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PREFORMED ELASTOMERIC JOINT SEALERS FOR BRIDGES

PHASE I: SUMMARY AND IMPLEMENTATION GUIDE

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16. Abstract

In this summary, a proven effective solution is provided to the problem of sealing joints in bridge decks. At the heart of the solution is the use of specially designed joint armor in combination with currently available preformed elastomeric sealers. The approach recommended is adequate for simple span, composite, concrete and steel structures with span lengths up to 170 feet. The special armored joint system was field tested on three structures. Two of these structures were monitored both manually and with automatic instrumentation to determine the causes and range of magnitudes of bridge end movements; they were also tested for leakage with dyes at periodic intervals over a five year time span. The third structure was used to conduct load tests on the joint armor and armor anchorage components. Application of the study results led to the development of practical procedures for the design and construction of armored bridge joints and for the selection of an appropriate size of preformed elastomeric sealer.

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A. Introduction

In 1965, the then New Jersey Highway Department began an investigation into the behavior of preformed elastomeric bridge joint sealers which, by that time had become a common means of sealing bridge joints in this state. Preliminary findings cited a lack of adequate knowledge on the characteristics of the sealer material and the behavior of bridge joints. In response to these findings a formal research project was launched. The research effort was divided into two phases. The purpose of Phase I, the subject of this report, was to test and to improve upon suggested methods of design and construction of joint systems and to establish relationships between deck temperatures, air temperatures and joint movements. Phase II was to concentrate on the development of realistic acceptance specifications for preformed sealers.

With the issuance of this report, Phase I of the research reaches its formal ending point having accomplished each of the originally established goals. Methods for the design and construction of an effective joint sealing system for bridges have been developed and proven successful. Armored joints, sealed with preformed sealers have been installed on two experimental bridges and have functioned flawlessly for over five years. The particular structures used are considered to typify most highway bridges in New Jersey. Also, the relationship between joint movements and air temperatures for simple span bridges has been determined.

This summary of Phase I of the joint sealing project omits many details of how the research was conducted. Such information, if desired, can be obtained from the project's full report and assigned supplements.[1] It is expected that the reader here is most interested in the project's accomplishments. Accordingly, this summary primarily presents research results and, in particular, those methods for design, construction and sealing of bridge joints that have been found successful for New Jersey highway structures.

B. Scope of Phase I

The scope of Phase I of this project was to test suggested methods of design and construction of joint systems and to establish the causes of joint movements, as well as identify their precise relationships to such movements. For the most part, the research limited its study to two simple span, composite design bridges which possess significantly different joint skews and length to width ratios. Together, these bridges typify the great majority of highway bridges built in New Jersey today. The study included the design, construction and performance evaluation of armored and sawed joints of the two bridges.

The performance evaluation consisted of frequent visual observations; measurement of movements, structure and air temperatures; and utilization of liquid dye tests to locate leakage through joints. Movement and temperature data were obtained by both manual and automated means. The extensive data gathered by the automated method was used to establish the effective temperature of a bridge deck and its correlation to air temperature and to the movements at the bridge's deck joints. The long term stability of these relationships were checked using the manually recorded information.

In order to further evaluate the movement and temperature data, a determination of a concrete deck's thermal coefficient of expansion was also attempted during which the influence of moisture on the thermal coefficient of concrete was isolated.

Although not originally considered in the scope of this project, the research also included a test program involving the load testing of an armored bridge joint. The full scale field tests were intended to provide new light on load distribution and reinforcement stresses to be accommodated in designing joint armor.

C. Conclusions

This phase of the subject research has resulted in the development of procedures for:

1. Design of joint armor,
2. Construction of armored joints, and
3. Sealer selection.

Each of the above aspects has been successfully field tested in excess of five years on two experimental bridges. In a third experimental installation of an armored joint completed in the fall of 1974, the final modification of all procedures was accomplished. The conclusions hereinafter presented are in light of this research.

1. A practical and proven solution to the problem of bridge joint leakage and intrusion is available through the use of a combination of an armored joint and appropriate preformed elastomeric joint sealer.

2. It is essential to recognize the realities of joint design and construction. IN THE ABSENCE OF ADEQUATE QUALITY CONTROL IN CONSTRUCTION NO MATERIAL AND NO METHOD OF ITS APPLICATION WILL SUCCEED. The currently offered procedures require only a LITTLE CARE IN MANUFACTURING AND CONSTRUCTION, FROM WHICH, A TOTALLY SATISFACTORY RESULT SHOULD AND CAN BE EXPECTED. In general, there is no such thing as a foolproof design, but there is also NO REASON why the procedures proposed herein should not yield a complete and satisfactory design.

3. The formed and sawed joint methods of construction evaluated during this research were unsuccessful principally because they required an unattainable quality of workmanship from the contractor. In contrast, the success of the armored joint system can be attributed, in part, to its ability to be prefabricated and then installed within constraints of normal construction practices.

4. For simple span bridges, this research established that the movements of deck ends are affected predominately by temperature changes. The correlation between ambient air temperature and bridge expansion was shown to be linear. The effect of other environmental parameters, such as insolation, precipitation, moisture, etc. or physical characteristics such as creep were found to have no signi-

ficant influence on bridge end movements; hence, the correlation between bridge expansion and air temperature changes is also unique, i.e. for practical purposes it is the only correlation that need be considered.

For the design purposes then, it is the range of ambient air temperature at the particular site that may be assumed to be the "effective" temperature for a bridge. This finding, of course, is consistant with normal design assumptions. For New Jersey, climatological records indicate that this temperature range should be taken as 0° to 110°F.

5. The movements of fixed joints are insignificant although somewhat erratic. This demonstrates the adequacy of New Jersey's bridge bearing design and accentuates the validity of the basic design assumptions. The erratic features, of course, are due to the normal 1/16 inch tolerances which are permitted for bolted connections of metal parts.

6. The displacements at expansion joints, predicted from the air temperature range indicated above, in all probability, will never be exceeded provided the thermal coefficient of expansion of the particular bridge is known fairly accurately. For composite bridges constructed of a steel superstructure with a reinforced concrete deck, the said coefficient lies between the normal coefficient of the steel and concrete. Although the volume of the concrete in such a bridge is significantly greater than that of the steel, the thermal coefficient of the total mass lies closer to that of steel and is

suggested as being 6.3×10^{-6} in/in/°F. Furthermore, there seems to be no reason why the usual average values of the coefficient of expansion for all concrete or all steel structures should not be used. In any case, the thermal coefficients should be taken as linear throughout the temperature range.

7. For bridges with skews of less than 15 degrees and bearing systems as utilized by the New Jersey Department of Transportation, the overall joint movements may be accurately calculated using the general formula $\Delta L = \alpha L \Delta t$ (Eq. #1). The effects of the skew may be neglected.

8. For bridges with skew roughly between 15 and 50 degrees, the joint movements that occur in the direction perpendicular to the joint may be assumed as uniform across the length of the joint. The magnitude of these movements will be less than that predicted by use of the general formula, assuming that "L" is taken as the length of the bridge in the direction of the stringers. As the skew-angle increases and the ratio of the length of bridge to length of joint becomes less than unity, the joint movements in the direction parallel to the joint become substantially larger than those movements perpendicular to the joint. Caution should be exercised to assure that these movements are accommodated and/or minimized, depending upon the joint system in use.

9. In general, the bearing system that is currently standard for New Jersey Department of Transportation bridges is quite effective in controlling and directing bridge displacements.

10. In an 8 to 9 inch thick concrete bridge deck, temperature gradients through the thickness of the deck exceed 20 degrees Fahrenheit at times throughout the year. These large gradients are of short duration and occur primarily due to intense solar radiation. Most of the temperature differential at such times occurs within the one or two inches of concrete at the surface of the deck, and has little immediate influence on the displacement response of the bridge.

11. A general, though guarded conclusion can be made that -- provided there is compatibility of materials -- many of the above stated effects of temperature, or lack of same due to moisture, creep, etc. are transferable; i.e., generally similar effects can be expected in other simple span, composite type bridges and also in total steel and total concrete simple span structures. On the other hand, however, it is apparent that bridge-end displacements are additionally a function of the particular bridge design and are unique for each and every design system. For example, in a cantilever design system the bridge-ends behave differently from those in either a simple beam or continuous beam design. To attempt to lump together the movement characteristics of all bridge designs or to extrapolate from one to another could lead to gross error and would be as logical as claiming that Dutch shoes and Dutch pastry, being both Dutch, are both food.

And so at this point, it is apparent that the selection of the simple span type structure for instrumentation, because of its functional simplicity, served well to isolate the phonomena effecting bridge-end movements.

D. Recommendations

On the basis of the results of Phase I of this research recommendations of a positive nature can now be offered.

1. Procedures for the selection of an elastomeric joint sealer, the sizing of the joint opening, and the design and construction of an armored bridge joint are all detailed in the subsequent pages of this summary. Strict adherence to these procedures will provide leak and intrusion proof joints for bridges up to 170 feet in span length. It is strongly recommended that said procedures be adopted as part of the design standards for all simple span bridges in New Jersey.

2. For bridges with spans exceeding 170 feet in length, it is possible that an effective joint seal could be achieved with either the "modular sealing system" or the "WABO-MAUER" design advocated by S. C. Watson(2). However, no positive recommendation as to their use can be offered. It is expected that NJDOT research study #7784, Experimental Project for Development of Watertight Bridge Deck Joint Seals, and the associated FHWA NEEP study will provide guidance in this regard within the next few years.

3. In design, an ambient air temperature range of the particular construction site should be employed as the "effective" bridge temperature. For bridge joint design in New Jersey, it is recommended that a temperature range of 0°F to 110°F be used.

4. For adequate bridge-end movement control, the bearing system that is currently standard in the New Jersey Department of Transportation bridges is highly recommended.

5. In composite bridges, the thermal coefficient of expansion is most probably closer to steel than to concrete and it is suggested that a value of 6.3×10^{-6} in/in/°F be used to predict end movements. For total concrete or total steel structures, the coefficients of expansion of the specific material should apply and use of handbook values for these materials is recommended.

6. For bridges with skews less than 15 degrees and bearing systems as utilized by the New Jersey Department of Transportation, the effects of the skew should be neglected and the bridge-end movements calculated using the general formula $\Delta L = \alpha L \, \Delta t$ (Eq. #1).

For bridges having a skew roughly between 15 and 50 degrees the same general formula (Eq. #1) should be used, provided the ratio of bridge length to length of joint is equal to, or more than unity. However, if the skew angle is larger or if said ratio becomes less than unity caution is suggested in accommodating deck-end movements. Under these latter conditions, additional allowance must be made for for the effect of significant deck movements that occur parallel to the joint.

7. To assure a reasonable amount of quality control in armored joint installations, it is urgently suggested that the construction and inspection operations bulletins be prepared to define

the critical aspects of construction and plant and field inspection.*

8. Due to the unpredictable behavior of bridge approach slabs, under no circumstances should one attempt to place an armored, elastomeric sealer joint directly between a bridge deck and its approach slab.

9. The application of preformed elastomeric compression sealers in concrete pavement joints of the type (3/4" expansion joints) and spacing (78 feet) employed by this Department is believed to be unwarranted and such use is specifically not recommended.

10. For future research, it is suggested that consideration be given to extending the efforts of this study to varied locations, larger spans, and different types of bridges. The behavior of structures with skew angles larger than 50 degrees and a ratio of bridge length to length of joint less than unity was not quantified in this study and further investigation is warranted.

11. The manner in which a bridge approach slab should tie into a structure and indeed, the performance characteristics of such slabs are in reality unsettled questions. A research study is suggested to identify the warrants for and structural behavior of bridge approach slabs as used in New Jersey.

