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PREFORMED ELASTOMERIC BRIDGE JOINT SEALERS

INTERIM GUIDE FOR DESIGN AND CONSTRUCTION OF JOINTS

by

George S. Kozlov

8/70

NEW JERSEY DEPARTMENT OF TRANSPORTATION

DIVISION OF RESEARCH AND EVALUATION

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ABSTRACT

As a result of several years of research which culminated in the construction of two experimental bridges, it now becomes possible to present engineers with procedures for the design and construction of adequately sealed joints. These procedures are offered as an interim solution until present on-going research can be completed.

The paper suggests armored joint construction sealed with preformed elastomeric sealers as the most advantageous solution to the problem of sealing joints in bridges.

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INTRODUCTION

In 1965 the New Jersey Department of Transportation initiated a research study dealing with Preformed Sealers for Bridge Decks. The initial accomplishments of this study were presented in three previous papers (1,2,3). In the first of these articles it was stated that "the striking fact is the lack of an adequate solution to this problem" (sealing of joints in bridges). In this and the subsequent papers a succession of solutions was offered covering the design of sealers and joints, the application of the design, as well as the thermal characteristics of bridge end movements. Since that time two structures have been built utilizing experimentally these suggested design and construction procedures.

On two bridges of the New Jersey state route system (Bridges #1 and #5, Route #29, Section 12b-11a), all joints were redesigned and constructed as recommended by the research. Armored and sawed joint construction was utilized for expansion and fixed joints respectively.

Basically, the results of this experiment shall be the subject of this paper.

THE EXPERIMENT

The subject experiment was developed and executed utilizing as a basis the techniques outlined in the first two papers (1,2). The specific procedures used are summarized in the attached Appendices "A", "B" and "C" which describe, respectively, the design of the sealer, the design of the joint armor, and construction procedures of an armored joint. The sawed joint procedure is omitted because it proved ineffectual and it's implementation is not going to be recommended. As part of the experiment the sealer material was evaluated in accordance with latest New Jersey Department of Transportation Preformed Sealer specifications, which were also discussed in the second paper (2) under the chapters on "Material Specification" and in the Appendix.

Unfortunately, there were deviations in design and especially in execution of construction from that recommended by Research, either by mutual agreement but mostly due to circumstances beyond the control of the writer, who is the principal researcher for this study. The actual design and the construction of joints and installation of sealers was performed as is usually practiced by the New Jersey State Department of Transportation, i.e.,for the design to be accomplished by consultants. The result was that the joint armament was designed by a consultant, who, for example, specified much too excessive an anchor spacing, 18 inches, and supplied supplemental construction drawings with even less information on them than the instructional plans originally offered by research. The construction was, on the other hand, quite often performed in complete disregard to these supplemental drawings.

Due to the persistent insistence of the research observers, the anchorage was supplemented by welding every available reinforcement bar to the joint's armor, while the obvious construction errors were remedied one way or another. Regrettably, some construction deviations, such as in the forming and sawing of joints, could not be remedied, thus jeopardizing to some degree the efforts. These were the basic observations which are mentioned here in order to substantiate the conclusions and recommendations that follow.

The two experimental bridges have now been open to traffic for almost one full year. In the spring of this year a Dye Test was performed on these bridges for the purpose of detecting joint leakage. The description and analysis of this test are provided in Appendix "D".

CONCLUSIONS

The principal conclusion of this experiment, from the point of view of the researcher, is that the basic design and its application is a success. The behavior of joints, sealers, etc., even the malfunctioning of them is as anticipated.

It is time that some of us recognize and all of us admit the realities of joint design and construction. There is NO MATERIAL AND NO METHOD OF ITS APPLICATION THAT WILL BE ABLE TO SUCCEED UNLESS IT CAN OVERCOME TOTALLY INADEQUATE QUALITY CONTROLS IN CONSTRUCTION. For this reason alone the formed and sawed joint methods of construction must fail. In fact, in the opinion of the writer, unless joint-sealing and construction is placed into the hands of specialists, no further advancement in this field can be expected and without adequate construction supervision even this will not succeed. The additional conclusions and design recommendation that follow are believed to reflect to the utmost the acquired knowledge and the recognition of the existing realities in the design and construction fields.

