew, and using the conservative simplifying assumption that the form stringers are simply supported at each bracket, the anchor loads can be calculated for any given anchor spacing. The results are tabulated in Table 6 for three anchor spacings.

Table 6.--Example Calculations

Anchor Spacing (ft)	DL (lb)	LL (lb)	I (lb)	(DL+LL+I) (lb)	1.3[DL+1.66(LL+I)] (lb)
2	135	3318	461	3914	8363
3	202	4679	692	5573	11,899
4	270	4830	923	6023	12,815

The resultant tension in the 45° hanger rod will be approximately 141% of these vertical load components, neglecting the effects of any posting reaction. Assuming a manufacturer's specified rated safe working load of 6000 lb, based on a manufacturer's recommended factor of safety of 2.0 and an ultimate strength of 12,000 lb, it can be seen that the computed loads for 3 ft and 4 ft spacings are not safe, and only the 2-ft spacing is adequate--the same conclusion as was reached in the earlier calculations. It should be pointed out here that there is not general agreement among design professionals about appropriate design philosophies for anchorages exhibiting relatively brittle failures of the concrete, as was the case in some of the anchors tested. In fact, there are advocates of an additional multiplicative factor of safety of as much as 2.0 being applied to those anchors failing in a brittle fashion. Ductile behavior of anchors is desirable, for such behavior provides advance warning of impending failure and allows time for removal of loads or protection of personnel to reduce the severity of a failure. Because of this, an increased factor of safety against brittle failure is justified.

The capacities of those anchors exhibiting nonlinear behavior characterized by initial spalling of the concrete might be considered to be limited by the allowable bearing stress under the AASHTO allowable stress design method. AASHTO section 8.15.2.1.3 prescribes an allowable stress for bearing on reinforced concrete to be 0.30 f'_c, unless high edge stresses caused by deflection or eccentric loading occur, in which case the allowable bearing stress is given to be 0.225 f'_c. The anchors tested have bearing areas ranging from 0.16 in^2 to 2.25 in^2. While the actual compressive strength of the concrete in the test girder exceeds 10,000 psi, the AASHTO specifications (section 9.15) do not ordinarily allow the use of design strengths greater than 6000 psi for prestressed concrete. Using f'_c=6000 psi and the range of bearing areas mentioned earlier, the resulting allowable tension in a 45^o inclined hanger rod is 407 lb to 5730 lb. Those fabricated hangers without a bearing plate will almost certainly not satisfy the AASHTO allowable stress design method specifications in this regard, so that the use of the allowable stress design method will limit the safe capacity of such anchors, sometimes to very low loads. When adequate bearing surface is provided, premature bearing failures and spalling are not expected, and the AASHTO load factor design method may be used. Calculation of the ultimate capacity of the anchors is made difficult by the variety of failure modes which come into play, including yielding of the anchor components, rupture of the anchor components or of the welded connections, lateral buckling of the anchor, or excessive deflections of the anchor due to changes in geometry. From the test data presented here, the ultimate capacity of the various anchors tested may be identified. Application of a suitable ϕ factor to the measured ultimate capacity and comparison of the factored capacity with the factored loads allows a check of the safety of the anchor.

CONCLUSIONS

1. All anchors tested have adequate strengths to be used safely in a form-hanging system when appropriate engineering judgement is used, and when appropriate engineering data is available to the

designer and fabricator. In every case but one, the observed strengths exceeded twice the manufactur's rated safe load, and in that case the strength was 1.87 times the manufacturer's rated safe load.

2. The analysis of the loading on an individual form-hanger bracket requires careful and detailed consideration of the methods of assuring hanger stability.

3. Failure of the installed anchors may be ductile, as observed in the case of fabricated steel anchor installations, or brittle, as observed in the case of the cast steel anchor installations. Any significant unanticipated deflections of the cantilevered hangers observed by the inspector during construction might indicate impending collapse. In the case of the anchors exhibiting ductile failure, such deflection might be as much as several inches above that anticipated, while for the anchors exhibiting brittle failures, much smaller deflections can be expected at failure. For this reason, a brittle failure mode is not desirable. The consequences of inadvertent overloading are often lessened when large deflections preceed collapse.