*The construction procedures provided in details in the Significance of Results section in Reference #1 of this report should be used as the basis for these bulletins.

E. Procedures for Design, Construction and Sealer Selection

1. General

The selection of one specific joint armor design rests on the basis of rather extensive experimentation in construction. It has been established that there is no such thing as fool-proof design, but there is also no reason why a complete and a satisfactory solution cannot or should not be expected, i.e. if at least a little care is exercised in the manufacture and construction of a joint and if the following basic design principles are adhered to:

a. Deck joints must be horizontally straight from outer edge to outer edge and the sidewalk joints are to be directly above in the same straight fashion: main sealers must be placed out to out.

b. Sidewalk sealers are placed also out to out, i.e. bottom of curb to outside of structure with only one vertical shallow bend (60°) at the curb. For illustration see subsequent Figure 3, 4, and 5.

2. Joint Armor Design Procedure

2.1 Basic Design Considerations

In the United States there is no official specification that deals directly with the design of armored joints. Therefore, in order to partially fill this void AASHTO specifications (3) were initially adopted by the researchers for the purpose of establishing loads, load distribution, and impact factors for the design of armored joints. In order to shed more light on this area of design, an armored joint was then designed and constructed, instrumented, and tested for stress-strain determination under load. The information gained from those tests is reflected in the final armored joint design hereinafter presented. It is emphasized that the tests were limited, and to deviate from the 9" x 2" x 1/2" armor angle, or the offered anchorage, would require discreetly exercised engineering. Basically, the important features of the armored joint presented and shown in Figure 1, are as follow:

a. a small top flange to minimize incurred loads;

b. no bottom flange (as would occur with the use of a channel section);

c. top and bottom anchors (as opposed to a single row of anchors);

d. 1/2" minimal thickness of armament to minimize localized deflections;

e. close anchor spacing (to assure that more than one anchor takes the brunt of incurred loads).

The problem of actual stress analysis of this structurally indeterminate system was solved by reasonably severe but safe assumptions based on engineering judgement. Once the effectiveness of the concrete beneath the turned down angle is neglected, a unit length of joint may be rendered statically determinate and further stress analysis is rather straightforward. The size and spacing of anchorage reinforcement follow directly from consideration of the assumed loads, which also reflect the aforementioned field tests. It is the principal researcher's judgement that the joint armor must be designed to carry the full dual wheel load of the AASHTO HS20-44 loading which is sufficiently conservative to assure a safe design.

When the dynamic nature of a wheel load is considered, allowance in the joint armor design must be given to impact and frictional effects that increase the vertical load and create horizontal forces on the armor angle.

Regrettably, in the joint armor tests no dynamic load response could be ascertained, therefore, the true impact and/or friction effects, which must be considered, remained unknown. As a result, the degree of allowance for the effects are left to the designer's descretion. The subsequent design procedures reflect this attitude with an impact factor between 0 and 30% and a coefficient of friction between 0 and 0.80 being permitted. However, in the follow-up example and in the standard drawings shown in Figures 3, 4 and 5 definite practical values for these two parameters are offered.

2.2 Design Loads and Allowable Stressss

Figure 1 gives a schematic representation of how the wheel load and its horizontal friction component is applied to the joint armor. The following data are AASHO[3] specifications for the HS20-44 loading which is considered applicable for this joint armor design procedure:

1. Wheel load. . . . . 16.0 Kips
2. Impact fraction . . . . I = 0 to 30%
3. Friction factor . . . . C = 0 to 80%

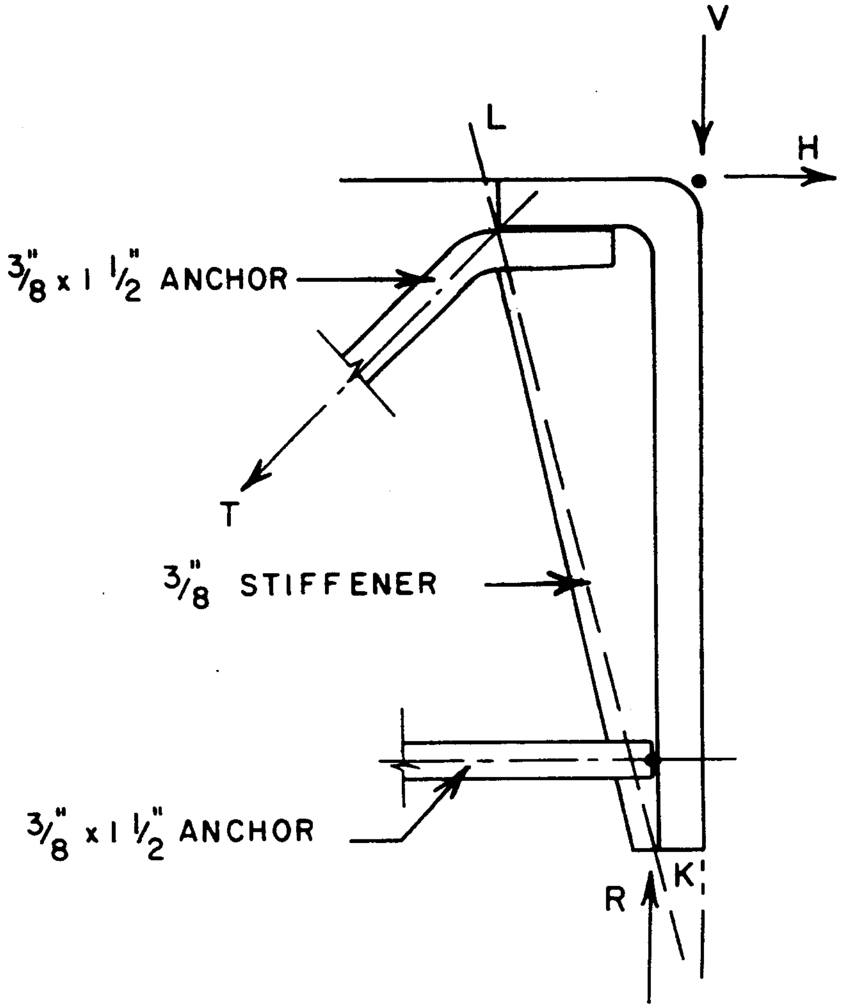


FIGURE 1

PRINCIPAL JOINT ARMOR DESIGN ASSUMPTION

As per the load tests of the armored bridge joints, the load distribution can be assumed as being: E = 4.0 feet. The applied loads will, of course, create stresses in the various armored joint components and the concrete surrounding the joint system. A safe armor design will be one which keeps these stresses below allowable limits. The applicable allowable stresses are given by AASHTO Specifications in Sections 5 and 7 and are as follows:

a. Concrete:

Compression:

f'_c = 3000 lbs/sq. in;

Shear:

$f_{sh} = 0.02\ f'_c$ = 60 lbs/sq. in.;

Bond:

$f_{bond} = 3\sqrt{f'_c}$ = 164 lbs/sq. in.; use 160 lbs/sq. in.;

Bearing:

f_{bear} = 700 lbs/sq. in.

b. Steel (A36):

If, as recommended, A242 or equal steel is used, higher allowable stresses are permitted.

f_s = 20 kips/sq. in.

f_{sv} = 12.0 kips/sq. in.

c. Fillet Welds:

f_{all} = 12.4 kips/sq. in.

2.3 Anchorage Reactions

In Figure 1, the concrete in contact with the steel angle is considered to be ineffective with that portion above line L-K giving no support to the angle. It is quite possible that poor construction practices could produce such a condition. By omitting this concrete from consideration the applied loads are transmitted into the deck only by the upper and lower anchor bars. The field investigations established that it is appropriate to assume the load reactions of these anchor bars to be as given in Figure 1. The magnitudes of the reactions can be computed from straightforward analysis of the static equilibrium conditions.

In the diagrams, formulas and their derivation shown herein, the following notations are used:

V = a vertical load (wheel load)

H = a horizontal load

T = a top anchor reaction

T_H = a horizontal component of T

T_V = a vertical component of T

R = a bottom anchor reaction

R_H = a horizontal component of R

R_V = a vertical component of R

I = impact fraction

C = a friction factor

W_L = a resultant of the shearing stress in the weld

W_R = a resultant of the bearing stress at the end of anchor

P_R = a total allowable load to be carried by cross weld between top of anchor and angle

D = leg size of fillet welds in inches

L = effective length of weld in inches

L_1 = effective length of cross-weld in inches

n = number of anchors per foot length

a = thickness of anchor in inches

b = width of an anchor in inches

A = an area for an anchor in square inches

L_{bond} = effective bond length in inches

L_{bear} = effective bearing length in inches

f_{all} = allowable unit stresses of fillet welds

f'_c = unit ultimate compression strength of concrete

f_{sh} = allowable shearing unit stress of concrete

f_{bond} = allowable bond unit stress of concrete

f_{bear} = allowable bearing unit stress of concrete

f_s = allowable tensile unit stress of steel

f_{sv} = allowable shearing unit stress of steel

f_r = combined unit stress due to shear and moment

f_v & f_h = vertical and horizontal components of combined unit stress

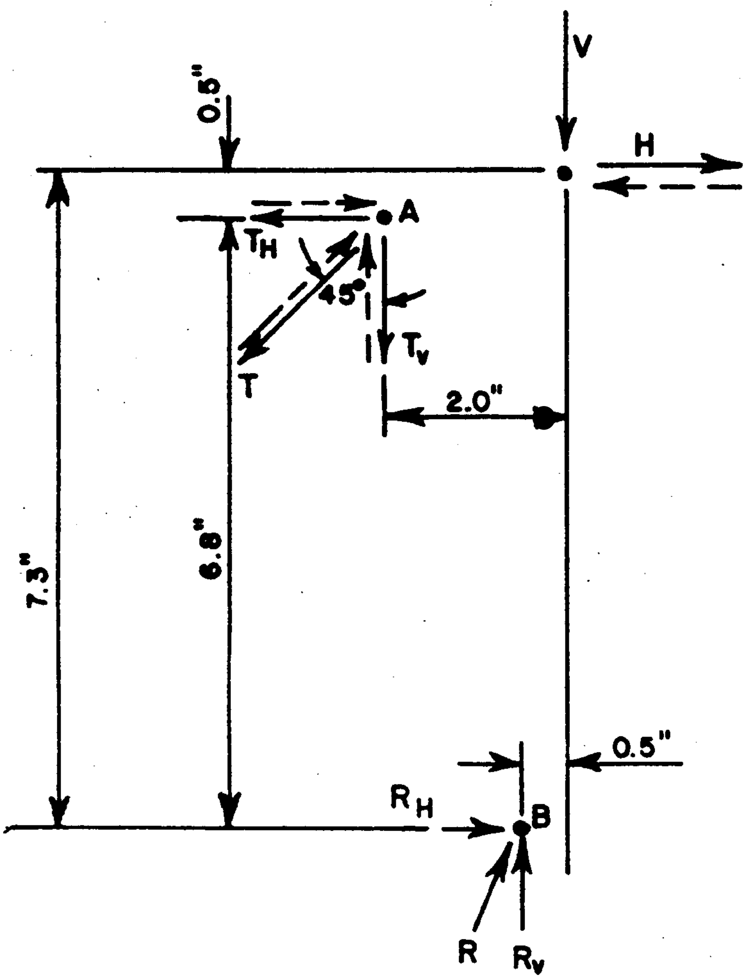
The applied loads are given by the following:

$V = 16.0\ (1 + I)$ (kips/E);

$H = 16.0 \times C$ (kips/E):

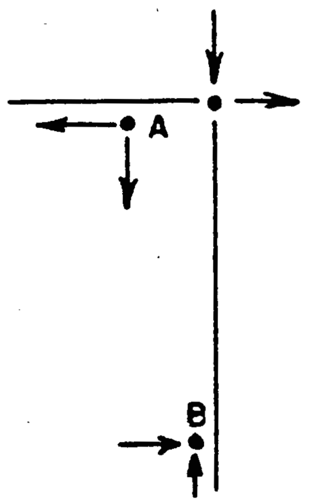
$V = \frac{16}{E}\ (1 + I) = 4.0\ (1 + I)$ (kips/ft.);

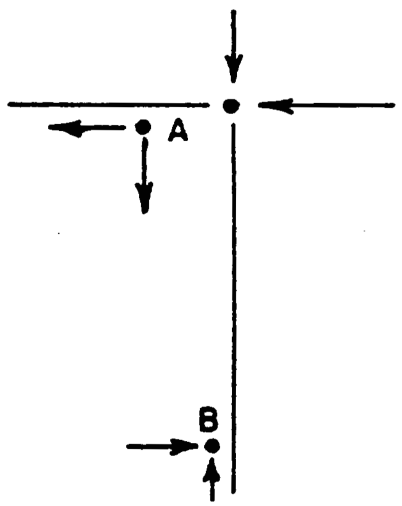
$H = \frac{16}{E} \times C = 4.0 \times C$ (kips/ft.)