Specifically, these are the conclusions derived from the study of the two experimental bridges:

1. To date the sawed and armored joints on both subject bridges do not leak.
2. The experimental design approach initiated on these bridges has proved its merit. Briefly, basic design principles are:
   - 2.1 Main sealers are placed out to out in a straight deck joint.
   - 2.2 Sidewalk sealers are placed also out to out, i.e., bottom of curb to outside with only one vertical shallow bent (60°) at the curb. For details see Figures B2 through B5.
3. Sawed joints are functioning well because, in spite of the rather poor quality of construction, they are "fixed joints" in which the advantages of design developed by Research are clearly manifested. To remedy the results of poor sawed joints construction the sealers in some of them might be replaced. To this and any

other similar effort the following word of caution is offered: When sealers in fixed joints (sawed) are replaced, as they might and should be in at least Bridge #1, care must be taken not to jeopardize the functional efficiency of the replacement sealers. After the sealer is removed, joints must be thoroughly cleaned and adequately repaired and prepared. Immediately thereafter a proper size continuous sealer should be installed in accordance with the originally established procedures. Sufficiently prior to installation, sealers must be tested and approved by the Department's Material Division laboratory. If the contractor is not adequately supervised, one can be sure of gross violations on every step.

4. The armored type of joint design is the most advantageous of existing solutions to the joint design problem.

RECOMMENDATIONS

Based on the preceding conclusions summarized above, it is only logical to suggest the following actions:

1. Adoption of bridge joint design approach as outlined in the New Jersey State Report #29 and later published in HRR #200 & #287 and once more summarized in Appendix "A". In both experimental bridges, design and development of joints was fashioned in accordance with suggestions made in the report and papers.
2. Adoption of the design and construction procedures for joint armor as originally suggested by the researcher. For clarification please see Appendix "B" and "C". For further illustration of armored joint

details design and development, Waldemar Koester's book (4) is recommended.

3. For bridges with spans larger than indicated either in Table A1 or A2 in Appendix "A" experimental installation of "modular sealing system" advocated by S. C. Watson (5) should be attempted. Its design approach is similar to the one suggested above.

4. Concerning the design of joint armor some additions to the above suggestions are further offered. In the opinion of the writer the best guidelines for design of armor are provided in the "Technical Memorandum (Bridges) No. BE6" by A. D. Holland, Deputy Chief-Engineer, Bridges, Engineering Division, Ministry of Transport, England (6), with following clarifications by Mr. L. G. Deuce, Assistant Chief Engineer, in behalf of Mr. Holland:

   4.1 Regarding Clause 7e, the use of anchor bars was not generally recommended because (from the Survey of Expansion Joints in the UK mentioned in BE6) such details did not appear to be altogether satisfactory and principally because they are difficult to repair when they do prove to be faulty. <u>However, soundly constructed joints designed in accordance with the instructions given in Clause 7e should be satisfactory</u> and it is not proposed to repeat the first sentence of this sub-clause in the revised edition of the Memo.

   4.2 In Clause 7d "Base plates with holding down bolts", the word "base" is being dropped and <u>it was not intended to limit the application of the rules from applying to</u>

the turned down angle type of armouring. It has a particular connotation where additional surface plates are bolted to anchored base plates.

4.3 With regard to the loads to be used in the design of the armor Mr. Deuce further writes - the vertical loading is taken directly from BS 153(7) and has not so far been substantiated by actual loading measurements on joints. The horizontal loading was recommended as a result of the Survey of Expansion Joints.

(please see also the note following Clause 7d iii).

In the U.S.A. there seems to be no specification available that is directly involved in the design of Armored Joints. For this reason to fill at least an interim need, the writer has adopted the existing AASHO specification (8) for this purpose, as shown in the Appendix "D".

5. Future Research - Dynamic Load Tests for Armored Bridge Joints:

Although various types of armored joints are offered, their design is often questionable from structural as well as functional points of view.

Considering the structural aspect of the design of an armored joint there are basically two problems which may lead to either structural failure or, on the other hand, gross over-design. The first problem is that of determining accurate load distribution factors and dynamic load and impact factors. The second problem is in the actual stress analysis of the structurally indeterminate armored joint.