   In either case it is possible that the failure may be progressive and catastrophic, that is, failure at one anchor may cause adjacent anchors to fail, or the failure may be arrested, depending on the details of the loading. An important factor tending to prevent progressive and catastrophic collapse is deflection of the anchor system--the more flexible each anchor is, the more anchors tend to share a local overload such as might be caused by the failure of one anchor. Those anchors failing in a ductile fashion also exhibited greater deflections and increased flexiblity. It is expected then, that anchors failing in a brittle fashion are more likely to lead to progressive and catastrophic collapse than are anchors failing in a ductile fashion. The potential of catastrophic failure should be considered when selecting a safe working load for a given anchor.

For these reasons, a higher factor of safety is recommended for anchor systems failing in a brittle mode, compared with anchor systems exhibiting sufficient ductile deformation, such that readily visible formwork deflections preceed failure. Those anchors which exhibited a brittle failure mode also exhibited an ultimate strength greater than four times the manufacturer's rated safe load.

4. Impact loading is typically not considered in the design of the anchor spacing. Impact is minimized by some contractors by placing the plastic concrete on the outside girder and screeding or shovelling it out onto the cantilevered overhang. It is believed that dumping of plastic concrete from a bucket onto cantilevered form-hanger brackets may cause large enough dynamic load factors to cause failure of one or more anchors.

RECOMMENDATIONS

1. Until specifications are available for the inspection of form-hanger anchors, inspectors should at least obtain descriptions of proper installation procedures from the manufacturers and use engineering judgement. Anchors which deviate significantly from manufacturers' specifications regarding edge distance should not be used, or should be derated. Anchors whose strengths are known with some confidence to be linearly dependent on edge distance may be derated using the data presented in this report, other factors being equal.

2. Calculation of anchor spacing should be accomplished and sealed by a registered professional engineer, at the direction of the contractor, and his calculations should be made available to the Department's inspectors. The engineer's calculations should indicate the brand and type of anchor, the manufacturer's rated safe working load at the intended angle of loading, the specified edge distance, and any specified welding procedures. The

engineer's calculations should specify the minimum anchor spacing, requirements for providing vertical reactions at the bearing points of the hanger bracket against the prestressed girder to guarantee vertical stability, and the calculated maximum load in the inclined hanger rod. The calculations should also indicate the anticipated deflection of the form-hanger brackets at the outside edge of the cantilevered slab under the load of the plastic concrete. The engineer's calculations should reflect an increased factor of safety when brittle failure modes are anticipated, to reflect the increased hazard of catastrophic collapse, unless data exists allowing confident prediction of ultimate deflections which are large enough to be accurately monitored in the field. Sample calculations are included in Appendix B.

3. The draft specification submitted as Appendix A is suggested as the basis of a specification for ultimate adoption by SDHPT to allow systematic inspection and approval of the installed anchors. The draft specification is prepared approximately in the format of ASTM specifications and covers various aspects of selection, procurement, spacing, installation, handling, inspection, and usage of form-hanger anchors which may affect the integrity of the formwork and the safety of SDHPT personnel and any highway traffic below the formwork.

4. Since the form-hanger brackets are inherently unstable with respect to horizontal motion until the formwork is placed, contractors' personnel and inspectors should never step onto any partially completed formwork without safety lines.

APPENDIX A

DRAFT SPECIFICATIONS

D R A F T   S P E C I F I C A T I O N S

For review by the
Texas State Department of Highways and Public Transportation

Standard Specifications for
PREFABRICATED FORM-HANGER ANCHORS FOR PRESTRESSED CONCRETE GIRDERS SUPPORTING CANTILEVERED DECK FORMS

1. Scope

1.1 This specification covers the mechanical requirements for various types of prefabricated form-hanger anchors used for supporting cantilevered deck formwork supports from exterior prestressed concrete bridge girders during construction of the cast-in-place reinforced concrete deck slab. Three general types of anchors are covered in this specification:

1.1.1 *Type 1*--Anchors fabricated from formed steel components, assembled by welding and cast in place in the girder.

