than 50 percent of the increased corrosion damage can be assigned to deicing salts (17). The assessed portion of costs against deicing salts of a postconstruction corrosion-proofed vehicle, therefore, will have to be small.

Also, since consumers have not taken advantage of the available postconstruction rust-proofing process, the apparent costs of corrosion to them must be less than the additional costs of postconstruction protection and increased depreciation. Therefore, the costs assessed against deicing salts cannot be greater than the costs assignable to rust proofing a vehicle.

Proposition Two

The corrosion resistance of the average automobile is significantly better now than it was in 1955. So much so that one motor company is offering a 36-month warranty against perforation corrosion on all of its passenger automobile lines in Canada. On the basis of this warranty offering and the general improvement in technology, as shown by the metal coupons in Figure 10, it can be assumed that the average well-manufactured automobile today should be able to withstand at least 2 years in a severely corrosive area without perforating, and in the near future, 3 years.

The differences between this proposition and proposition one are these: (a) the depreciation assignable to corrosion would at present begin in the third year of the lifetime of the automobile and in the near future in the fourth year, and (b) there would be no charges assignable to postconstruction rust proofing.

SUMMARY

It is reasonable to assume that changes in manufacturing practices were more responsible than deicing salts for the high rust susceptibility of automobiles manufactured after 1955. However, in the last 20 years, the corrosion resistance of the average automobile has been improved substantially. And, if the manufacturers continue their present trend in adopting new corrosion technology, the automobile should continue to improve in corrosion resistance for some time to come. As the population of vehicles with improved corrosion resistance increases, the costs assignable to the corrosive environment, with or without deicing salts, will be reduced, eventually to the capitalized investment of manufacturer-installed corrosion protection.

In the spring of 1976, the cost to coil coat all the nongalvanized body parts of an automobile [approximately 270 kg (600 lb) of sheet steel] would have been approximately $35 dollars.

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Preformed Elastomeric Joint Sealers for Bridges

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A proven effective solution to the problem of sealing joints in bridge decks is described. The basis of the solution is the use of specially designed joint armor in combination with currently available, preformed elastomeric sealers. The approach is adequate for simple-span, composite, concrete, or steel structures having span lengths up to 52 m (170 ft). The special armored-joint system was field tested on three structures. Two of these 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 5-year time span. The third structure was used to conduct load tests on the joint armor and armor anchorage components. Application of the results led to the development of practical

procedures for the design and construction of armored bridge joints and the selection of an appropriate size of elastomeric sealer.

In 1965, the New Jersey Highway Department, now the New Jersey State Department of Transportation (NJDOT), began a study of the behavior of preformed, elastomeric bridge-joint sealers, which had by that time become a common means of sealing bridge joints. The preliminary results showed a lack of adequate knowledge of the characteristics of sealer materials and the behavior of bridge joints, and in response, a formal research project was launched. The first phase of the project, the subject of this report, concentrated on testing and improving the suggested methods of design and construction of joint systems and establishing relations among deck temperatures, air temperatures, and joint movements. The second phase will concentrate on the development of realistic acceptance specifications for preformed sealers.

In phase 1, methods for the design and construction of an effective joint-sealing system for bridges have been developed and proved successful. Armored joints sealed with preformed sealers have been installed on two typical highway bridges and have functioned flawlessly for over 5 years. The relation between joint movements and air temperatures for simple-span bridges has been determined.

This summary of phase 1 of the joint-sealing project omits many details of how the research was conducted, which are available elsewhere (1, 2, 3, 4). It is expected that the reader here will be most interested in the project's accomplishments. Accordingly, this summary primarily presents the results, particularly those methods for design, construction, and sealing of bridge joints that have been found successful for New Jersey highway structures.

SCOPE OF PHASE 1

The scope of phase 1 of this project was to test suggested methods for the design and construction of joint systems, to establish the causes of joint movements, and to identify the relations of design and construction methods to such movements. For the most part, the study was limited to two simple-span, composite-design bridges that had significantly different joint skews and length-to-width ratios. Together, these bridges typify the great majority of highway bridges in New Jersey today. The study included the design, construction, and performance evaluation of the armored and sawed joints of the two bridges.

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

To further evaluate the movement and temperature data, the coefficient of thermal expansion of a concrete deck was determined, and the influence of moisture on it was isolated.

Although it was not originally considered within the scope of this project, a test program was included involving the load testing of an armored bridge joint. The full-scale field tests were designed to evaluate the load distributions and reinforcement stresses that must be accommodated in designing joint armor.

CONCLUSIONS OF PHASE 1

This phase of the project has resulted in the development of procedures for (a) the design of joint armor, (b) the construction of armored joints, and (c) the selection of sealers. Each of these has been successfully field tested for more than 5 years on two experimental bridges, and a third experimental installation of an armored joint that was completed in the fall of 1974 included the final modifications of all of the procedures. The conclusions of phase 1 are summarized below:

1. The use of a combination of an armored joint and an appropriate preformed, elastomeric joint sealer is a practical and proven solution to the problem of bridge-joint leakage and intrusion.
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 procedures suggested here and given in detail elsewhere (1) require only a little care in manufacturing and construction and should lead to a totally satisfactory result.
3. The formed and sawed methods of joint construction evaluated 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 the fact that it can be prefabricated and then installed within the constraints of normal construction practices.
4. For simple-span bridges, 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. Other environmental parameters (such as insolation, precipitation, and moisture) and physical characteristics (such as creep) were found to have no significant 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 design purposes then, it is the range of ambient air temperatures at the particular site that may be assumed to be the effective temperature for a bridge. This is, of course, consistent with normal design assumptions. For New Jersey, climatological records indicate that this temperature range should be taken as -18 to 43° C (0 to 110° F).

5. The movements of fixed joints are insignificant, although somewhat erratic. This demonstrates the adequacy of the bridge-bearing design used in New Jersey and the validity of the basic design assumptions. The erratic features, of course, are due to the normal 1.5-mm ($^1/_{16}$-in) tolerances that are permitted for bolted connections of metal parts. In view of the critical need of attaining well-controlled movements, the bearing system used in New Jersey is compatible with the use of preformed elastomeric joint sealers; furthermore, only bearing systems that are known to similarly control the joint movements should be used in conjunction with these sealers.
6. The displacements at expansion joints that are predicted on the basis of the normal air temperature range will probably not be exceeded provided the thermal coefficient of expansion of the particular bridge is known fairly accurately. For composite bridges constructed of a steel superstructure and a reinforced concrete deck, this coefficient lies between the accepted coefficients of 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 probably 0.000 011 5 m/m/°C (0.000 006 3 in/in/°F). 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, and the thermal coefficients should be taken as linear throughout the temperature range.

7. For bridges having skews of less than 15° (0.25 rad) and the kinds of bearing systems used by NJDOT, the overall joint movements may be accurately calculated by using the following general formula:

$$\Delta L = \alpha L \Delta t \qquad (1)$$

where

L = length of bridge in direction of stringers,
t = temperature, and
α = coefficient of thermal expansion.

The effects of the skew can be neglected.