One approach to these design problems is that of designing and building an armored joint based on existing techniques and knowledge, instrumenting the structure for stress (strain) and load determination, and running live load tests on the joint. This may be accomplished via the use of electric strain gages and electronic pressure transducers and dynamic response recording equipment such as visual, multi-trace oscilloscopes. In general, in order to get a detailed picture of loads and strains, electronic strain gages would be placed in areas of anticipated maximum strains on anchorage members of the armored joint and perhaps pressure cells could be placed on the under-side of the armor.

The instrumentation would take in at least 3 feet of joint in order to include the effect of distribution. Live load tests may consist of a heavily loaded truck of known wheel loads passing over the joint at various speeds until the worst (reasonable) case is encountered; or, they may simply consist of "typical" highway traffic after the joint is opened to traffic.

The strain gage results would be used to determine maximum stresses in the members of the armored joint. The load cells would be used to determine "effective" load areas or load distribution factors:

On the basis of data, it is believed enough information could be gathered not only to identify the design method but also more importantly, the loads and their distribution.

It could evolve into a rather expensive research, but to the extent as planned here it need not be. Anyhow, this research, if successful, could pay for itself many, many times over. It must be realized that this is an integral part of current research efforts to develop properly sealed bridge joints; in view of this, the cost benefits of this particular branch of research may obviously be quite substantial on a long-term basis. However, at this time the funds and opportunity to materialize these tests are lacking.

SIGNIFICANCE OF FINDINGS AND GENERAL CONCLUSIVE COMMENTS

This experimental construction includes only two bridges each having only one simple span, yet far reaching conclusions and broad recommendations have been made in this paper. This is possible because these experimental installations culminate several years of research effort in this field. The recommendations made here are presented only as guide lines for which, in the absence of any other similar data, the need is great. As already indicated, these are offered as an interim solution until more scientific factual information becomes available.

ACKNOWLEDGMENT

The author is grateful to the personnel of the New Jersey Department of Transportation for giving him assistance in gathering and evaluation of data for this paper.

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8. AASHO, 1969, Standard Specifications for Highway Bridges, paragraphs 1.2.5, 1.2.12 (C) and 1.3.2 (H).

9. Grover, LaMotte. Manual of Design for ARC Welded Steel Structures, Air Reduction, New York 17, N. Y.

10. A.C.I., Committee 317, Reinforced Concrete Design Handbook, Detroit, Michigan.

APPENDIX "A"

DESIGN PROCEDURE FOR SEALERS

The design procedure described here accomplishes, basically, two purposes. It establishes the size of sealer to be used in a joint, and it determines at what width the joint must be constructed in order to insure the effectiveness of the sealer. To utilize these procedures one must set forth ahead of time the capabilities of the sealer in terms of three parameters - "X_{max}", "Y_{avg}", and "Z_{min}". Each of the parameters is the ratio of the sealers width of a certain level of compression to its original preformed width "W_n", multiplied by 100. "Z_{min}" is the value of the ratio at the maximum permitted compression of the sealer. "Y_{avg}" is the desired value of the ratio at the time of sealer installation. "X_{max}" is the value of the ratio at the minimum permitted compression of the sealer (enough compression to prevent leakage between sealer and joint face). These ratios or limits are adapted from the DuPont publication "Size Selection of Neoprene Compression Joint Seals."

It is necessary to mention that for now the limits "X_{max}", "Y_{avg}", and "Z_{min}" are empirical values based on experience. For the type of sealers presently available it would appear that "X_{max}" can be no more then 80%, "Z_{min}" should be 40% to 50%, and therefore "Y_{avg}" should be approximately 60%.

The design essentially consists of establishing from Figure A1 the maximum expansion and contraction movements to be experienced at the joint for expected differences between installation deck temperature and subsequent deck temperatures. Using these movements and applying the "X_{max}", "Y_{avg}", and "Z_{min}" values to an estimated sealer size, the construction width of the joint is then determined through a trial and error process.

To illustrate the application of charts Fig. A1 and A2, and to show hereafter the method used in sizing sealers and joints, a solution for a bridge with a span L - 60 ft. follows.

For the State of New Jersey a concrete temperature range of 0° to 100°F is assumed as being realistic. The wide range of sealer installation and construction temperatures of 30°F to 90°F is selected with required limits on efficiency coefficients taken as:

Z_{min} = ±40% at minimum width of joint ($W_{j_{min}}$) and 100°F.