Considering the specific steel angle proposed for the armor and the suggested location and orientation of the anchor bars, the geometric relationship between the loads and anchor bar reactions is as given in the figure shown on the left side of this page.

Depending on the direction for the horizontal friction force, the anchor reactions are given by the following:





Moment @ B

$V \times 0.5 + H \times 7.3 - T_V \times 1.5 - T_H \times 6.8 = 0;$

$T_H = T_V;$

$V \times 0.5 + H \times 7.3 - T_V \times 8.3 = 0;$

$T_V = T_H = V \times \frac{0.5}{8.3} + H \times \frac{7.3}{8.3};$

$T_V = T_H = \frac{1}{8.3} (V \times 0.5 + H \times 7.3);$

$R_H = T_H - H = V \times \frac{0.5}{8.3} + H \times \frac{7.3}{8.3} - H;$

$R_H = \frac{1}{8.3} (V \times 0.5 - H);$

$R_V = V + T_V = V + V \times \frac{0.5}{8.3} + H \times \frac{7.3}{8.3};$

$R_V = \frac{1}{8.3} (V \times 8.8 + H \times 7.3);$

Moment @ B

$V \times 0.5 - H \times 7.3 - T_V \times 1.5 - T_H \times 6.8 = 0;$

$T_H = T_V;$

$V \times 0.5 - H \times 7.3 - T_V \times 8.3 = 0;$

$T_V = T_H = V \times \frac{0.5}{8.3} - H \times \frac{7.3}{8.3};$

$T_V = T_H = \frac{1}{8.3} (V \times 0.5 - H \times 7.3);$

$R_H = T_H + H = V \frac{0.5}{8.3} - H \times \frac{7.3}{8.3} + H;$

$R_H = \frac{1}{8.3} (V \times 0.5 + H);$

$R_V = V + T_V = V + V \times \frac{0.5}{8.3} - H \times \frac{7.3}{8.3}$

$R_V = \frac{1}{8.3} (V \times 8.8 - H \times 7.3);$

Inserting the load values in the above equations and considering the possible ranges of impact and friction factors the anchor reactions are as shown in Table 1 and Figure 2. In Figure 2, for design purposes, R_H and minimum R_V values are disregarded. While R_H is negligibly small, **minimum R_V values depend on the direction of traffic and therefore,** should be neglected because both joint armor angles must be designed to carry maximum load reactions.

2.4 Application Example

After the two anchor reactions have been computed, the number of anchor bars, their size, their spacing and the welding requirements for the armored joint system can be determined by relating results and stresses to allowable stresses. Based on the experience gained by the principal researcher during this study, it is proposed that for New Jersey traffic conditions the design reactions should be those associated with a loading impact factor of 30% and a horizontal friction factor of not more than 0.8. The design process required to accommodate these specific reaction values are detailed in the following pages with the final design being shown on Figures 3, 4 and 5. The load carrying capacity of this particular design is identified in Figure 2.

TABLE 1 SUMMARY OF JOINT ARMOR DESIGN LOAD REACTIONS

Applied Vertical Load = 4.0 Kips/ft. Impact Fraction I = 0 to 30%
Applied Horizontal Load = 4.0 x C Kips/ft. Friction Factor C = 0 to 80%

V (Kips) for I (%)	H (Kips) for C = 0% to C = 80%	$V\frac{0.5}{8.3}$	$H\frac{7.3}{8.3}$	$T_V = T_H = V\frac{0.5}{8.3} \pm H\frac{7.3}{8.3}$		$R_H = V\frac{0.5}{8.3} \pm H\frac{7.3}{8.3} \mp H$		$R_V = V + V\frac{0.5}{8.3} \pm H\frac{7.3}{8.3}$		$T = T_V\sqrt{2}$		$R = \sqrt{R_V^2 + R_H^2}$	
				+H	-H	+H	-H	+H	-H	+H	-H	+H	-H
I=0	0.0		0.0	0.241	0.241	0.241	0.241	4.241	4.241	0.341	0.341	4.248	4.248
4.0	3.20	0.241	2.814	3.055	-2.573	-0.145	0.627	7.055	1.427	4.320	-3.639	7.056	1.559
I=5	0.0		0.0	0.253	0.253	0.253	0.253	4.453	4.453	0.358	0.358	4.460	4.460
4.20	3.20	0.253	2.814	3.067	-2.561	-0.133	0.639	7.267	1.639	4.337	-3.622	7.268	1.759
I=10	0.0		0.0	0.265	0.265	0.265	0.265	4.665	4.665	0.375	0.375	4.673	4.673
4.40	3.20	0.265	2.814	3.079	-2.549	-0.121	0.651	7.479	1.851	4.354	-3.605	7.480	1.962
I=15	0.0		0.0	0.277	0.277	0.277	0.277	4.877	4.877	0.392	0.392	4.885	4.885
4.60	3.20	0.277	2.814	3.091	-2.537	-0.109	0.663	7.691	2.063	4.371	-3.588	7.692	2.167
I=20	0.0		0.0	0.289	0.289	0.289	0.289	5.089	5.089	0.409	0.409	5.097	5.097
4.80	3.20	0.289	2.814	3.103	-2.525	-0.097	0.675	7.903	2.275	4.388	-3.571	7.904	2.373
I=25	0.0		0.0	0.301	0.301	0.301	0.301	5.301	5.301	0.426	0.426	5.310	5.310
5.00	3.20	0.301	2.814	3.115	-2.513	-0.085	0.687	8.115	2.487	4.405	-3.554	8.115	2.580
I=30	0.0		0.0	0.313	0.313	0.313	0.313	5.513	5.513	0.443	0.443	5.522	5.522
5.20	3.20	0.313	2.814	3.127	-2.501	-0.073	0.699	8.327	2.699	4.422	-3.537	8.327	2.788

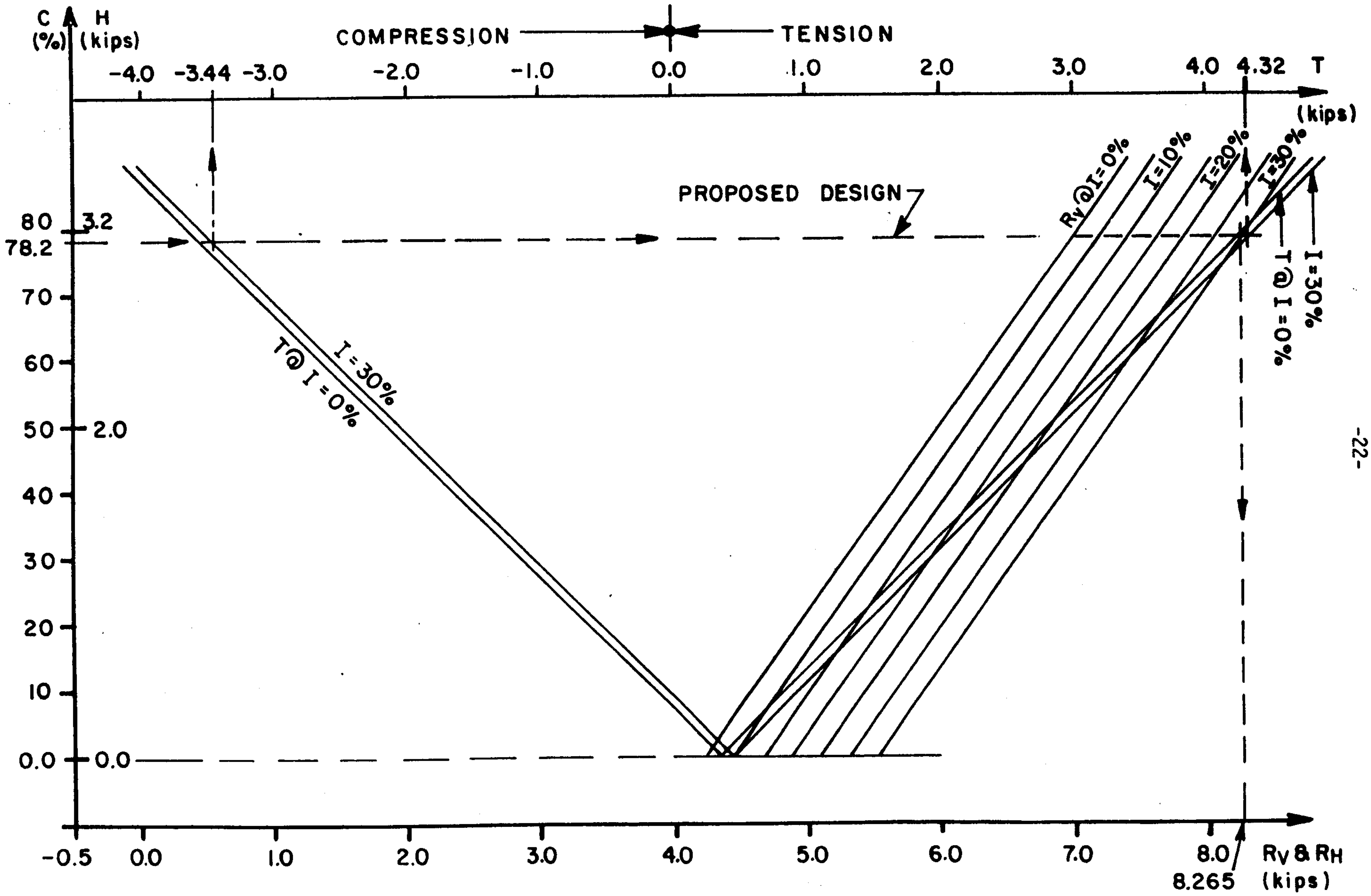
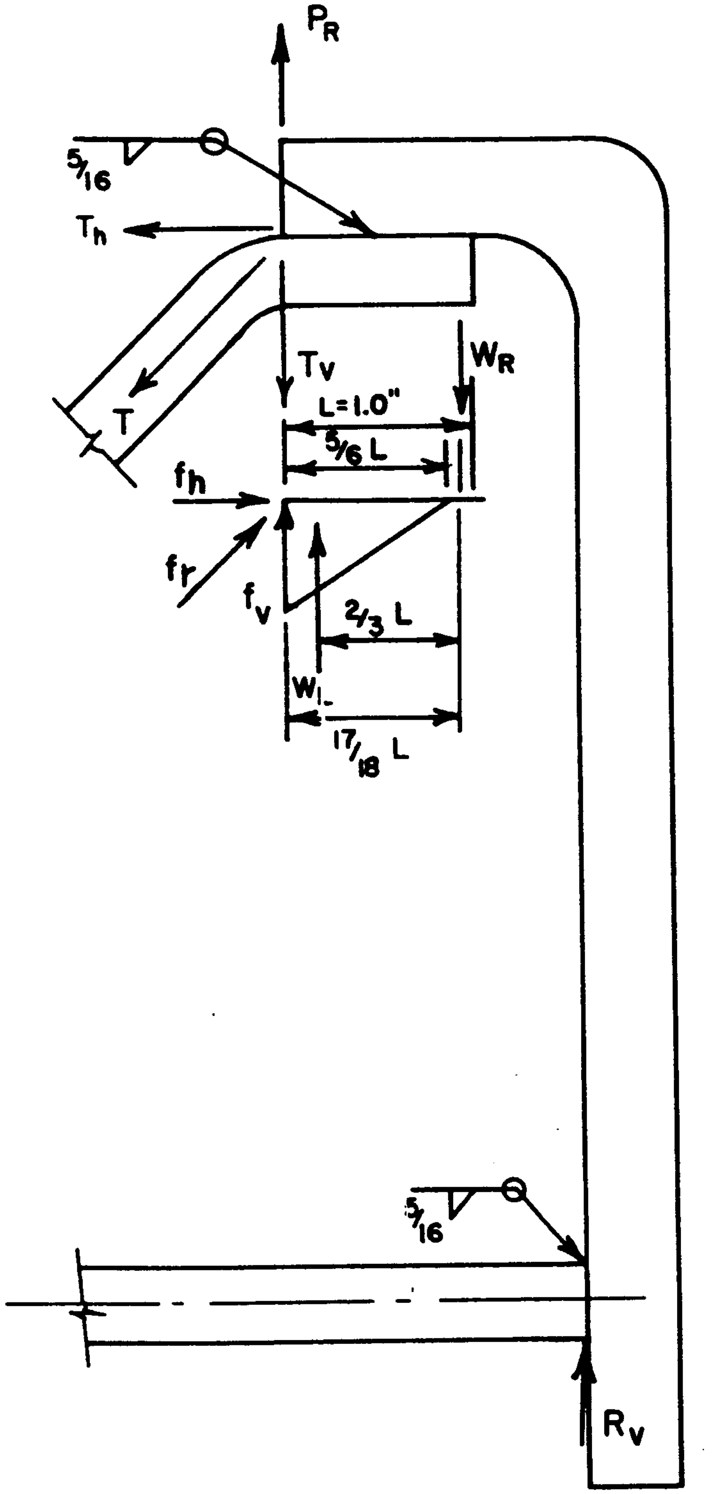