8. For bridges having skews approximately between 15 and 50° (0.25 and 0.9 rad), the joint movements that occur in the direction perpendicular to the joint can be assumed as uniform across the length of the joint. The magnitudes of these movements will be less than that predicted by use of the general formula. However, as the skew angle increases and the ratio of the length of the bridge to the length of the joint becomes less than 1, the joint movements in the direction parallel to the joint become substantially larger than those in the direction perpendicular to the joint. Thus, caution is necessary to ensure that these movements are accommodated or minimized, depending on the type of joint system in use.

9. In general, the bearing system that is in standard use by NJDOT for bridge construction is effective in controlling and directing bridge displacements.

10. In a 200 to 230-mm (8 to 9-in) thick concrete bridge deck, temperature gradients through the thickness of the deck exceed 11°C (20°F) at times throughout the year. These large gradients are of short duration and are primarily due to intense solar radiation. Most of the temperature differential at such times occurs in the top 25 or 50 mm (1 or 2 in) at the surface of the deck and has little immediate influence on the displacement response of the bridge.

11. A general, although guarded, conclusion can be made that—provided there is compatiblity of materials—many of the effects of temperature or the lack of effects of moisture, creep, and such are transferable; i.e., generally similar effects can be expected in other simple-span, composite bridges and also in all-steel and all-concrete simple-span structures. On the other hand, however, bridge-end displacements are also a function of the particular bridge design and are unique for each and every design system. For example, the bridge-ends in a cantilever design system behave differently from those in a simple beam or a continuous beam design. To attempt to combine the movement characteristics of all bridge designs or to extrapolate from one to another could lead to gross errors. Thus, the selection of the simple-span type of structure for instrumentation, because of its functional simplicity, served well to isolate the phenomena affecting bridge-end movements.

RECOMMENDATIONS OF PHASE 1

In addition to those that can be derived from the above conclusions, phase 1 led to the following recommendations:

1. The 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 given below. Strict adherence to these procedures will provide leak-proof and intrusion-proof joints for bridges up to 52 m (170 ft) in span length.

2. For bridges with spans exceeding 52 m in length, many deficient joint seals are being marketed. Recommendations on this subject have been given elsewhere (5).

3. Because of the unpredictable behavior of bridge approach slabs, under no circumstances should an armored, elastomeric sealer joint be placed directly between a bridge deck and its approach slab.

4. The use of preformed, elastomeric compression sealers in concrete pavement joints of the type [19-mm (¾-in) expansion joints] and spacing [24 m (78 ft)] used by NJDOT is unwarranted and is specifically not recommended.

5. The investigations performed in this study should be broadened to include varied locations, larger spans, and different types of bridges. The behavior of structures having skew angles larger than 50° (0.9 rad) and a ratio of bridge length to length of joint of less than 1 was not quantified, and further investigation is warranted.

6. The manner in which a bridge approach slab should tie into a structure and 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 these slabs.

PROCEDURES FOR DESIGN, CONSTRUCTION, AND SEALER SELECTION

General

The selection of one specific joint-armor design rests on the basis of rather extensive experimentation in construction. There is no such thing as a foolproof 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:

1. The deck joints must be horizontally straight from outer edge to outer edge, and the sidewalk joints must be directly above in the same straight fashion.

2. The main sealers must be placed out to out, and the sidewalk sealers must also be placed out to out; i.e., bottom of curb to outside of structure with only one vertical shallow bend [60° (1 rad)] at the curb.

Joint-Armor Design Procedure

Basic Design Considerations

In the United States, there is no official specification that deals directly with the design of armored joints. Therefore, the AASHO specifications (6) were adopted for the purpose of establishing loads, load distributions, and impact factors for the design of armored joints, and 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 presented here. However, the tests were limited, and deviation from the 230 by 50 by 13-mm (9 by 2 by 0.5-in) armor angle or the offered anchorage would require discretely exercised engineering judgment. Basically, the important features of the armored joint, shown in Figure 1, are as follows:

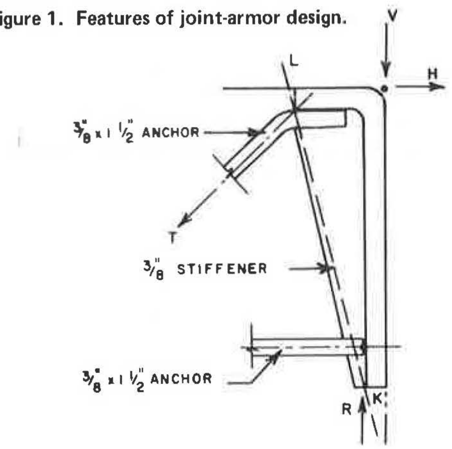
1. A small top flange to minimize incurred loads,
2. No bottom flange (as would occur with the use of a channel section),
3. Top and bottom anchors (as opposed to a single row of anchors),
4. Thirteen-mm (0.5-in) minimal thickness of armament to minimize localized deflections, and
5. Close anchor spacing (to ensure that more than one anchor takes the burden of incurred loads).

The problem of an actual stress analysis of this structurally indeterminate system was solved by the use of reasonably severe but safe assumptions based on engineering judgment. If the effectiveness of the concrete beneath the turned-down angle is neglected, a unit length of joint may be rendered statically determinate, which makes further stress analysis rather straightforward. The size and spacing of the anchorage reinforcement then follows directly from consideration of the assumed loads, which also reflect the field-test results. (In my engineering judgment, the joint armor should be designed to carry the full, dual-wheel load of the AASHO HS20-44 loading, which is sufficiently conservative to ensure a safe design.)

Because of the dynamic nature of a wheel load, the joint-armor design must also allow for the impact and frictional effects that increase the vertical load and create horizontal forces on the armor angle.

Regrettably, no dynamic load response could be ascertained in the joint-armor tests and therefore, the true impact and frictional effects, which must be considered, remained unknown. As a result, the degree of allowance for these effects is left to the designer's discretion. The design procedures reflect this; an impact factor of between 0 and 30 percent and a coefficient of friction of between 0 and 0.80 were permitted. However, in the follow-up example and in the standard drawings shown in Figures 2, 3, and 4, definite practical values for these two parameters are used. (The design procedure discussed in this paper was developed for U.S. customary units only; therefore, SI units are not given in the figures, tables, or remainder of the text.)

Figure 1. Features of joint-armor design.



Design Loads and Allowable Stresses

Figure 1 also gives a schematic representation of how the wheel load and its horizontal frictional component is applied to the joint armor. The AASHO specifications (6) for the HS20-44 loading, which is considered applicable for this joint-armor design, are given below:

Figure 2. Deck-joint plan and sections.