1.1.2 *Type 2*--Anchors made of cast iron or cast steel and cast in place in the girder.

1.1.3 *Type 3*--Anchors fabricated from formed steel components which are attached by welding to embedded reinforcing steel projecting from the top surface of the girder.

1.2 The following definitions apply to terms used in the specification:

1.2.1 Owner: The owner of the bridge to be constructed.

1.2.2 Contractor: The contractor employed to erect the prestressed concrete girders.

1.2.3 Contractor's Engineer: A Professional Engineer, registered in the state where the girders are to be erected, retained or hired by the Contractor to design the safe installation of the form-hanger anchors.

1.2.4 Fabricator: The prestressed concrete fabricator employed by the Contractor to fabricate the prestressed concrete girder and install the form-hanger anchors.

1.2.5 Manufacturer: The manufacturer of the metallic components of the form-hanger anchor.

1.2.6 f_c: The design compressive strength (psi) of the concrete in the prestressed concrete girder, which may be taken to be 6000 psi for purposes of designing and testing anchors.

1.2.7 f_c': The actual compressive strength (psi) of 6-in. diameter test cylinders made from the same batch of concrete as the girder, and tested at the time of the placement of the plastic concrete deck on the form-hanger anchors.

2. Applicable Documents

2.1 "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials. Washington, D.C.

3. Ordering Information

3.1 Orders for products under this specification shall include the following:

3.1.1 Quantity (number of pieces of anchors and accessories),

3.1.2 Name of products, including accessories such as threaded rod when desired,

3.1.3 Dimensions, when dimensional choices are made available by the manufacturer,

3.1.4 Structural capacity, the manufacturer's rated safe working load for an installed anchor in a prestressed concrete girder having f_c as specified in 1.2.6, measured as the load in pounds applied at a 45° angle below the horizontal to an installed anchor,

3.1.5 Whether proof load tests are required, and

3.1.6 Any special requirements.

4. Materials and Manufacture

4.1 Steel for Type 1 anchors shall be made by the open-hearth, basic-oxygen, or electric-furnace process, and have a minimum 36,000 psi yield strength, minimum 58,000 psi ultimate tensile strength, and minimum 20 percent elongation in 2-in. gage length or shall comply with ASTM A36.

4.1.1 Specifications for welding of Type 1 components to reinforcing steel bars shall be provided by the manufacturer.

4.2 Iron for Type 2 anchors shall be ASTM A536 cast iron having a minimum 45,000 psi yield strength, minimum 65,000 psi ultimate strength, and minimum 12 percent elongation in 2-in. gage length. Cast steel Type 2 anchor shall have a minimum 45,000 psi yield strength, minimum 65,000 psi ultimate strength, and minimum 12 percent elongation in 2-in. gage length.

5. Mechanical Requirements

5.1 The anchor shall be capable of developing its manufacturer's rated safe load, multiplied by the load factor specified in 5.1.1 or 5.1.2 without permanent inelastic deformation of the anchor or failure of the anchor or of the girder when installed in prestressed concrete girders, in accordance with the manufacturer's installation instructions.

5.1.1 The load factor shall be not less than 2.0 for anchors which have been demonstrated to fail in a ductile fashion, in which significant plastic deformations occur in the anchor which will allow easily visible deflections of the supported form hanger brackets prior to ultimate failure of the overloaded anchor.

5.1.2 The load factor used in shall be not less than 4.0 for anchors which do not meet the ductility failure criteria of 5.1.1.