Y_{avg} = ±60% at installation width of joint ($W_{j_{inst}}$) and installation temperature from 30°F to 90°F.

X_{max} = ±70% at maximum width of joint ($W_{j_{max}}$) and 0°F. *

Step 1. For utilizing chart, Fig. A1, we consider first that the maximum differences between the extreme temperatures of concrete and of installation are Δt = 70°F (100" - 30°) and Δt = 90°F(90°-0°).On this basis we can read off the chart that Δ = 0.28 in. for expansion and 0.36 in. for contraction.

Step. 2. (See Tables A3 and A4) by estimating the sealer size W_n = 2.5 in. and utilizing the limits $Z = 0.4\ W_n$ and $X = 0.7\ W_n$ we find from chart Fig. A2 that: $W_{j_{min}}$ = 1.0 in., and $W_{j_{max}}$ = 1.75 in.

$$\frac{W_{j_{max}} - W_{j_{min}}}{2} = \frac{0.75}{2} = 0.375 \text{ in} > 0.36 \text{ in. (max. joint movement).}$$

Thus, the joint construction width should be:

$W_{j_{constr.}}$ = 1-3/8" with $Y_{instal.} = 0.56 W_n$

The width of joint ($W_{j_{constr.}}$) is measured at the upper portion of the joint where the sealer is located.

(Approximate place for Figs. A1 and A2 and Tables A3 and A4)

* Note: An "X_{max}" of 70% is advisable in case of formed joint construction.

APPENDIX "B"

CONSTRUCTION PROCEDURE FOR ARMORED JOINTS

The concept of this method is that the whole system (armor plates with straps and seats welded to them, sealer properly precompressed between the plates and the supporting elements, such as clamps and attached bolts) is preassembled 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 the one hand, and on the other, it must give the fullest possible consideration to the de facto construction practices.

On this basis, the best approach would be to have the elements of the system preassembled to the fullest practicable degree, delivered to the construction site and there the assembly completed. The deck should be poured so as to leave the necessary recess with deck reinforcement properly extended into it. After the concrete is set, the assembly can be placed into the recess, properly located, and the width of the joint between the armor plates adjusted in accordance with the design requirements; then the bar-straps should be welded to the main deck reinforcement. The recess should be filled to the level "A" with optimum-packed-up concrete of good quality. After the concrete in the recess is set, the supporting elements should be removed and the surfacing of the deck at the joint carefully completed.

This procedure, with a little care in construction, should give a satisfactorily sealed joint.

The armored deck joints should be continuous throughout the full width of the deck, and termination should be accomplished as shown in Figs. B2 through B5. 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 B1 through B5, but a stay-in-place anchor seat could be added in the curb end at the bottom outside face of the armor shapes.