FIGURE 2 : JOINT ARMOR DESIGN LOAD REACTIONS

(KIPS PER FT. WIDTH OF JOINT)



Welding Stresses (4)

The subsequent analyses are based on the Manual of Design for Arc Welded Steel Structures chapter covering the determination of allowable eccentric loads on welded connections. The Manual's author states that the assumption offered in the text follows a middle course and yields results that appear reasonable by comparison with test results.

In the top anchors:

$W_L = W_R = \frac{5}{12} f_v L;$

$P_R = 0.707 f_{all} DL_1;$

Moment about W_R;

$(T_V - P_R) \times \frac{17}{18} L = W_L \times \frac{2}{3} L \times 2;$

Therefore:

$$f_v = \frac{17}{10L}(T_V - 0.707\ f_{all}\ DL_1);$$

$$f_h = \frac{T_H}{(2L + L)}$$

$$f_r = \sqrt{f_v^2 + f_h^2};$$

If spacing of the top anchors is 12 inch O.C. as suggested by findings of the field load tests and

$T_V = T_H$

$L_1 = 1.5$ inch

$L = 1.0$ inch

$D = \frac{5}{16}$ inch

$f_{all} = 12.4$ Kips/Sq. in.

Then:

$$f_r = 0.707\ f_{all}\ D = 0.221\ f_{all};$$

$$f_v = 1.7\ T_H - 0.563\ f_{all};$$

$$f_h = 0.286\ T_H;$$

$$(0.221\ f_{all})^2 = (1.7\ T_H - 0.563\ f_{all})^2 + (0.286\ T_H)^2;$$

$$2.97\ T_H^2 - 1.914\ T_H\ f_{all} + 0.268\ f_{all}^2 = 0;$$

$$T_H^2 - 7.99\ T_H + 13.87 = 0:$$

$$T_H = 5.45 \text{ Kips};$$

$$T = 5.45 \times \sqrt{2} = 7.71 \text{ Kips}; > 4.35 \text{ Kips}$$

In the bottom anchors:

For a spacing of 12 inch O.C. ($n = 1$) and

$a = 0.375$ ($\frac{3}{8}$) inch

$b = 1.5$ inch

$f_{all} = 12.4$ Kips/sq. in

$R_V = 0.707\ f_{all} D \times 2\ (a + b) \times n = 10.3$ Kips $>$ 8.265 Kips

Shearing Stresses:

In the bottom anchors:

For a 12 inch O.C. spacing ($n = 1$) and

$f_{sv} = 12.0$ Kips/sq. in.

$R_V = f_{sv} \times A \times n = f_{sv} \times a \times b \times n = 6.75$ Kips $<$ 8.265 Kips

For a spacing of 8 inch O.C.: $n = \frac{12}{8} = 1.5$

$R_V = 6.75 \times 1.5 = 10.13$ Kips $>$ 8.265 Kips

Bearing Stresses:

In the bottom anchors:

Assuming triangular bearing distribution with available $L_{bear} =$ 10.5 inch, 12 inch O.C. spacing ($n = 1$) and $f_{bear} = 0.7$ Kips/sq. in. the bearing load shall be:

$$R_V = f_{bear} \times b \times \frac{L_{bear}}{2} \times n = 5.51 \text{ Kips} < 6.92 \text{ Kips}$$

For a spacing of 8 inch O.C.: $n = 1.5$

$R_V = 5.51 \times 1.5 = 8.265$ Kips $>$ 6.92 Kips

Tension Stresses:

In the top anchors:

For 12 inch O.C. spacing and $f_s = 20$ Kips/sq. in.:

$T = f_s \times A = f_s \times a \times b = 11.25$ Kips $>$ 4.35 Kips

Bond Stresses:

In the top anchors:

Assuming that a hook shall develop 50 percent of the allowable stress in the strap, the bond load shall be:

$T = f_{bond} \times [2(a + b) \times L_{bond} \times 2] = 1.20\ L_{bond}$

For $L_{bond} = 7$ inch: $T = 8.4$ Kips > 4.35 Kips

Discussion:

It is noted that the foregoing design is for maximum impact and close to maximum friction factors, however, the selection of these factors is left to the descretion of the engineer. While it is felt that the impact factor should not be less than 30%, because of a conservatively chosen wheel load of 16.0 kips the friction factor of 40 to 50% should be more reasonable.

As shown on the graph in Figure 2 and listed in Table 1 in the preceeding design analysis, it is the spacing of the bottom anchors that might cause critical bearing stresses. As can be seen on the proposed standard drawing shown on Figures 3, 4, and 5 the only practical bottom anchor spacing that will prevent interference with other elements would be 4, 6, 8, or 12 inches, since spacing of top anchors can not exceed 12 inches and the approximate spacing of clamping devices should be 3' - 0" to control armor angle deflections prior to installation.

The foregoing design calls for 8 inch bottom anchor spacing. Hence, the next alternative to be analyzed would be a 12 inch bottom spacing for the condition of I = 30% and C = 40%.

This requirement yields from the graph in Figure 2: Ry = 6.92 kips.

The preceeding analysis indicates that the maximum load that can be carried in bearing by the bottom anchors at 12 inch spacing, I = 30% but C = 0%, is 5.51 kips < 6.92 kips. Thus, the bottom spacing as indicated above for practical considerations must remain at 8 inches.

3. Procedure for Header Design

Failure of headers is not uncommon. It is believed that the causes for their failure are as follows:

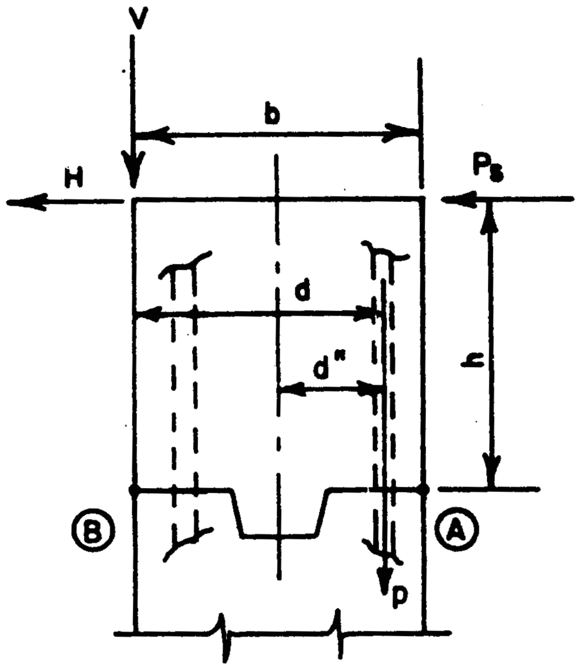
a. Loading, such as indicated in armor design

b. Inadequate preparation of the backfill

c. Concrete approach slabs directly supported by headers.

For the second problem, only one remedy can be suggested -- improvement of quality control in construction. In view of the above, the severity of the assumptions made below is well justified. Additionally, the stresses produced on the commonly used section shown below are well within the practical range.

3.1 Design Loads



Many of the load conditions considered in the joint armor design are also applicable to bridge headers. For design purposes the header loading is given by the following:

Wheel loads:

16.0 Kips

Impact fraction:

I = 30%

Friction factor for tire against concrete:

C = 1.0

Load distribution:

E = 3.0 ft.

$$V = \frac{16.0}{E} \quad (1 + I) \text{ (Kips/ft)}$$

$$H = \frac{16.0}{E} \times C \text{ (Kips/ft)}$$

3.2 Analyses of Applied Loads (5)

To determine reinforcement sizes and spacing and basic header dimensions an analysis of the applied loads is necessary. In effecting this analysis the diagrams, formulas and their derivation shown herein utilized the following notations:

V = a vertical load (wheel load)

H = a horizontal load

P_S = a reasonable estimate of sealer load (1.0 Kip/ft) when compressed to 50%

w = unit weight of concrete

N = total vertical load (axial load)

M = moment

p = load on reinforcement

b = a header width

h = a header height

d = effective depth of flexural member

d" = distance from centerline of concrete section to tensile reinforcement

e = eccentricity measured from tensile steel axis

j = ratio of distance (jd) between resultants of compressive and tensile stresses to effective depth

MOMENT ABOUT PLANE "A - B":

$$M = V \times \frac{b}{2} + (H + P_S) \times h;$$

TOTAL VERTICAL LOAD IS:

$$N = V + w \times b \times h;$$

REINFORCEMENT DESIGN (12):

$$e = \frac{12xM}{N} + d'' \text{ (in)}$$

$$p = \frac{N(e-jd)}{jd} \text{(Kips/ft. width)}$$

Although the stresses in the vicinity of point "B" due to moving loads are somewhat smaller, it is suggested to use the same reinforcement on both sides of a header.