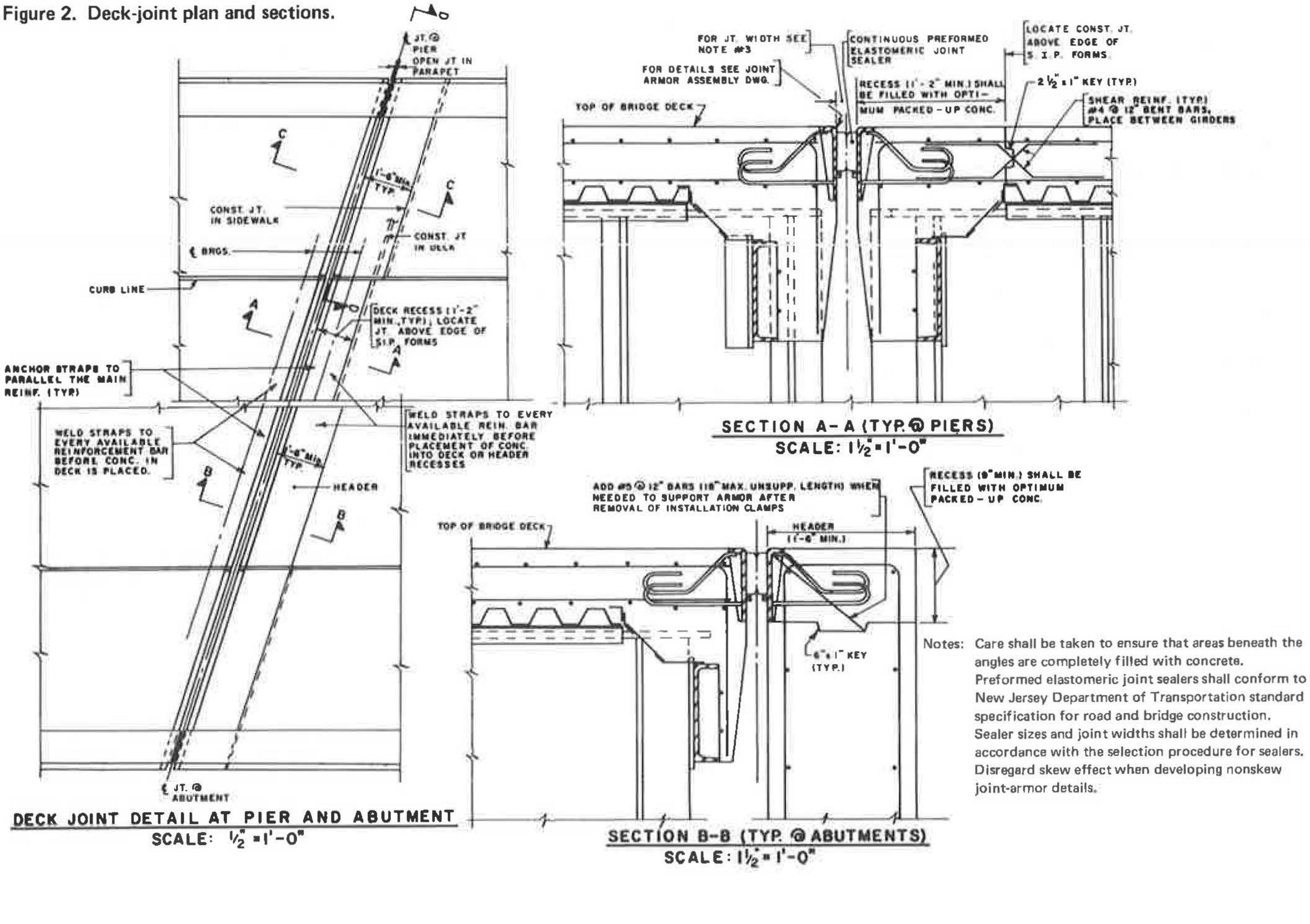
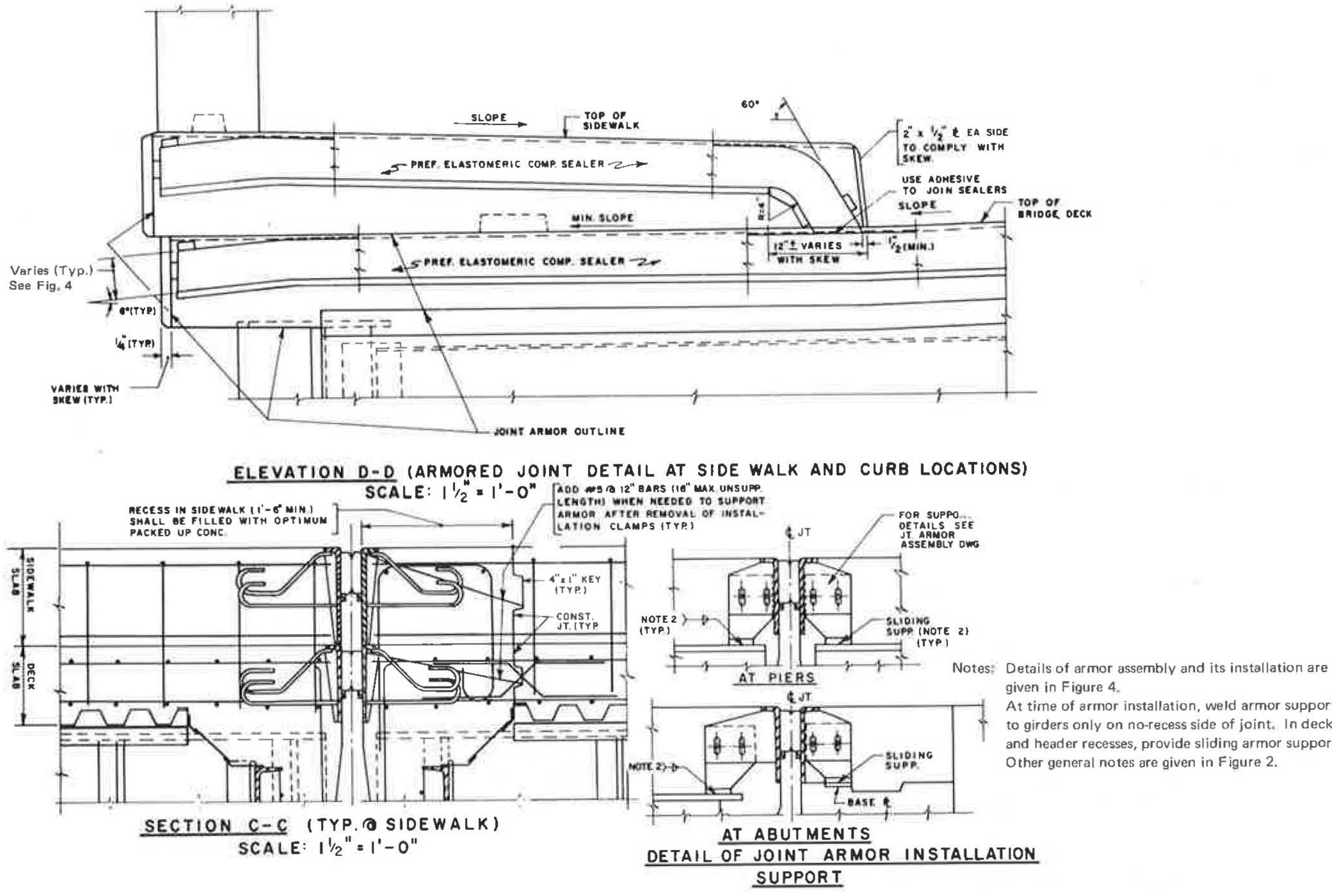


Figure 3. Sidewalk and curb elevations and sections.