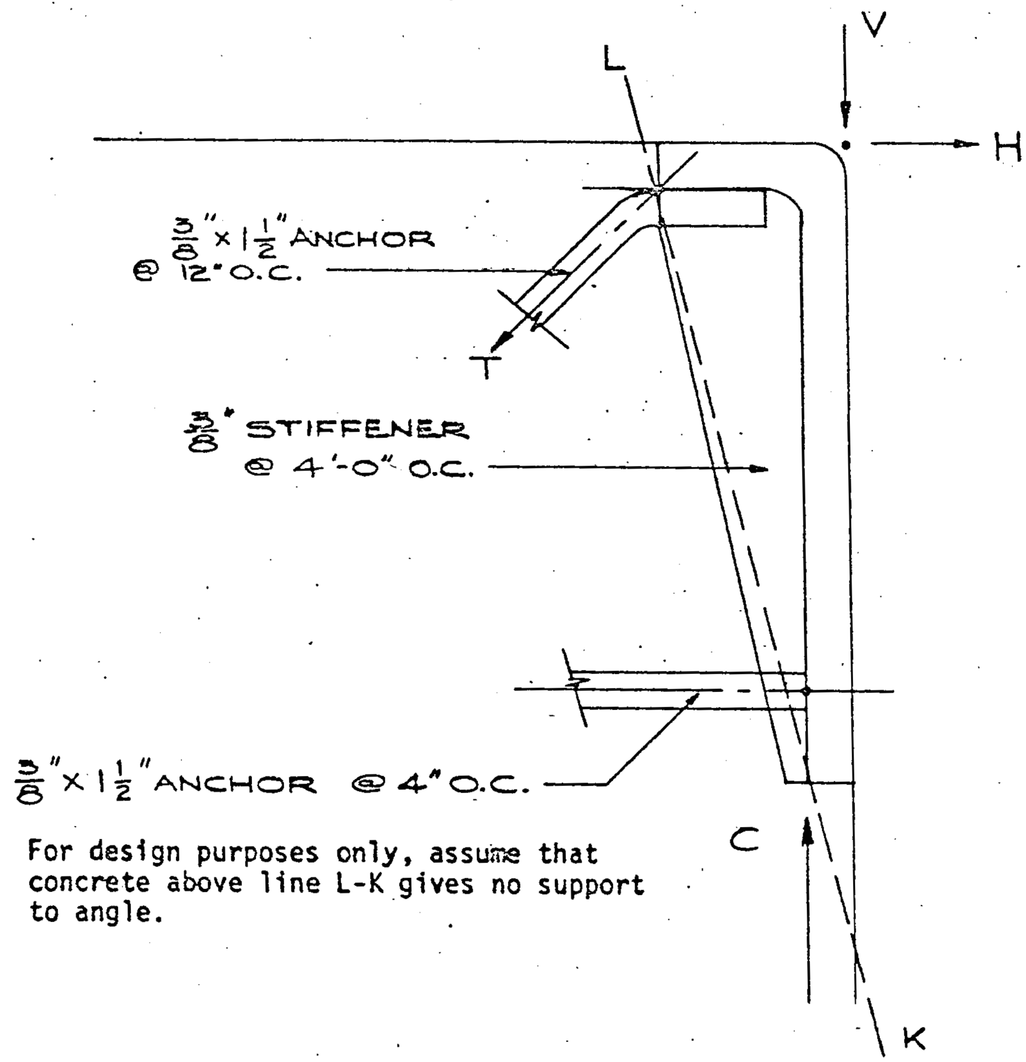
All steel of the armor network should be painted. In addition it is recommended that the armor be of ASTM A-242 steel. The stable rust characteristics of this material will serve advantageously in those areas where paint is likely to deteriorate rapidly with traffic.

Standard lubricant-adhesive shall be applied on both sides of the sealer when located in the armor.

(Approximate place for Figures B1 through B5)

APPENDIX "C"

ARMORED JOINT DESIGN



For design purposes only, assume that concrete above line L-K gives no support to angle.

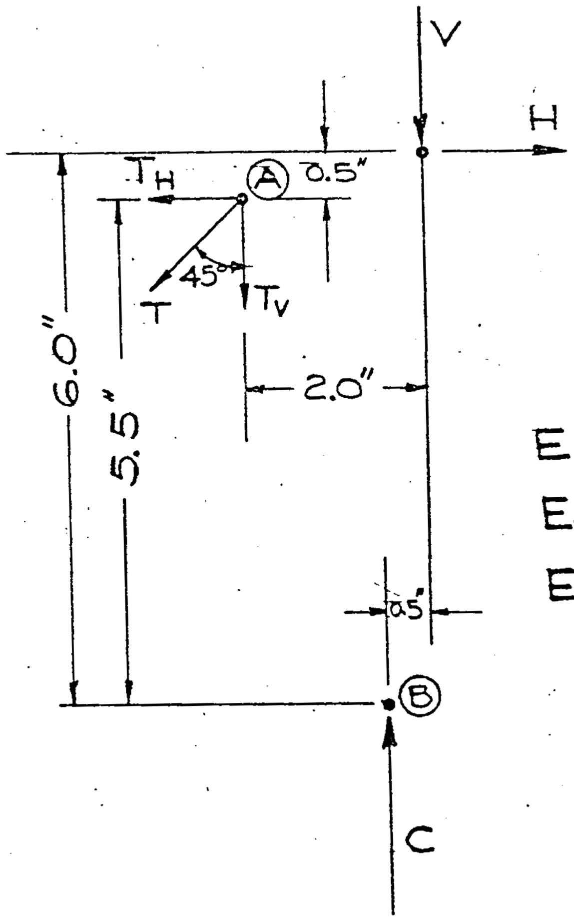
LOADS

For reference please see AASHO, 1969 (8)

paragraph 1.2.5 (HS 20-44):