It is strongly emphasized that the headers used in conjunction with joints sealed by preformed elastomeric sealers must be designed as absolutely stationary in order for the sealers to function properly. There can be no horizontal movement and no rotation of the header since the sealer and joint width are selected on the basis of predictable movements, i.e. only on the basis of bridge deck expansion.

The problem of approach slabs is however rather complex, especially if a rigid slab is supported on one end elastically and on the other end by a vertically rigid but horizontally flimsy support such as a header. In such an instance the effect on the header would be to have an eccentrically located, static, vertical load and possibly substantial horizontal static force, as well as other dynamic reactions.

Even a perfect solution of the joint sealing problem will be useless if a header failure disallows proper functioning of the joint.

In the experimental bridges of this research, approach slabs were removed and inadequate preparation of the backfill had to be overcome.

4. Construction Procedure for Armored Joints

The concept of this method is that the entire system (armor angles with straps and seats welded to them, sealer properly pre-compressed between the angles and the supporting elements, such as clamps and attached bolts) is assembled and then placed into the joint before the concrete is poured.

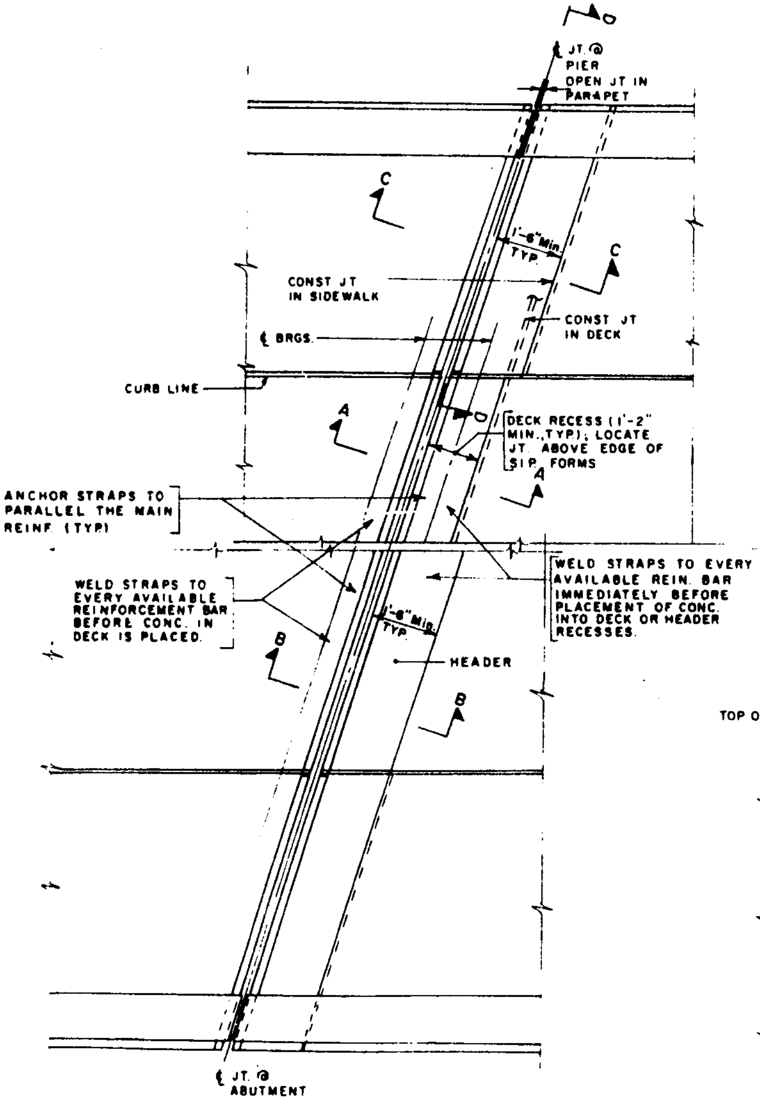
There are many ways of doing the above. The procedure used should satisfy the design requirements on one hand, and on the other, it must give the fullest possible consideration to de facto construction practices.

On this basis, the best approach would be to have the elements of the system fully assembled, delivered to the construction site and placed true to its elevations, joint widths, and the proper position in the bridge deck. The width of the joint between armors, adjusted in accordance with the design requirement, as well as all other pertinent information is shown on the attached standard drawing in Figures 3 to 5. If complete factory assembly is not feasible, it should be factory preassembled to the fullest practicable degree, completing the assembly on the construction site. Standard lubricant shall be applied on each joint armor face when the sealer is located on the armor. Before the assembly is lifted into place on the bridge, the joint opening should be checked at each clamp and reset if necessary.

As shown in Figures 3 and 4, the deck should be poured without a recess only on one side of the joint, leaving the necessary recess on the other side of the joint. The recess should preferably be left on that side of the joint for which there is a moveable deck end, or for which there is an abutment header, with the deck (or header) reinforcement properly extended into it. This recess area is the last of all deck and header concrete to be poured.

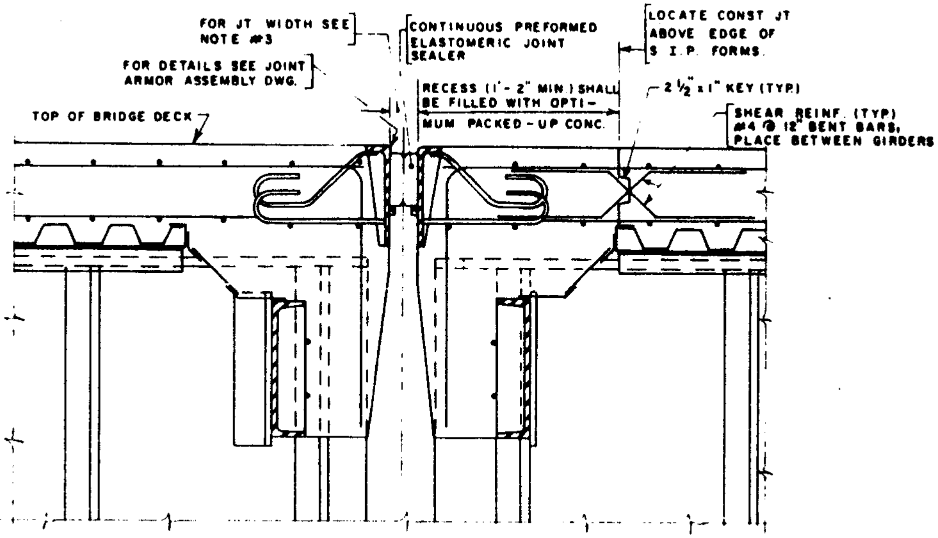
On the side of the joint where the deck is poured without a recess, the armor installation supporting plates should be welded to the main bridge girders at the time of installation of the joint-assembly; this fixes the assembly at its proper elevations and positions it in the bridge deck. The anchorage straps are welded to all available deck reinforcement bars only after those bars are checked for proper placement; this assures stress transfer continuity into the concrete deck which can be poured any time thereafter.

On the recess side of the joint, from the time of initial setting of the assembly, up to the time of pouring of the recess with concrete, the armor supporting plates on this side must slide freely



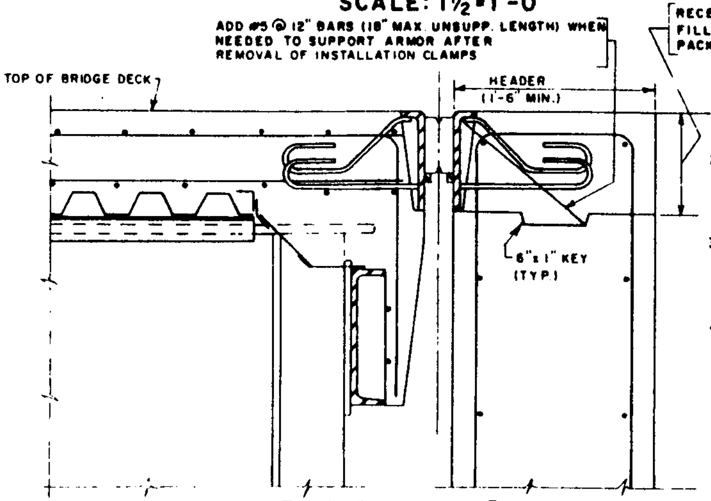
DECK JOINT DETAIL AT PIER AND ABUTMENT

SCALE: 1/2" = 1'-0"



SECTION A-A (TYP. @ PIERS)

SCALE: 1 1/2" = 1'-0"



SECTION B-B (TYP. @ ABUTMENTS)

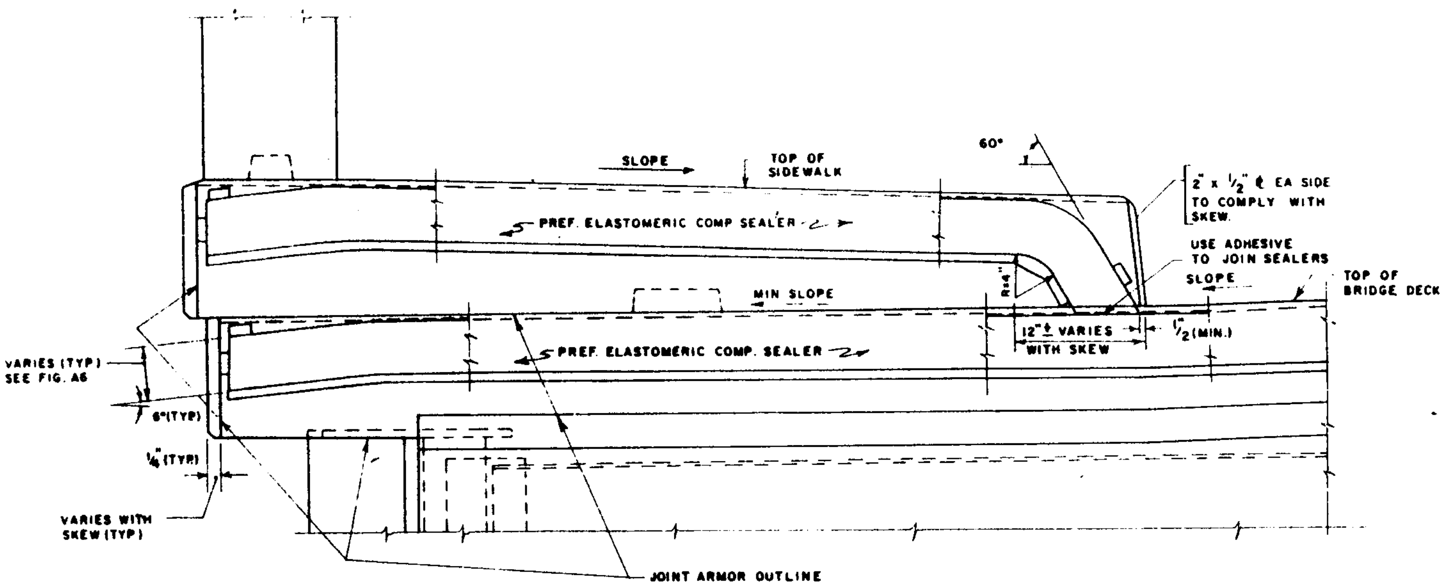
SCALE: 1 1/2" = 1'-0"

GENERAL NOTES:

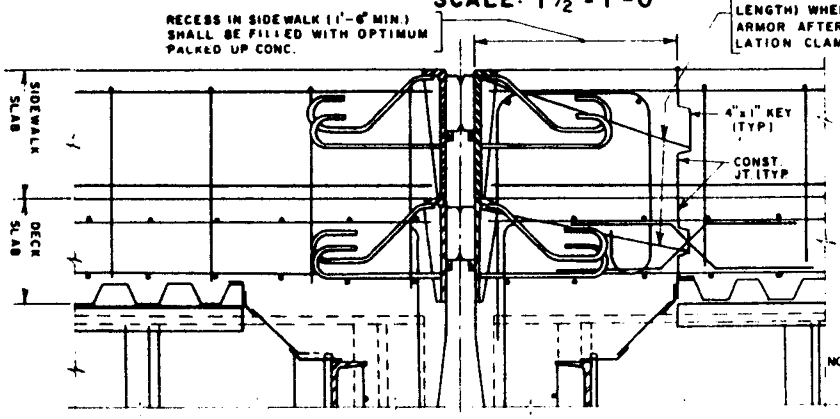
1. CARE SHALL BE TAKEN TO ASSURE THAT AREAS BENEATH THE ANGLES ARE COMPLETELY FILLED WITH CONCRETE.
2. THE PREFORMED ELASTOMERIC JOINT SEALERS SHALL CONFORM TO THE N.J. D.O.T. STANDARD SPECIFICATION FOR ROAD AND BRIDGE CONSTRUCTION.
3. SEALER SIZES AND JOINT WIDTHS SHALL BE DETERMINED IN ACCORDANCE WITH THE "SELECTION PROCEDURE FOR SEALERS".
4. DISREGARD SKEW EFFECT WHEN DEVELOPING NON-SKEW JOINT ARMOR DETAILS.