Factor	Value
Wheel load, kips	16.0
Impact fraction, %	0 to 30
Friction factor, %	0 to 80

As shown by the load tests of the armored bridge joints, the load distribution can be assumed to be E = 4.0 ft. The applied loads will, of course, create stresses in the various armored-joint components and in the concrete surrounding the joint system. A safe armor design will be one that keeps these stresses below allowable limits. The applicable allowable stresses are given in sections 5 and 7 of the AASHO specifications (6).

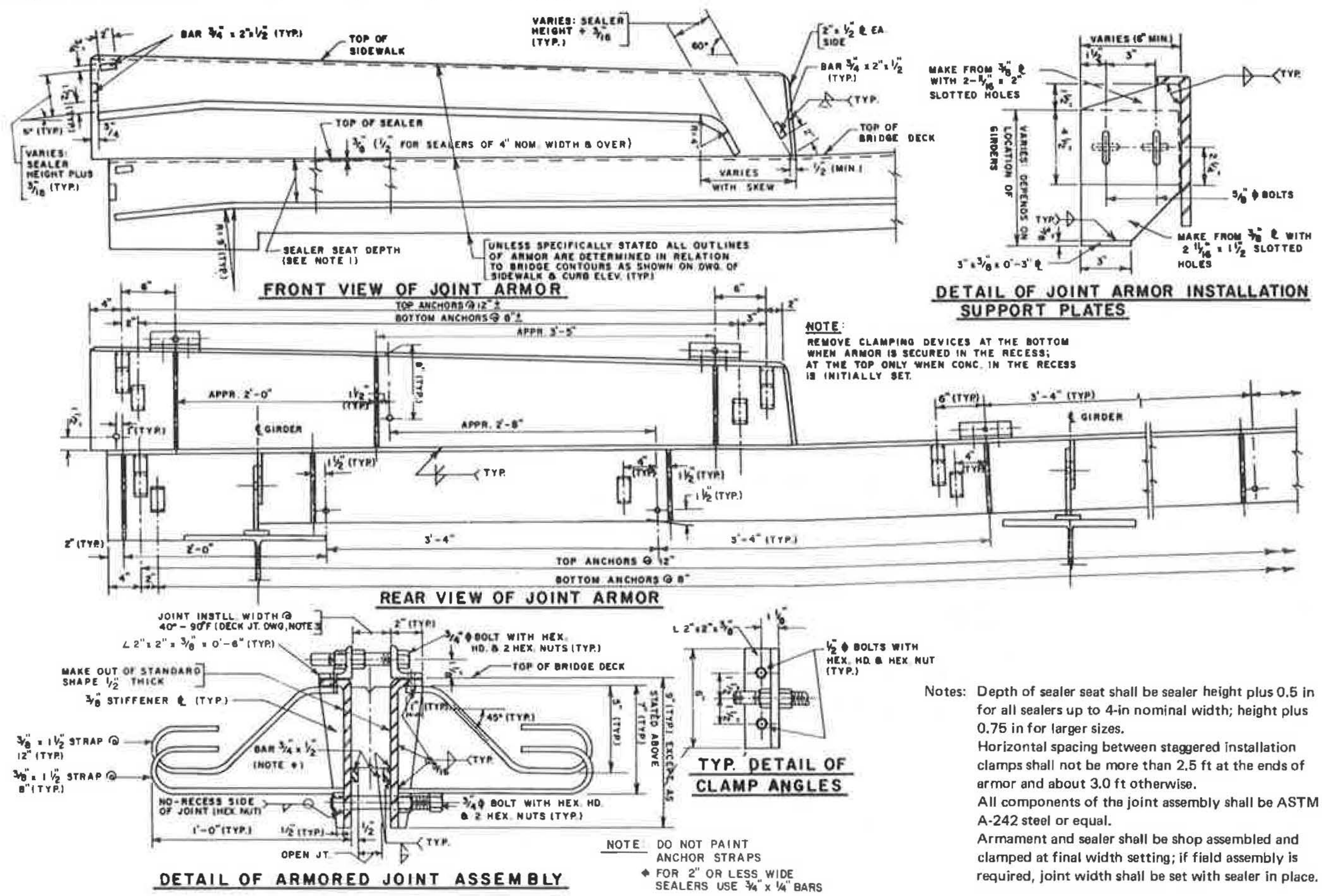
Anchorage Reactions

In Figure 1, the concrete in contact with the steel angle is considered to be ineffective, with that portion above line LK giving no support to the angle. It is quite possible that poor construction practices could produce such a condition. If this concrete is omitted 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 shown in Figure 1. The magnitudes of the reactions can be computed from straightforward analysis of the static equilibrium conditions.

In the diagrams and formulas and their derivations, the following notation is used:

V = vertical load (wheel load),
H = horizontal load,
T = top anchor reaction,
T_H = horizontal component of T,
T_V = vertical component of T,
R = bottom anchor reaction,
R_H = horizontal component of R,
R_V = vertical component of R,
I = impact fraction,
C = friction factor,
E = distribution factor (taken as 4 ft),
W_L = resultant of shearing stress in the weld,
W_R = resultant of bearing stress at end of anchor,
P_R = total allowable load to be carried by cross weld between top of anchor and angle,
D = leg size of fillet welds (in),
L = effective length of weld (in),
L_1 = effective length of cross weld (in),
n = number of anchors/ft of length,
a = thickness of anchor (in),
b = width of anchor (in),
A = area of anchor (in^2),
L_{bond} = effective bond length (in),
L_{bear} = effective bearing length (in),
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 = vertical component of combined unit stress, and
f_h = horizontal component of combined unit stress.

Figure 4. Joint-armor assembly—details, sections, and elevations.



Notes: Depth of sealer seat shall be sealer height plus 0.5 in for all sealers up to 4-in nominal width; height plus 0.75 in for larger sizes.
Horizontal spacing between staggered installation clamps shall not be more than 2.5 ft at the ends of armor and about 3.0 ft otherwise.
All components of the joint assembly shall be ASTM A-242 steel or equal.
Armament and sealer shall be shop assembled and clamped at final width setting; if field assembly is required, joint width shall be set with sealer in place.

The applied loads are given by the following equations:

$$V = 16.0(1 + I) \tag{2}$$

$$H = 16.0 \times C \tag{3}$$

both in kips per E, and

$$V = (16/E)(1 + I) = 4.0(1 + I) \tag{4}$$

$$H = (16/E) \times C = 4.0 \times C \tag{5}$$

both in kips per feet.

The geometric relations between the loads and the anchor-ban reactions for the specific steel angle proposed and the suggested location and orientation of the anchor bars are shown in Figure 5 (the horizontal force can act in either direction). By using the laws of statics and taking Σ(moments about B), Σ(horizontal forces), and Σ(vertical forces), the following equations for the reaction components (T_H, T_V, R_H, and R_V) and the anchor reactions (T and R) are readily developed. (Selection of an armor and anchorage system having different geometry and dimensions from that shown in Figure 5 would result in different equations and reactions.)

$$T_H = T_V = (0.5/8.3)V \pm (7.3/8.3)H \tag{6}$$

$$R_H = (0.5/8.3)V \pm (7.3/8.3) \mp H \tag{7}$$

$$R_V = V + (0.5/8.3)V \pm (7.3/8.3)H \tag{8}$$

$$T = \sqrt{2}\, T_V \tag{9}$$

$$R = (R_V^2 + R_H^2)^{1/2} \tag{10}$$

The anchor reactions derived by inserting appropriate load values for V and H in the equations and considering the possible ranges of impact and friction factors are shown in Tables 1 and 2 and Figure 6. In Figure 6, for design purposes, the R_H and minimum R_V values are disregarded; R_H is negligibly small and 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.