Concentrated Loads (for shear):	26.0 kips
Wheel load (for horizontal shear):	16.0 kips

paragraph 1.2.12(C)	Impact fraction:	30%
	Friction factor for horizontal load:	0.75
paragraph 1.3.2(H)	Cantilever Slabs:	
	Moment per foot of slab	$= \frac{P}{E}X$;
	Case B: E	$= 0.35\ X + 3.2$;



$V = 26.0 + 0.3 \times 26.0 = 33.8\ k/E$

$H = 16.0 \times 0.75 = 12.0\ k/E$

LOAD DISTRIBUTION

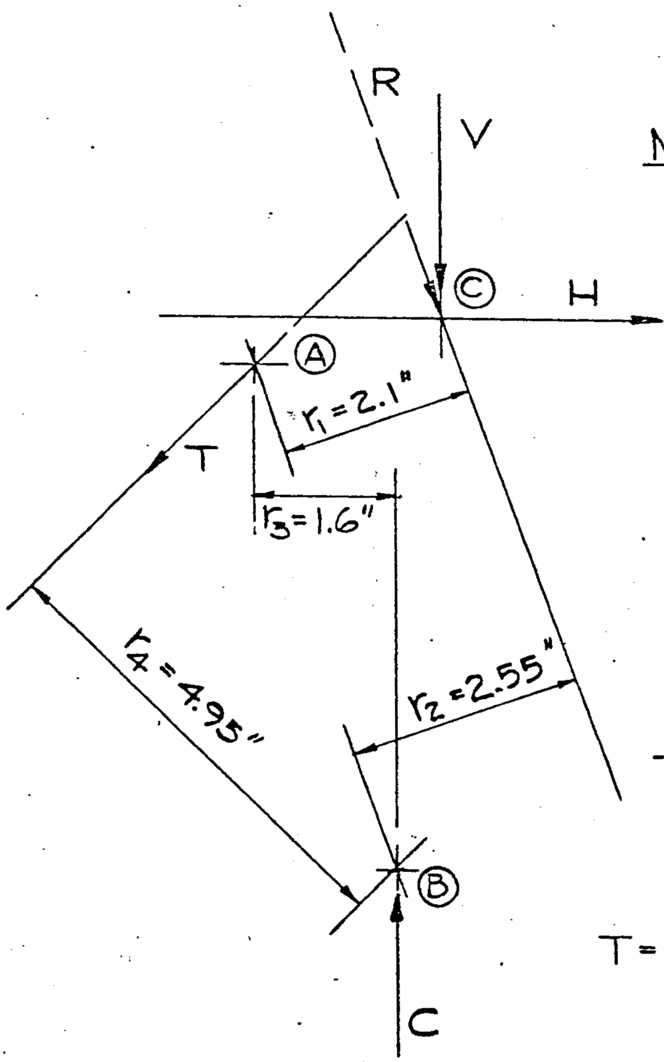
$E_1 = 0.35 \times 0.5 + 3.2 = 3.375$ FT.

$E_2 = 3.2$ FT.

$E_3 = 0.35 \times 0.17 + 3.2 = 3.26$ FT.

In absence of definite guidelines I am exercising my judgment in making reasonably severe assumptions. This is what W. Koester said about it in his previously referred book:

"The severity of the forces acting on the edges of the joint increases with the gap width. This necessitates the provision of a steel edge-protection strip which must be so rigid and so closely anchored that it forms an indissoluble composite structure with the bridge deck. The prerequisite for this is that the steel components should be securely joined to the concrete at all contact surfaces."



$$R = \sqrt{V^2 + H^2}$$

MOMENT ABOUT SUPPORT "A":

$$R \times r_1 - C \times r_3 = 0$$

$$C = \frac{r_1}{r_3} \times R = \frac{r_1}{r_3} \times \sqrt{V^2 + H^2} \quad ;$$

$$C = \frac{2.1}{1.6} \sqrt{\left(\frac{33.8}{3.375}\right)^2 + \left(\frac{12.0}{3.20}\right)^2} \quad ;$$

$$C = 14.03 \text{ K/FT.}$$

MOMENT ABOUT SUPPORT "B"