PROPOSED STANDARD DESIGN DETAILS FOR ARMORED JOINTS FOR BRIDGES SEALED WITH PREFORMED ELASTOMERIC COMPRESSION SEALERS.

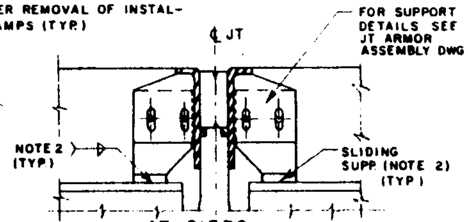
FIG. 3 DECK JOINT PLAN AND SECTIONS



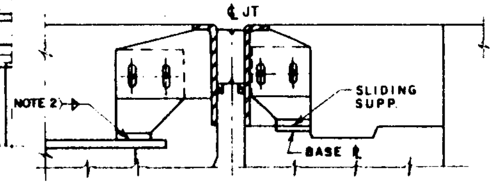
ELEVATION D-D (ARMORED JOINT DETAIL AT SIDE WALK AND CURB LOCATIONS)
SCALE: 1 1/2" = 1'-0"



SECTION C-C (TYP. @ SIDEWALK)
SCALE: 1 1/2" = 1'-0"



AT PIERS



AT ABUTMENTS

DETAIL OF JOINT ARMOR INSTALLATION SUPPORT

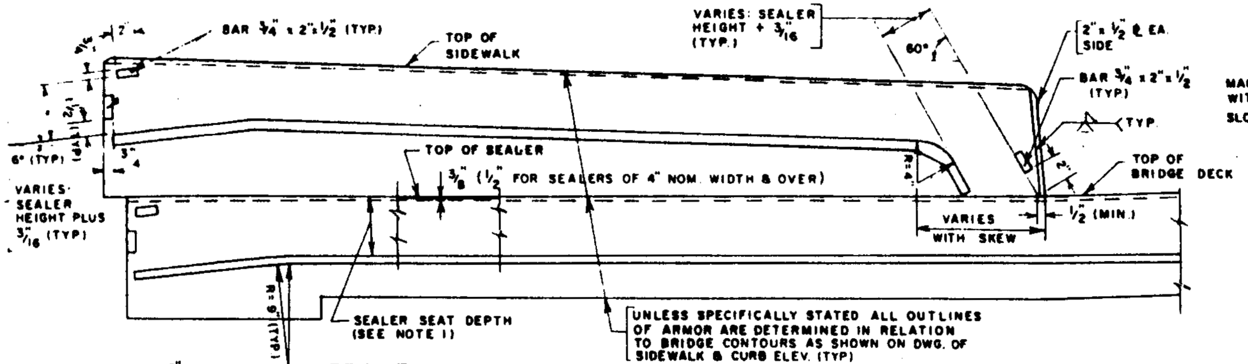
GENERAL NOTE:

1. FOR DETAILS OF ARMOR ASSEMBLY AND ITS INSTALLATION SEE JT. ARMOR ASSEMBLY DWG.
2. AT THE TIME OF ARMOR INSTALLATION WELD ARMOR SUPPORTS TO GIRDERS ONLY ON NO-RECESS SIDE OF THE JOINT. IN DECK AND HEADER RECESSES PROVIDE SLIDING ARMOR SUPPORTS.
3. FOR OTHER GENERAL NOTES SEE DWGS. OF DECK JOINT AND ARMOR ASSEMBLY.

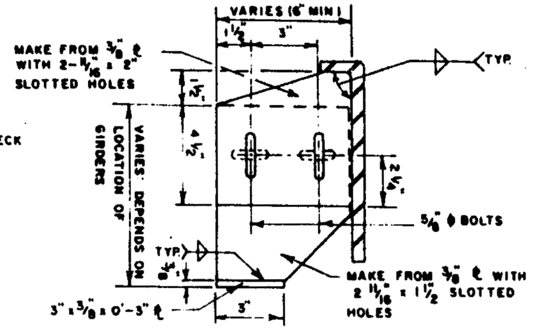
PROPOSED STANDARD DESIGN DETAILS FOR ARMORED JOINTS FOR BRIDGES SEALED WITH PREFORMED ELASTOMERIC COMPRESSION SEALERS

FIG. 4

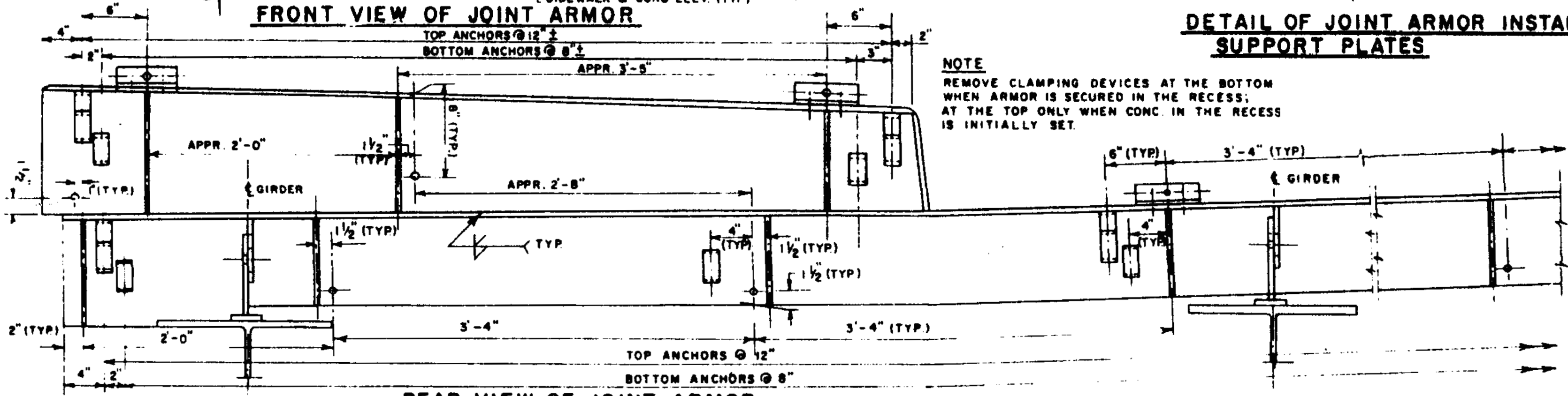
SIDE WALK AND CURB LOCATIONS
ELEVATION AND SECTIONS



FRONT VIEW OF JOINT ARMOR

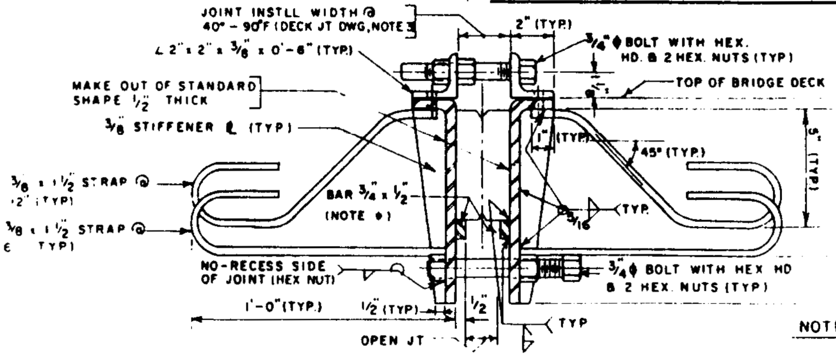


DETAIL OF JOINT ARMOR INSTALLATION SUPPORT PLATES

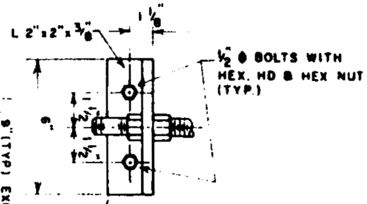


NOTE
REMOVE CLAMPING DEVICES AT THE BOTTOM WHEN ARMOR IS SECURED IN THE RECESS; AT THE TOP ONLY WHEN CONC. IN THE RECESS IS INITIALLY SET.

REAR VIEW OF JOINT ARMOR



DETAIL OF ARMORED JOINT ASSEMBLY



TYP. DETAIL OF CLAMP ANGLES

NOTE DO NOT PAINT ANCHOR STRAPS

♦ FOR 2" OR LESS WIDE SEALERS USE 3/4" x 1/4" BARS

GENERAL NOTES:

1. DEPTH OF SEALER SEAT SHALL BE SEALER HEIGHT PLUS 1/2" FOR ALL SEALERS UP TO 4" NOM. WIDTH; HEIGHT + 3/4" FOR LARGER SIZES.
2. THE HORIZONTAL SPACING BETWEEN STAGGERED INSTALL. CLAMPS SHALL BE NOT MORE THAN 2'-6" AT THE ENDS OF ARMOR AND ABOUT 3'-0" OTHERWISE.
3. THE DESIGN OF JOINT ARMOR SHALL BE AS PROPOSED IN THE JOINT ARMOR DESIGN PROCEDURE.
4. ALL COMPONENTS OF THE ARMORED JOINT ASSEMBLY SHALL BE ASTM A-242 STEEL OR EQUAL.
5. ARMAMENT AND SEALER TO BE SHOP-ASSEMBLED AND CLAMPED AT FINAL WIDTH SETTING (SELECTION PROCEDURE FOR SEALERS). IF FIELD ASSEMBLY IS REQUIRED, JOINT WIDTH SHALL BE SET WITH SEALER IN PLACE

PROPOSED STANDARD DESIGN DETAILS FOR ARMORED JOINTS FOR BRIDGES SEALED WITH PREFORMED ELASTOMERIC COMPRESSION SEALERS.

FIG. 5 JOINT ARMOR ASSEMBLY DETAILS SECTIONS AND ELEVATIONS on top of the stringers (or header recess). This allows the joint-assembly to maintain its proper joint width during construction of the bridge decks. After all deck concrete has stabilized from curing, shrinking and camber settlement, (about one week after pouring) the recess may be poured.

On the day when the concrete is poured into the recess there are essentially 3 steps which should be performed during the two hours immediately preceding the actual pour. Experience indicates that this time allowance is ample.