Application Example

Assume that an armament and anchors of the size and configuration shown in Figure 1 are selected for design, and select an impact factor of 30 percent and a maximum horizontal friction factor of 80 percent. The anchor reactions (per linear foot of joint)—R and T—can be found from Table 2 or Figure 6. The size, spacing, and welding requirements are determined as shown below, and the final, detailed joint design is that shown in Figures 2, 3, and 4 and requires the following: T = 4.42 kips and R_V = 8.327 kips.

1. The welding requirements can be determined from the procedures given in a standard design manual (7). (These procedures are described in the manual as following a middle course and yielding results that appear reasonable by test results.) From Figure 7, in the top anchors, $W_L = W_R = {}^5\!/_{12} f_V L$ and $P_B = 0.707 f_{all} D L_1$. The moment about W_R is $(T_V - P_R) \times {}^{17}\!/_{18} L = W_L \times {}^2\!/_3 L \times 2$, and therefore, f_V =

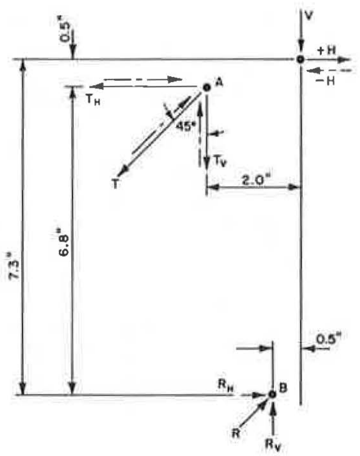


Figure 5. Geometric relations between loads and anchor-bar reactions.

$^{17}\!/_{10}(T_V - 0.707 f_{all} DL_1)/L$; $f_h = T_H/(2L + L)$; and $f_r = (f_v^2 + f_h^2)^{1/2}$.

2. If the top anchor spacing is 12 in on center (OC) (determined by the field-load tests), $T_V = T_H$, $L_1 = 1.5$ in, $L = 1.0$ in, $D = {}^5\!/_{16}$ in, and $f_{all} = 12.4$ kips/in^2. Then, by using the formulas for f_v, f_h, and f_r, we can solve for: $T_H = 5.45$ kips and $T = T_H\sqrt{2} = 7.71$ kips > 4.42 kips (Figure 6).

In the bottom anchors (for a spacing of 8-in OC), $n = {}^{12}\!/_8 = 1.5$, and $a = 0.375$ in, $b = 1.5$ in, $f_{all} = 12.4$ kips/in^2, and $R_V = 0.707 f_{all} D \times 2(a + b) \times n = 15.45$ kips > 8.327 kips (Figure 6).

3. The shearing stresses in the bottom anchors [for an 8-in OC spacing ($n = 1.5$) and $f_{sv} = 12.0$ kips/in^2] are $R_V = f_{sv} \times A \times n = 10.13$ kips > 8.327 kips.

4. The bearing stresses in the bottom anchors [assuming a triangular bearing distribution with available $L_{bear} = 10.5$ in, 8-in OC spacing ($n = 1.5$), and $f_{bear} = 0.7$

Table 1. Components of joint-armor design-load reactions.

			$T_V = T_H$ (kips)		R_H (kips)		R_V (kips)	
I(%)	V (kips)	H (kips)	+H	-H	+H	-H	+H	-H
0	4.00	0.0 to 3.20	0.241 to 3.055	0.241 to -2.573	0.241 to -0.145	0.241 to 0.627	4.241 to 7.055	4.241 to 1.427
5	4.20	0.0 to 3.20	0.253 to 3.067	0.253 to -2.561	0.253 to -0.133	0.253 to 0.639	4.453 to 7.267	4.453 to 1.639
10	4.40	0.0 to 3.20	0.265 to 3.079	0.265 to -2.549	0.265 to -0.121	0.265 to 0.651	4.665 to 7.479	4.665 to 1.851
15	4.60	0.0 to 3.20	0.277 to 3.091	0.277 to -2.537	0.277 to -0.109	0.277 to 0.663	4.877 to 7.691	4.877 to 2.063
20	4.80	0.0 to 3.20	0.289 to 3.103	0.289 to -2.525	0.289 to -0.097	0.289 to 0.675	5.089 to 7.903	5.089 to 2.275
25	5.00	0.0 to 3.20	0.301 to 3.115	0.301 to -2.513	0.301 to -0.085	0.301 to 0.687	5.301 to 8.115	5.301 to 2.487
30	5.20	0.0 to 3.20	0.313 to 3.127	0.313 to -2.501	0.313 to -0.073	0.313 to 0.699	5.513 to 8.327	5.513 to 2.699

Note: Applied vertical load = 4.0 kips/ft, applied horizontal load = (4.0 x C) kips/ft, and C = 0 to 80%.

Table 2. Joint-armor design-load reactions.

			T (kips)		R (kips)	
I (%)	V (kips)	H (kips)	+H	-H	+H	-H
0	4.00	0.0 to 3.20	0.341 to 4.320	0.341 to -3.639	4.248 to 7.056	4.248 to 1.559
5	4.20	0.0 to 3.20	0.358 to 4.337	0.358 to -3.622	4.460 to 7.268	4.460 to 1.759
10	4.40	0.0 to 3.20	0.375 to 4.354	0.375 to -3.605	4.673 to 7.480	4.673 to 1.962
15	4.60	0.0 to 3.20	0.392 to 4.371	0.392 to -3.588	4.885 to 7.692	4.885 to 2.167
20	4.80	0.0 to 3.20	0.409 to 4.388	0.409 to -3.511	5.097 to 7.904	5.097 to 2.373
25	5.00	0.0 to 3.20	0.426 to 4.405	0.426 to -3.554	5.310 to 8.115	5.310 to 2.580
30	5.20	0.0 to 3.20	0.443 to 4.422	0.443 to -3.537	5.522 to 8.327	5.522 to 2.788

Note: Applied vertical load = 4.0 kips/ft, applied horizontal load = (4.0 x C) kips/ft, and C = 0 to 80%.

Figure 6. Joint-armor design-load reactions.