$$R \times r_2 - T \times r_4 = 0$$

$$T = \frac{r_2}{r_4} R = \frac{r_2}{r_4} \sqrt{V^2 + H^2} \quad ;$$

$$T = \frac{2.55}{4.95} \sqrt{\left(\frac{33.8}{3.2}\right)^2 + \left(\frac{12.0}{3.26}\right)^2} \quad ;$$

$$T = 5.77 \text{ K/FT.}$$

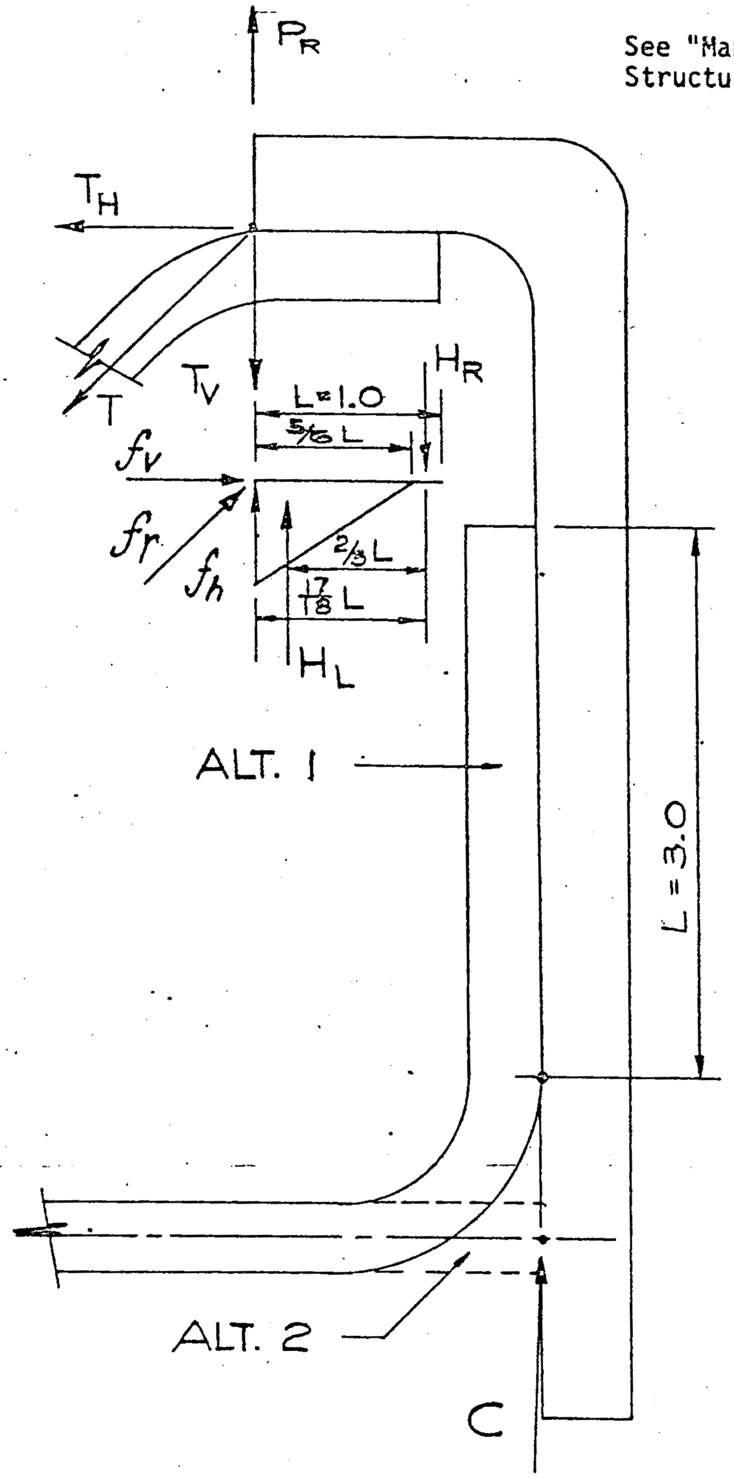
IF: $T = \sqrt{T_V^2 + T_H^2}$

AND $T_V = T_H$ (@ 45°)

THAN: $T_V = T_H = 0.707\,T = 4.08$ K/FT.