5.2 Unless the anchor is intended by the manufacturer to be limited for use with certain types of girders, the anchor shall be capable of developing its manufacturer's rated safe load, multiplied by the load factor specified in 5.1 when installed according to the manufacturer's instructions in girders of all types listed, for all concrete compressive strengths f_c' equal to or greater than 6000 psi:

5.2.1 AASHTO types I, II, III, IV, V, and VI,

5.2.2 _______types A, B, C, and D, and

5.2.3 TSDHPT types 48, 54, 60, 66, and 72.

6. Quality Assurance of Mechanical Requirements

6.1 Metallic Components--The manufacturer shall determine through testing that the strength of the metallic components exceeds the installed strength, as defined below. Consideration of yield, rupture, and buckling shall be included.

6.2 Installed Anchors--The manufacturer shall determine the installed strength of the anchors by testing each anchor design according to a test plan that takes into account the following factors:

6.2.1 The actual strength of the concrete in the tests, f_c', which shall be not more than the design value of f_c as specified in 1.2.6, plus a tolerance of 500 psi.

6.2.2 The loading rate, which shall be the time to failure for a linearly increasing load.

6.2.3 Installation tolerances--Installation resulting in critical tolerances shall be considered, except when such installation will interfere with the formwork for the girder, in which case the most critical positioning compatible with the girder formwork may be assumed.

6.2.5 The strength of the installed anchor determined in testing shall be the mean strength of at least three tests.

7. Marking Requirements

7.1 Each anchor or piece of an anchor shall be marked with the following information, in a waterproof fashion, so that it will be legible at all times until the concrete deck is placed.

7.1.1 Manufacturer's name

7.1.2 Manufacturer's anchor model name or number

8. Installation Instructions

8.1 Requirements for proper installation shall be provided by the anchor manufacturer in the form of an *Installation Instruction Sheet* which shall include as a minimum the following information:

8.1.1 The rated safe working load for the anchor, in terms of a 45° inclined tensile force (pounds) in the form-hanger rod, shall be stated for anchors installed in prestressed concrete girders having f_c as specified in 1.2.6. If the anchor is designed for only certain types of girders, the intended application shall be clearly stated.

8.1.2 An orthogonal projection drawing or a perspective drawing of the installed anchor, clearly showing the design position and embedment depth in the prestressed girder and calling out the clear distance between the edge of the girder and an unambiquous reference point on the anchor. This design distance should be expressed in inches, to the nearest 1/4 in. with a tolerance stated to the nearest 1/8 in, and should clearly state that the capacity of the anchor will be reduced if this edge-distance requirement is not satisfied. Any other geometric relationship with the girder or the reinforcing or prestressing steel strands necessary to develop the manufacturer's rated safe load shall be identified on the drawing.

8.1.3 Installation instructions shall state that anchors are to be spaced not more than a maximum spacing to be determined and specified by the contractor's engineer.

8.1.3.1 The contractor's engineer shall not specify a maximum spacing of less than 18 in, unless test results or engineering analysis can demonstrate that failure surfaces for more closely spaced anchors do not overlap, resulting in lesser failure loads than for more widely spaced anchors.

9. Packaging Requirements

9.1 Anchors shall be packaged in packaging systems designed to prevent mechanical and environmental damage which could reduce the installed strength of the anchor. Packaging systems shall consist of an inner container designed to contain a limited number of anchors, and optionally, an outer container designed to contain several inner containers.

9.2 The inner container shall be designed for handling by a single workman, containing a specified number of anchors which in the aggregate do not exceed 25 lb in weight. The inner container shall provide sufficient integrity for handling in a construction site environment.

9.3 The optional outer container shall be designed for shipment, manual or forklift handling as appropriate, and storage. Each outer container shall be marked with the name of the manufacturer and the number and model type of anchors contained, and with any information required for proper shipment and handling. Optionally, the inner container may be designed for these additional considerations, in which case an outer container is not required.