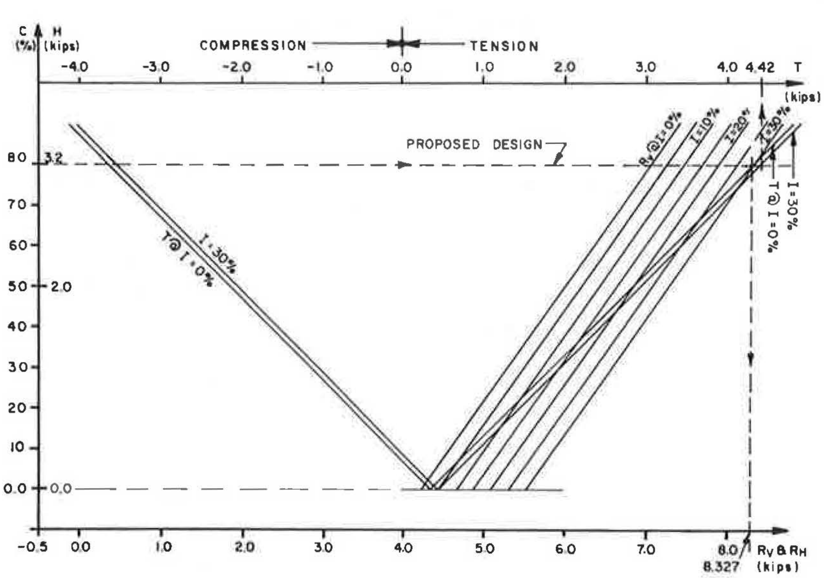
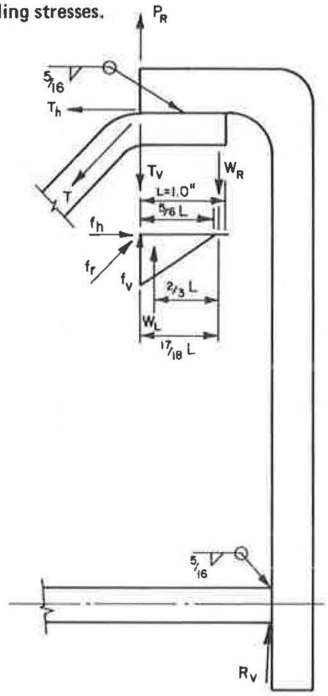


Figure 7. Welding stresses.



kips/in^2], will be $R_V = f_{bear} \times b \times (L_{bear}/2) \times n = 8.265$ kips, which is sufficiently close to 8.327 kips.

5. The tension stresses in the top anchors (for 12-in OC spacing and $f_s = 20$ kips/in^2) will be $T = f_s \times A = 11.25$ kips > 4.42 kips (Figure 6).

6. The bond stresses (neglecting the bottom anchors because R_H is negligible) in the top anchors (assuming that hook develops 50 percent of allowable stress) will be $T = f_{bond} \times [2(a + b) \times L_{bond} \times 2] = 1.20\ L_{bond}$, and if $L_{bond} = 7$ in, $T = 8.4$ kips > 4.42 kips.

DISCUSSION OF DESIGN

This design is for maximum impact and close to maximum friction factors; however, the selection of these factors is left to the discretion of the engineer. The impact factor should not be less than 30 percent, but because the wheel load of 16.0 kips has been conservatively chosen, the friction factor of 40 to 50 percent should be more reasonable.

As shown in Figure 6 and described in Tables 1 and 2 and the preceding design analysis, it is the bearing in the bottom anchors that controls their spacing. The proposed standard drawing shown in Figures 2, 3, and 4 shows that the only practical bottom anchor spacing that will prevent interference with other elements would be 4, 6, 8, or 12 in because the spacing of the top anchors cannot exceed 12 in and the approximate spacing of clamping devices should be 3.0 ft to control armor-angle deflections before installation.

The design discussed above uses an 8-in bottom-anchor spacing. For a 12-in spacing and I = 30 and C = 40 percent respectively, Figure 6 gives $R_V = 6.92$ kips required.

The bearing analysis for 12-in spacing gives $R_V = 5.51$ kips < 6.92 kips. A similar calculation with 8-in bottom spacing gives $R_V = 8.265$ kips > 6.92 kips. Therefore, 8-in bottom-anchor spacing is shown in Figures 2, 3, and 4.

Finally, the basic design requirements must include the following considerations:

1. Armored deck joints should be continuous throughout the full width of the deck, and termination should be accomplished as shown in Figures 2, 3, and 4 to armor the joints where necessary and to form the best sealed joint possible.

2. The seal groove in the sidewalk should be armored in the same manner as the curb and the outside ends should be installed as shown in Figures 3 and 4, but a stay-in-place anchor seat could be added to the curb end at the bottom outside face of the armor shapes.

3. All the steel at the armor network except for the parts in contact with concrete should be shop painted and touched up in the field after removal of the armor-holding elements. ASTM A 242 or A 588 steel is recommended for the armor because its stable rust characteristics will be advantageous in those areas where paint is likely to deteriorate.

Procedure for Header Design

Failure of headers is not uncommon. The probable causes are as follows: (a) loading, such as indicated in armor design; (b) inadequate preparation of the backfill; and (c) concrete approach slabs directly supported by headers. Only one remedy can be suggested—improvement of quality control in construction—for problem (b). In view of these causes, the severity of the assumptions made below is well justified. Also, the stresses produced on the commonly used section discussed below are well within the practical range.

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, I = 30 percent, C = 1.0, and E = 3.0 ft; thus, $V = (16.0/E)(1 + I)$ kips/ft, and $H = (16.0/E) \times C$ kips/ft.

Analyses of Applied Loads

To determine the reinforcement sizes and spacing and the basic header dimensions, an analysis (8) of the applied loads is necessary. This analysis (Figure 8) uses the following additional notation:

- P_s = reasonable estimate of sealer load (1.0 kip/ft) when compressed to 50 percent,
- w = unit weight of concrete,
- N = total vertical load (axial load),
- M = moment,
- p = load on reinforcement,
- b′ = header width,
- h = 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, and
- j = ratio of distance (jd) between resultants of compressive and tensile stresses to effective depth.

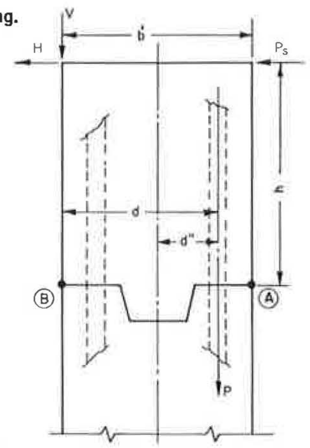
The moment about plane AB, the total vertical load, and the reinforcement design (8) are given by Equations 11, 12, and 13 and 14 respectively.

$$M = V \times (b'/2) + (H + P_S) \times h \quad (11)$$

$$N = V + (w \times b' \times h) \quad (12)$$

$$e = (12M/N) + d'' \quad (13)$$

Figure 8. Header loading.



where e is in inches

$$p = N(e - jd)/jd \qquad (14)$$

where p is in kips per foot of width.

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

The headers used with joints sealed by preformed elastomeric sealers must be designed as absolutely stationary for the sealers to function properly. There can be no horizontal movement and no rotation of the header because the sealer and the joint width are selected on the basis of predictable movements; i.e., on the basis of bridge-deck expansion only.

Because of this, approach slabs, which are supported on one end elastically and on the other by a vertically rigid member, present a complex problem. This is especially so if the rigid slabs have an eccentrically located, static, vertical load and possibly a substantial horizontal static force, as well as other dynamic reactions to a horizontally flimsy header. 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 studied, the backfill was adequately compacted, and bituminous pavement was substituted for approach slabs.