1. Perform a final joint-opening check at each top clamp and adjust if necessary.

2. Weld anchorage straps to every available reinforcement bar, and auxiliary #5 @ 12" support-bars.

3. Remove the bolts from the bottom clamping devices of the joint assembly.

The welding of anchor straps is done to assure stress transfer continuity, but also to prevent joint width opening before interaction between the armor and the concrete is assured. Curb and gutter areas present the most problem in this regard.

At this point the two sides of the armored joint are fastened not only to each other, but to their respective deck spans as well. Therefore, the importance of pouring the concrete immediately is obvious. The concrete should be vibrated during pouring to achieve optimum density. Experience indicates that the entire pour would require about one hour. Immediately following initial set of the concrete, the top clamps of the assembly should be removed, and concrete in these areas should be refinished if necessary. This procedure will provide a satisfactorily sealed joint with a minimal amount of care.

The armored deck joints should be continuous throughout the full width of the deck, and termination should be accomplished as shown on the standard drawings, Figures 3 to 5. It is obvious that the armor is utilized for a dual purpose: to armor the joints where necessary, and to form the best sealed joint possible.

The seal-groove in the sidewalk should also be armored in the same manner with the curb and outside ends installed as shown in Figures 4 and 5 but a stay-in-place anchor seat could be added to the curb end at the bottom outside face of the armor shapes.

All steel at the armor network except for parts in contact with concrete should be shop painted and touched-up in the field after removal of armor holding elements. In addition, it is recommended that the armor be of ASTM A-242 steel or its equivalent. The stable rust characteristics of this material will serve advantageously in those areas where paint is likely to deteriorate rapidly with traffic or where bridges are open to traffic at a later date.

In summary for successful construction of this type of bridge joint, the following basic requirements are absolutely essential:

a. The structural integrity of said armor must be preserved, i.e., it must be fabricated and constructed exactly in accordance with the drawings and the specifications.

b. Once the joint armor is fabricated and assembled with a sealer in place, there should be no tampering with its integrity until completion of construction.

c. Precise placement of armor is absolutely essential, i.e. before concrete is poured said armor must be located true to the bridge deck surface outline. The installation should be performed in such a manner that until concrete is set no bridge end movements are transferred into the joint armor.

5. Selection Procedure for Sealers

The Selection Procedure for Sealers is an empirical method that establishes the size of sealer to be used in a joint and determines at what width the joint must be constructed in order to assure the effectiveness of the sealer. It sets forth in advance the capabilities of the sealer in terms of three parameters - "X", "Y", and "Z". Each of the parameters is the ratio of the sealers width at a certain level of compression to its nominal width "W_n", multiplied by 100. "Z" is the value of the ratio at the maximum permitted compression of the sealer. "Y" is the desired value of the ratio at the time of sealer installation. "X" is the value of the ratio at the minimum permitted compression of the sealer (enough compression to prevent leakage between sealer and joint face).

The limits "X", "Y", and "Z" are empirical values based on experience. For the compression type of sealer, "X" can be no more than 80%, "Z" should be not more than 50% and therefore "Y" should be approximately 60 to 65%.

The designer must first realize that the bridge joint will be constructed at some width that is pre-set at the factory assembly of the armored joint. The bridge temperature at the time when the joint will become an integral part of the deck can not be known, but it is known that all subsequent bridge movements will occur from that pre-set width. Therefore, the design essentially consists of establishing the maximum magnitude of these movements (Figures 6 or 7) and selecting a sealer and construction joint width on the basis of "X" and "Z" values.

To illustrate the application of Charts in Figures 6, 7, and 8 a solution for a composite bridge design with a span L = 100 ft. is given below.

For New Jersey a controlling ambient temperature range of 0° to 110° is considered realistic as established by this research. The wide range of joint armor installation and construction ambient temperatures of 40°F to 90°F* is selected with required limits of efficiency coefficients taken as:

Z = 50% ± at minimum width of joint (W_{jmin}) and 110°F

Y = 60% ± at installation width of joint (W_{jinst}) and construction air temperature from 40°F to 90°F

X = 80% ± at maximum width of joint (W_{jmax}) and 0°F

Step 1. If the joint is secured into place at a temperature of 40°F, and at some time thereafter warms up to 110°F, its temperature increase would be Δt = 70°F (110° - 40°). Entering Figure 6 with Δt,

*The construction air temperature range between 40°F and 90°F is selected as the only realistic approach to existing construction practices.

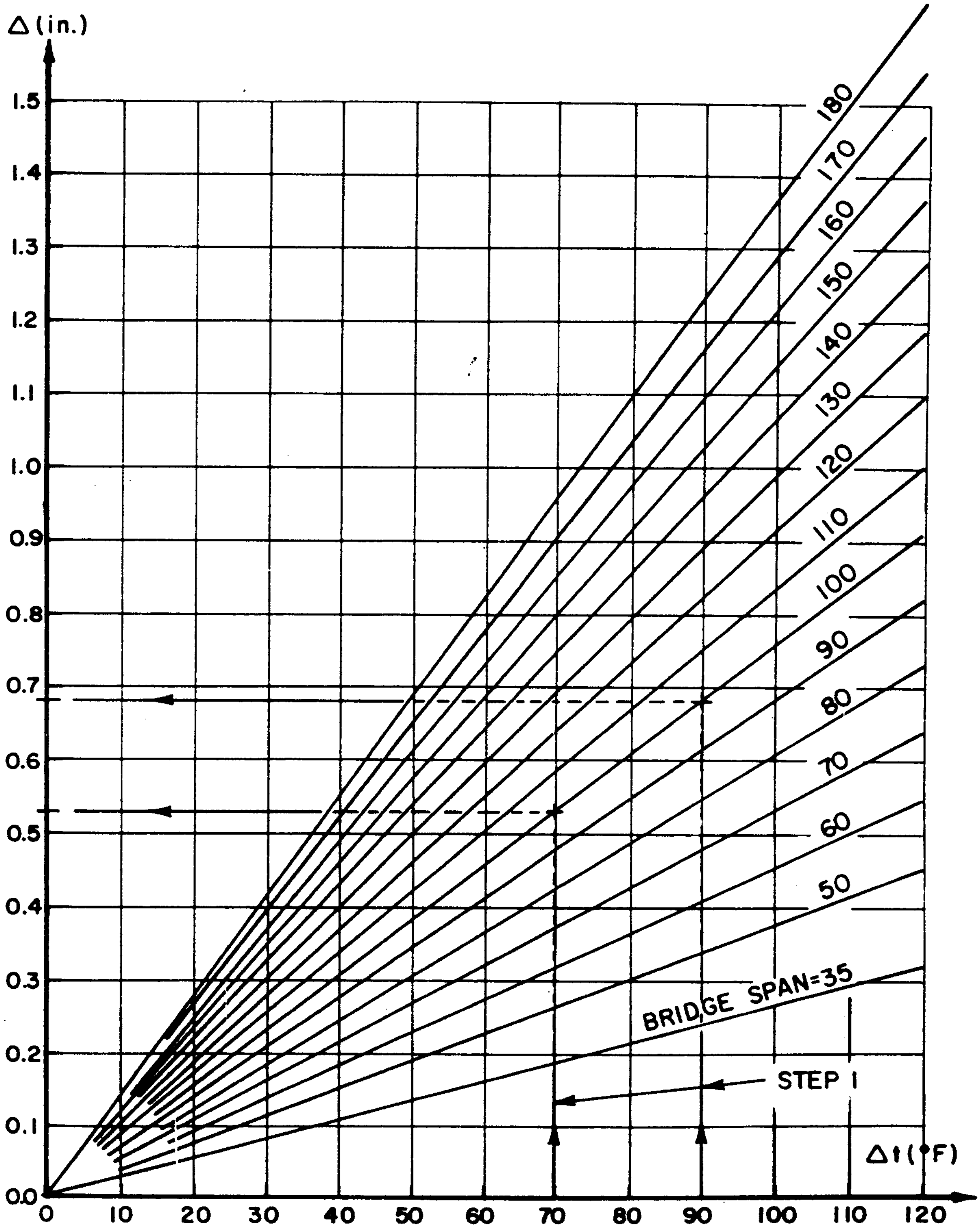


FIGURE 6 : "Δ" MOVEMENT FOR L = 0 to 180 FT.
Δt = 0 to 120°F and α = 6.3 X 10^{-6} in./in./°F

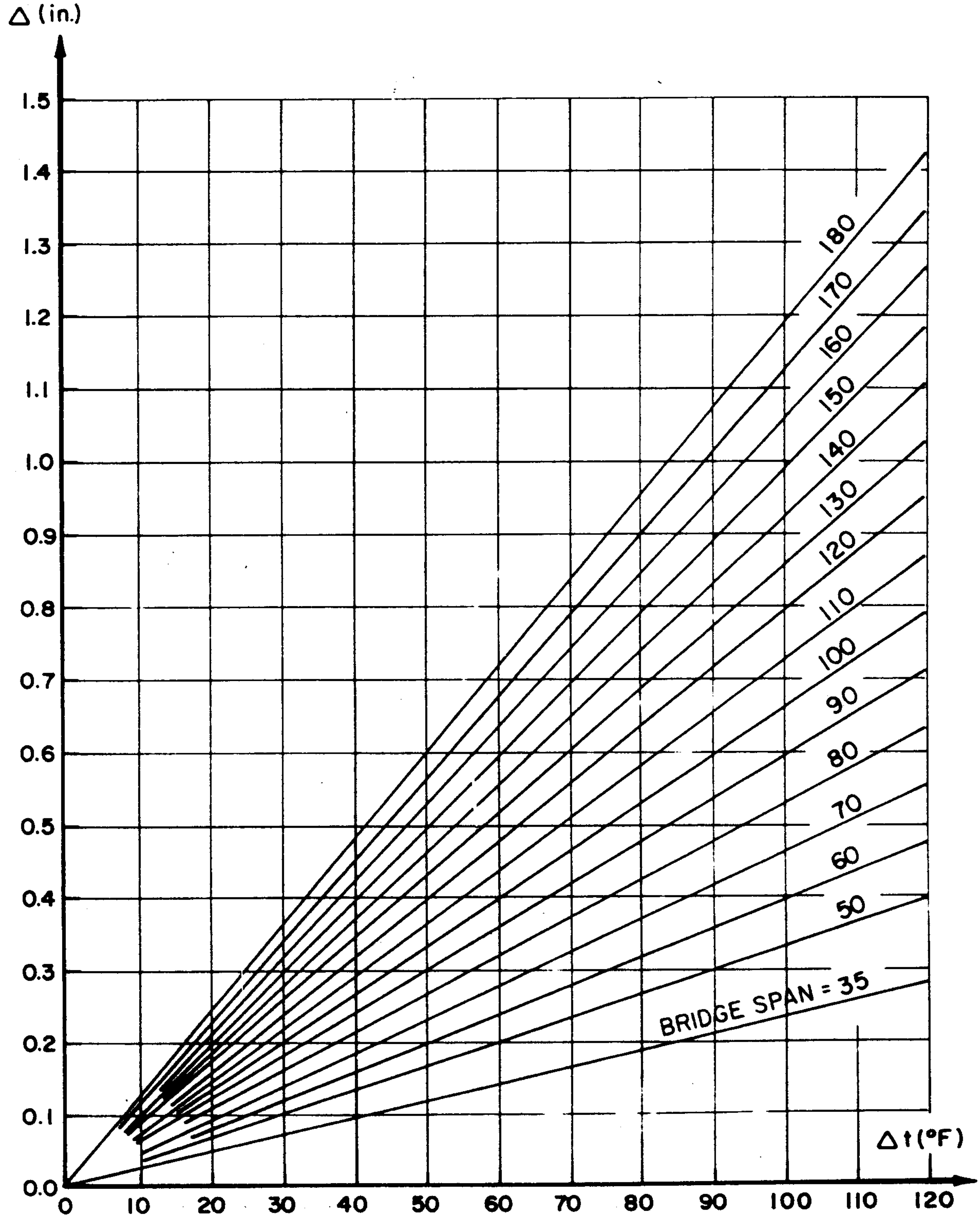


FIGURE 7: "Δ" MOVEMENT FOR L = 0 to 180 FT.
$\Delta t = 0°$ to $120°F$ and $\alpha = 5.5 \times 10^{-6}$ in./in./°F