9.4 A copy of the *Installation Instruction Sheet* described in section 8 shall be included in each inner container.

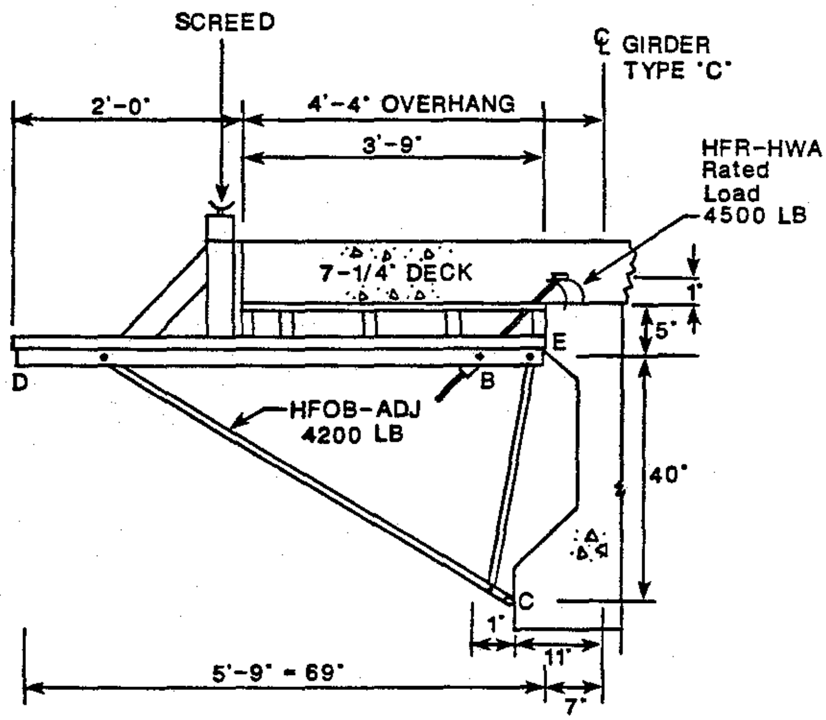
APPENDIX B

EXAMPLE CALCULATIONS

EXAMPLE

Given: 7-1/4" RC Deck
4'-4" Overhang
2-1170 LB Screed Wheels
Formwork as shown

Determine: Anchor spacing, s
Deflection of hanger bracket



Loads on Hanger

Concentrated Loads

Bracket	30 LB	
4x4x6'	25 LB	
Screed	1170 LB $\left(1-\frac{s-2}{s}\right)$	
Total	55+1170 $\left(\frac{2s-2}{s}\right)$	

Unif. Dist. Loads

Formwork	33.7 s	LB/FT
Slab	328 s	
Crew	287 s	
Total	649 s	

s (FT)	Vert. Load on hanger (LB)	Tension in 45° Rod (LB)
2'	2523	3568
3'	3561	5036
4'	4406	6231

$\boxed{\text{Choose 2' Spacing}}$ 3568 LB < 4500 LB OK.
2523 LB < 4200 LB OK.

Deflection Estimate 0.25" defl. at rate load, at bracket end

For 2523 LB; defl. = $\left(\frac{2523}{4200}\right)$ 0.25 = 0.15 inch. (Expected deflection due to elastic deformation of bracket.)

Assume C does not slip: $AC = \sqrt{4^2 + 37^2} = 37.22"$

$AB = 6"/\sin 45° = 8.485"$

$BC = \sqrt{1^2 + 31^2} = 31.016"$

$$\text{define } \beta = \angle ABC = \cos^{-1}\left[\frac{-AB^2+AC^2+BC^2}{2(AC)(BC)}\right] = 9.77°$$

$$\Delta\beta \approx \left[\frac{(AB)}{(AB)\,(BC)(AC)\sin\beta}\right]\delta \quad (\delta = \text{anchor deflection})$$

$$\approx (0.044/\text{in})\,\delta$$

deflection D is approx. $(0.044)(69")\delta \approx 2.99\,\delta$

If $\delta \approx 0.5"$ to $1.0"$; tip deflection $\approx 1.5"-3.0"$

Conclusion

Elastic deflection is expected to be ~0.15"
Ultimate deflection is expected to exceed 1.5"–3.0"

Action

Monitor tip deflections. Investigate deflections ≥ 1.0 in.