Construction Procedure for Armored Joints

The concept on which this method is based is that the entire system (armor angles with straps and seats welded to them and sealer properly precompressed between the angles and the supporting elements such as clamps and attached bolts) is assembled and then placed in the joint before the concrete is poured. The best approach is to have the elements of the system fully assembled, delivered to the construction site, and placed true to its elevations, joint widths, and proper position in the bridge deck. (The width of the joint between armors, adjusted according to the design requirement, and all other pertinent information is shown in Figures 2, 3, and 4.) If complete factory assembly is not feasible, the joint should be factory preassembled to the fullest practicable degree and the assembly completed on the construction site. Standard lubricant should be applied on each joint-armor face when the sealer is located in 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 2 and 3, the deck should be poured without a recess on only one side of the joint. The necessary recess should be left on the abutment (or header) side of the joint, or in a movable deck end when the joint is between spans, with the deck (or header) reinforcement properly extended into the recess. The recess area should be the last concrete poured.

On the side of the joint on which 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 the available deck reinforcement bars only after those bars are checked for proper placement; this ensures stress-transfer continuity into the concrete deck, which can be poured any time thereafter.

From the initial setting of the assembly to the final pouring of concrete into the recess, the armor supporting plates on the recess side must slide freely 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 the deck concrete has stabilized by curing, shrinking, and camber settlement (about 1 week after pouring), the recess may be poured.

On the day when concrete is poured into the recess, there are three steps that should be performed during the 2 h immediately preceding the actual pour.

1. Make a final check of the joint opening at each top clamp and adjust it if necessary.
2. Weld the anchorage straps to every available reinforcement bar and to the auxiliary no. 5 by 12-in support bars. This is done to ensure stress-transfer continuity and also to prevent the joint from opening before interaction between the armor and the concrete is ensured. Curb and gutter areas present the greatest problem in this regard.

Because the two sides of the armored joint are fastened to each other, as well as to their respective deck spans, the importance of pouring the concrete immediately is obvious.

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

The entire pour requires about 1 h. Immediately after the initial set of the concrete, the top clamps of the assembly should be removed, and the concrete in these areas should be refinished if necessary. This procedure will provide a satisfactorily sealed joint with a minimal amount of care.

In summary, the successful construction of this type of bridge joint has the following basic requirements:

1. The structural integrity of the armor must be preserved; i.e., it must be fabricated and constructed exactly in accordance with the drawings and specifications.
2. 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.
3. Precise placement of the armor is absolutely essential; i.e., before the concrete is poured, the armor must be located true to the bridge-deck surface. The installation should be performed so that until the concrete is set, no bridge-end movements are transferred into the joint armor.

Selection Procedure for Sealers

The procedure for the selection of sealers is an em-

Figure 9. Δ-movement for L = 0 to 180 ft.

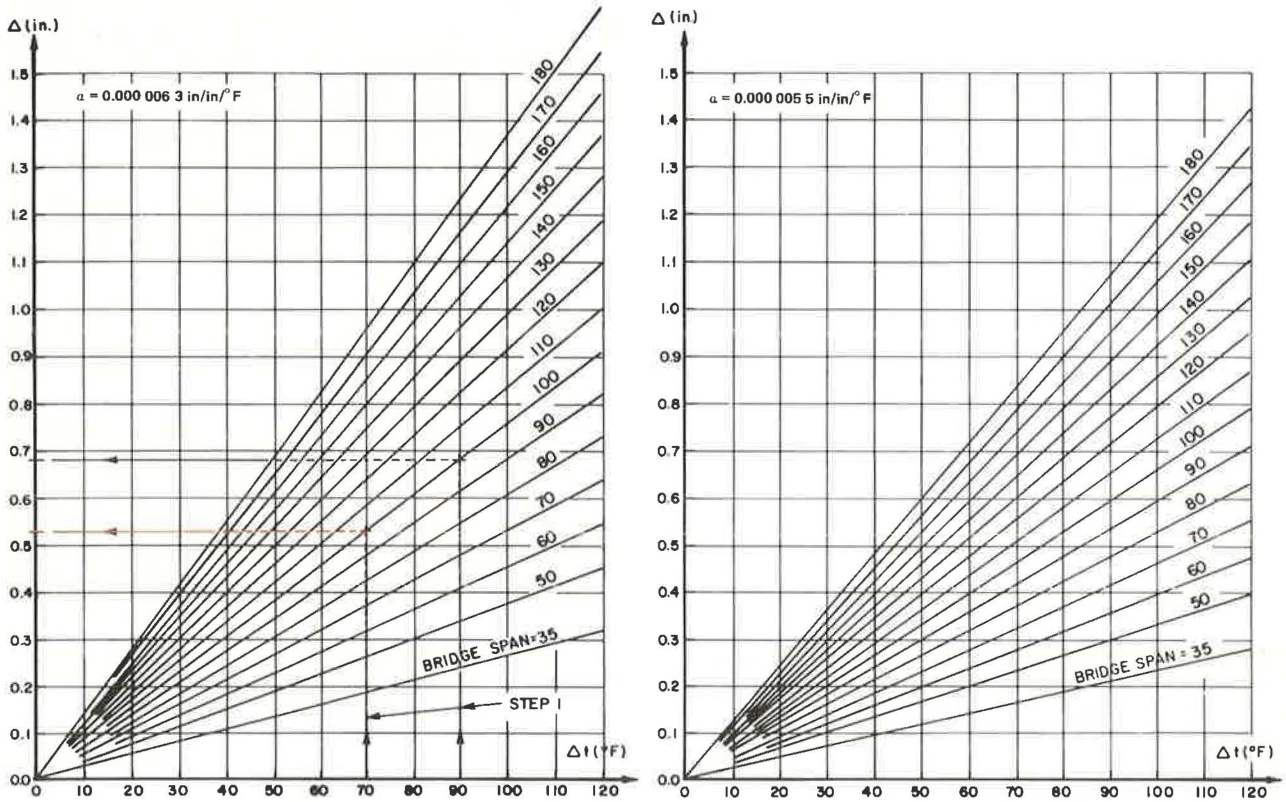
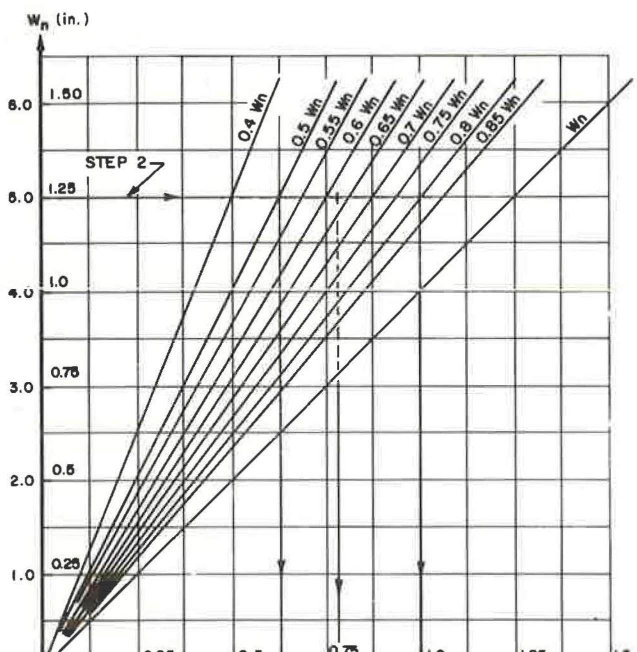


Figure 10. Joint-sealer efficiency chart.



pirical method that establishes the size of sealer to be used in a joint and determines the width at which the joint must be constructed to ensure the effectiveness of the sealer. The capabilities of the sealer are determined in terms of three empirical efficiency coefficients. Each of these parameters is the ratio of the width of the sealer at a certain level of compression to its nominal width (W_n), multiplied by 100. Z is the ratio at the maximum permitted compression of the sealer; Y is the desired ratio at the time of sealer installation; and X is the ratio at the minimum permitted compression of the sealer (enough compression to prevent leakage between sealer and joint face). In New Jersey, X cannot be greater than 80 percent. Z should be not more than 50 percent and, therefore, Y should be approximately 60 to 65 percent.