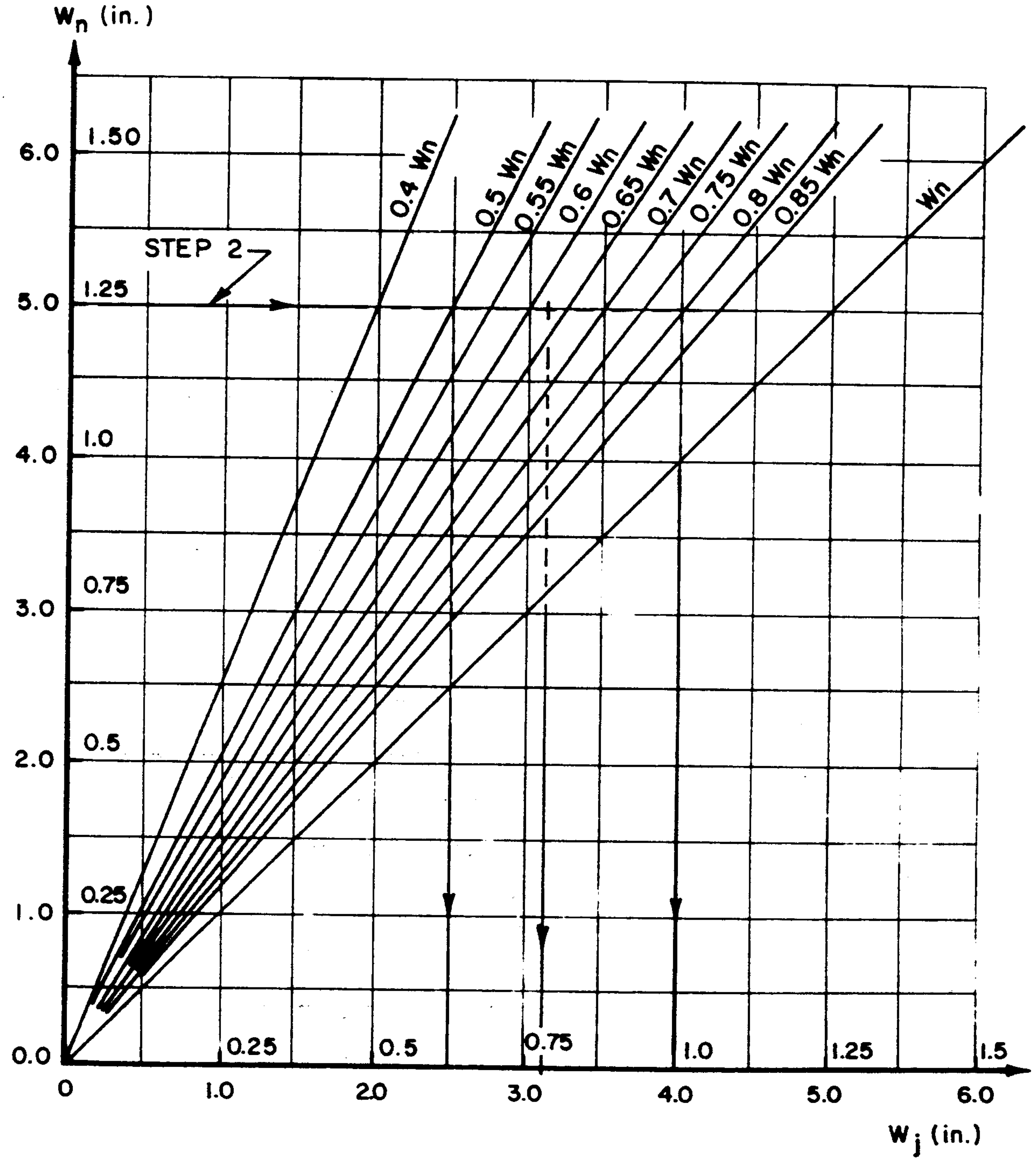


FIGURE 8 : JOINT SEALER EFFICIENCY CHART

a 100 foot span will incur a joint closure of $\Delta_1 = 0.52$ inch. On the other hand, if the installation temperature is 90°F, the bridge may subsequently cool to 0°F and $\Delta t = 90°$ (90° - 0°). Entering Figure 6 again with Δt, a 100 ft. span will yield $\Delta_2 = 0.68$ inch of joint opening. Notice that even though the bridge will only incur a temperature range of 110°F, because of the wide range of possible installation temperatures the sealer must actually be designed for a total range of 70°F + 90°F = 160°F.

Step 2. By estimating the sealer size $W_n = 5.0$ in. and using the limits $Z = 0.5W_n$ and $X = 0.8W_n$, we find from Figure 8 that:

$$W_{jmax} - W_{jmin} = 4.00 - 2.50 = 1.50 \text{ in.}$$

The pre-set width of the joint has a tolerance of ± 1/16 inch which effectively increases the required sealer movement range at each end by 1/16 inch.

Therefore: $\text{Req'd} = (\Delta_1 + 1/16) + (\Delta_2 + 1/16) =$

$0.58 + 0.74 = 1.32 \text{ inch} < 1.50 \text{ inch}$

Step 3. The joint installation width would be between 3.26 (4.00 - 0.74) and 3.08 (2.50 + 0.58) inches. Choose 3 1/8 + 1/16 inch = $W_{jconstr}$.

The preceding 3 steps illustrate the principles underlying sealer selection. However, it is noted that if the designer chooses different values of α (thermal coefficient), ambient air temperatures, and limits for X and Z from those proposed here, he may then construct his own tables similar to Tables 2 and 3. From such tables a required sealer and armored joint installation width could be easily obtained,

Table 2

Guide to the Design of Sealers in Composite and Steel Bridges
Coefficient of Thermal Expansion 6.3×10^{-6} in/in/°F
Controlling Termperature Range: 0° to 110°F*
Construction Temperature Range 40° to 90°F*
Degrees of Efficiency $Z = \pm 0.50\ W_n$

$Y = \pm 0.60$ to $0.65\ W_n$

$X = \pm 0.80\ W_n$

Limits of Span (ft)	W_n (in)	@ 110°F		Δ@ Δt= 70°F	@ 40 to 90°F		Δ@ Δt= 90°F	@ 0°F	
		W_j min (in)	Z		W_j (in) ($\pm$1/16" toler-ance)	Y		W_j max (in)	X
Up to 30	1-1/2"	0.875 0.715	0.58 0.48	0.00 0.16	15/16"	0.625	0.00 0.20	1.00 1.20	0.67 0.80
30 to 35	1-3/4"	0.90 0.87	0.515 0.50	0.16 0.19	1-1/8"	0.64	0.20 0.24	1.39 1.43	0.79 0.82
35 to 45	2"	1.00 0.95	0.50 0.47	0.19 0.24	1-1/4"	0.625	0.24 0.31	1.55 1.62	0.78 0.81
45 to 55	2-1/2"	1.26 1.21	0.50 0.48	0.24 0.29	1-9/16"	0.625	0.31 0.37	1.94 2.00	0.77 0.80
55 to 70	3"	1.52 1.44	0.51 0.48	0.29 0.37	1-7/8"	0.625	0.37 0.48	2.31 2.42	0.77 0.81
70 to 95	4"	2.07 1.94	0.52 0.49	0.37 0.50	2-1/2"	0.625	0.48 0.65	3.04 3.21	0.76 0.80
95 to 120	5"	2.56 2.42	0.51 0.48	0.50 0.64	3-1/8"	0.625	0.65 0.82	3.84 4.01	0.77 0.80
120 to 150	6"	3.05 2.90	0.51 0.48	0.64 0.79	3-3/4"	0.625	0.82 1.02	4.63 4.83	0.77 0.80

*Note: These temperatures are the actual ambient temperatures of a respective bridge site.

Table 3

Guide to the Design of Sealers in Concrete Bridges
Coefficient of Thermal Expansion 5.5×10^{-6} in/in/°F
Controlling Temperature Range: 0° to 110°F*
Construction Temperature Range: 40° to 90°F*
Degrees of Efficiency $Z = \pm 0.50\ W_n$

$Y = \pm 0.60$ to $0.65\ W_n$

$X = \pm 0.80\ W_n$

Limits of Span (ft)	W_n (in)	@ 110°F		Δ@ Δt= 70°F	@ 40 to 90°F		Δ@ Δt= 90°F	@ 0°F	
		$W_{j\,min}$ (in)	Z		W_j(in) ($\pm$1/16" tolerance	Y		$W_{j\,max}$ (in)	X
Up to 35	1-1/2"	0.875 0.715	0.58 0.48	0.00 0.16	15/16"	0.625	0.00 0.21	1.00 1.21	0.67 0.81
35 to 40	1-3/4"	0.90 0.88	0.515 0.50	0.16 0.18	1-1/8"	0.64	0.21 0.24	1.40 1.43	0.80 0.82
40 to 50	2"	1.01 0.96	0.51 0.48	0.18 0.23	1-1/4"	0.625	0.24 0.30	1.56 1.61	0.78 0.81
50 to 65	2-1/2"	1.27 1.20	0.51 0.48	0.23 0.30	1-9/16"	0.625	0.30 0.39	1.93 2.02	0.77 0.81
65 to 80	3"	1.51 1.44	0.50 0.48	0.30 0.37	1-7/8"	0.625	0.39 0.48	2.33 2.43	0.78 0.81
80 to 110	4"	2.07 1.93	0.52 0.48	0.37 0.51	2-1/2"	0.625	0.48 0.65	3.04 3.21	0 76 0.80
110 to 140	5"	2.55 2.41	0.51 0.48	0.51 0.65	3-1/8"	0.625	0.65 0.83	3.84 4.02	0.77 0.80
140 to 170	6"	3.04 2.90	0.51 0.48	0.65 0.79	3-3/4"	0.625	0.83 1.01	4.64 4.82	0.77 0.80

*Note: These temperatures are the actual ambient temperatures of a respective bridge site.

given only the length of the bridge deck.

6. Considerations for Replacement of Sealers

To remedy the results of poor joint construction, the sealer in a few of the experimental joints included in this research should have been replaced. To this and any other similar efforts the following word of caution is offered.

Replacement of sealers is ill-advised unless it is performed with great care; it involves considerable expense, and also inconveniences the riding public. For these reasons, at the researcher's recommendation no sealers were replaced on the experimental bridges. If the sealers in the fixed sawed joints studied were to be replaced, as they should have been in at least one bridge, care would necessarily have had to have been taken not to jeopardize the functional efficiency of the replacement sealers. Since the joints were sawed improperly, they would first need to be resawed, then thoroughly cleaned and/or adequately repaired and prepared. Immediately thereafter a proper size continuous sealer should be installed in accordance with the originally established procedures. Of course sufficiently prior to installation, testing and approval of the new sealers by the Department would be required. Continuous and adequate supervision is most essential.

However, to achieve the best results it would be necessary to utilize the armored joint construction method as a replacement procedure. This of course could be performed by removing concrete at the joint to provide a sufficient recess for the armored joint assembly. Further steps would be to follow the normal armored joint construction procedure.

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