Any bridge joint will be constructed at a width that is preset 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 cannot be known, but it is known that all subsequent bridge movements will occur from that preset width. Therefore, the design consists of establishing the maximum magnitude of these movements (Figure 9) and selecting a sealer and construction joint width on the basis of the values of X and Z.

The following calculation illustrates the use of Figures 9 and 10 by a solution for a composite bridge design with a span of 100 ft.

1. Assume (a) an ambient temperature range of 0 to 110° F and a construction ambient temperature range during installation of the joint armor of 40 to 90° F (the use of a construction air temperature range, rather than a particular value, is the only realistic approach to existing construction practices), (b) Z = approximately 50 percent at the minimum width of joint (W_{jmin}) and 110° F, (c) Y = approximately 60 percent at the installation width of joint (W_{jinst}) and a construction air temperature range of 40 to 90° F, and (d) X = approximately 80 percent at the maximum width of joint (W_{jmax}) and 0° F.

2. For a joint installation temperature of 40° F and a subsequent maximum bridge temperature of 110° F,

Table 3. Guide to design of sealers in composite and steel bridges.

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

Notes: α = 0.000 006 3 in/in/°F.
Controlling temperature range = 0 to 110°F and construction temperature range = 40 to 90°F (these are the actual ambient temperatures at a bridge site studied).
Z = approximately 0.50 W_n, Y = approximately 0.60 to 0.65 W_n, and X = approximately 0.80 W_n.

Table 4. Guide to design of sealers in concrete bridges.

Limit of Span (ft)	W_n (in)	At 110°F: W_{jmin} (in)	Z	Δ At Δt = 70°F	At 40 to 90°F: $W_J \pm \frac{1}{16}$ in Tolerance (in)	Y	Δ At Δt = 90°F	At 0°F: W_{jmax} (in)	X
≤35	$1\frac{1}{2}$	0.875 to 0.715	0.58 to 0.48	0.00 to 0.16	$\frac{15}{16}$	0.625	0.00 to 0.21	1.00 to 1.21	0.67 to 0.81
35 to 40	$1\frac{3}{4}$	0.90 to 0.88	0.515 to 0.50	0.16 to 0.18	$1\frac{1}{8}$	0.64	0.21 to 0.24	1.40 to 1.43	0.80 to 0.82
40 to 50	2	1.01 to 0.96	0.51 to 0.48	0.18 to 0.23	$1\frac{1}{4}$	0.625	0.24 to 0.30	1.56 to 1.61	0.78 to 0.81
50 to 65	$2\frac{1}{2}$	1.27 to 1.20	0.51 to 0.48	0.23 to 0.30	$1\frac{9}{16}$	0.625	0.30 to 0.39	1.93 to 2.02	0.77 to 0.81
65 to 80	3	1.51 to 1.44	0.50 to 0.48	0.30 to 0.37	$1\frac{7}{8}$	0.625	0.39 to 0.48	2.33 to 2.43	0.78 to 0.81
80 to 110	4	2.07 to 1.93	0.52 to 0.48	0.37 to 0.51	$2\frac{1}{2}$	0.625	0.48 to 0.65	3.04 to 3.21	0.76 to 0.80
110 to 140	5	2.55 to 2.41	0.51 to 0.48	0.51 to 0.65	$3\frac{1}{8}$	0.625	0.65 to 0.83	3.84 to 4.02	0.77 to 0.80
140 to 170	6	3.04 to 2.90	0.51 to 0.48	0.65 to 0.79	$3\frac{3}{4}$	0.625	0.83 to 1.01	4.64 to 4.82	0.77 to 0.80

Notes: α = 0.000 005 5 in/in/°F.
Controlling temperature range = 0 to 110°F and construction temperature range = 40 to 90°F (these are the actual ambient temperatures at a bridge site studied).
Z = approximately 0.50 W_n, Y = approximately 0.60 to 0.65 W_n, and X = approximately 0.80 W_n.

Δt = 70° F. From Figure 9a, if Δt = 70° F and L = 100 ft, Δ_1 = 0.52 in. However, if the installation temperature is 90° F, the bridge may subsequently cool to 0° F and Δt = 90° F, and from Figure 9a, if Δt = 90° F and L = 100 ft, Δ_2 = 0.68 in. Although 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 160° F.

3. Estimate a sealer size for W_n = 5.0 in, Z = $0.5W_n$, and X = $0.8W_n$. Figure 10 gives W_{jmax} - W_{jmin} = 4.00 - 2.50 = 1.50 in. The preset width of the joint has a tolerance of $\pm\frac{1}{16}$ in, which effectively increases the required sealer movement range at each end by $\frac{1}{16}$ in, and therefore, the required $\Delta = (\Delta_1 + \frac{1}{16}) + (\Delta_2 + \frac{1}{16}) = 1.32$ in < 1.50 in.

4. The joint installation width would be between 3.26 (4.00 - 0.74) and 3.08 (2.50 + 0.58) in, and $3\frac{1}{8} \pm \frac{1}{16}$ in is an appropriate choice of $W_{jconstr}$.

The preceding steps illustrate the principles underlying sealer selection. Each designer can choose particular values of α, ambient air temperatures, and limits for X and Z with which to construct tables (such as Tables 3 and 4), from which the required sealer and armored-joint installation widths can be easily obtained for various bridge-deck lengths.

Considerations for Replacement of Sealers

Replacement of sealers is ill-advised unless it is performed with great care; it involves considerable expense and inconveniences the riding public. Thus, if a sealer needs replacement, the cause of its demise should first be determined. But generally, if the joint is not a proper, intact armored joint, then achieving the best results will require the use of the armored-joint construction method described above as the replacement procedure. This could be performed by removing concrete at the joint to provide a sufficient recess for the armored joint assembly and then following the normal armored-joint construction procedure.

The replacement of a sealer in a properly constructed armored joint could be due to failure of the sealer itself or to the addition of an overlay on the bridge deck. In the first case, the sidewalk concrete at the joint would be removed and the armament spread to facilitate the removal of the old and the installation of the new deck sealer. In the latter case, it would be necessary to build up the top lateral surface of the existing deck armament and to add higher sealer seats.

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The contents of this report reflect our views; we are responsible for the facts and the accuracy of the data presented here. The contents do not necessarily reflect the official views or policies of the state of New Jersey or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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