WELDING STRESSES

See "Manual of Design for ARC Welded Steel Structures" Air Reduction, N.Y.(9)



$$H_L = H_R = H = \frac{5}{12} f_h L \ ;$$

$$P_r = 0.7707 f_{ALL} D L_1$$

$$(T_V - P_r) \times \frac{17}{18} L = H_L \times \frac{2}{3} L$$

THEREFORE :

$$f_h = \frac{17}{5L}(T_V - 0.707 f_{ALL} D L_1) \ ;$$

$$f_V = \frac{T_H}{2L + L_1} \ ;$$

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

FOR ANCHORS @12"O.C.:

$T_V = 4.08_K$

$L_1 = 1.5"$ (WIDTH OF STRAP)

$L = 1.0"$

$D = \frac{5}{16}"$ (SIZE OF WELD)

$f_r = 0.707 f_{ALL}.D \ ;$

$$f_h = \frac{17}{5 \times 1.0}(4.08 - 0.331 f) = 13.85 - 1.13 f$$

$$f_V = \frac{4.08}{3.5} = 1.17$$

$$0.221 f = \sqrt{(13.85 - 1.13 f)^2 + 1.17^2}$$

[illegible]

ALTERNATE 1, ANCHORS @ 12" O.C.

$$0.707 \times f \times D \times 2L = C$$

$$f = \frac{C}{1.41 \times D \times L} = \frac{14.03}{0.442L} = \frac{31.8}{L};$$

FOR L = 3.0 in. EACH SIDE:

$$f = \frac{31.8}{3.0} = 10.6\ ^{K}/_{in^2} < f_{ALL}$$

BEARING

AVAILABLE L = 9.0 in.

ASSUMING TRIANGULAR BEARING DISTRIBUTION:

BEARING STRESS SHALL BE:

$$f_{BEAR.} = \frac{C}{A} = \frac{C}{1.5 \times \frac{L}{2}} = \frac{2C}{1.5L} = \frac{2 \times 14.03}{1.5L} = \frac{18.73}{L}$$

$$f_{BEAR.} = \frac{18.73}{9.0} = 2.08\ ^{K}/_{in^2} > f_{ALL}.$$

ALTERNATE 2, "n" ANCHORS (WELDING STRESSES)

$$0.707 f \times D \times 2(a+b) \times n = C$$

$$f = \frac{C}{0.707 \times D \times 3.75 \times n} = \frac{14.03}{0.83 \times n} = \frac{16.95}{n};$$

FOR n = 2 OR @ 6 in. O.C.:

$$f = \frac{16.95}{2} = 8.5\ ^{K}/_{in^2} < f_{ALL}$$

SHEAR STRESSES:

$$f_{SH} = \frac{C}{A \times n} = \frac{14.03}{0.375 \times 1.5 \times n} = \frac{25.0}{n};$$

FOR n = 2:

$$f_{SH} = \frac{25.0}{2} = 12.5\ ^{K}/_{in^2} < f_{ALL}.$$

BEARING (ALTERNATE 2)

AVAILABLE L = 10.5 in.

$$f_{BEAR} = \frac{C}{1.5 \times \frac{L}{2} \times n} = \frac{2 \times C}{1.5 \times L \times n} = \frac{2 \times 14.03}{1.5 \times 10.5 \times n} = \frac{1.78}{n};$$

FOR n = 3 OR @ 4 in O.C. :

$$f_{BEAR} = \frac{1.78}{3} = 0.593\ ^{K}/_{in^2} < f_{ALL}.$$

USE: BOTTOM ANCHORS @ 4 in. O.C.

TENSION STRESSES IN TOP ANCHORS:

$$f_T = \frac{T}{A} = \frac{5.77}{0.375 \times 1.5} = 10.28\ ^{K}/_{in^2} < f_{ALL}.$$

BOND STRESSES:

ASSUMING THAT HOOK SHALL DEVELOPE 50 % OF THE ALLOWABLE STRESS IN THE STRAP, THE BOND STRESS SHALL BE:

$$f_{BOND} = \frac{T}{2(a+b)L \times 2} = \frac{5.77}{7.5 \times L} = \frac{0.77}{L};$$

FOR L = 7″

$$f_{BOND} = \frac{0.77}{7.0} = 0.11\ ^{K}/_{in^2} < f_{ALL}$$

Headers

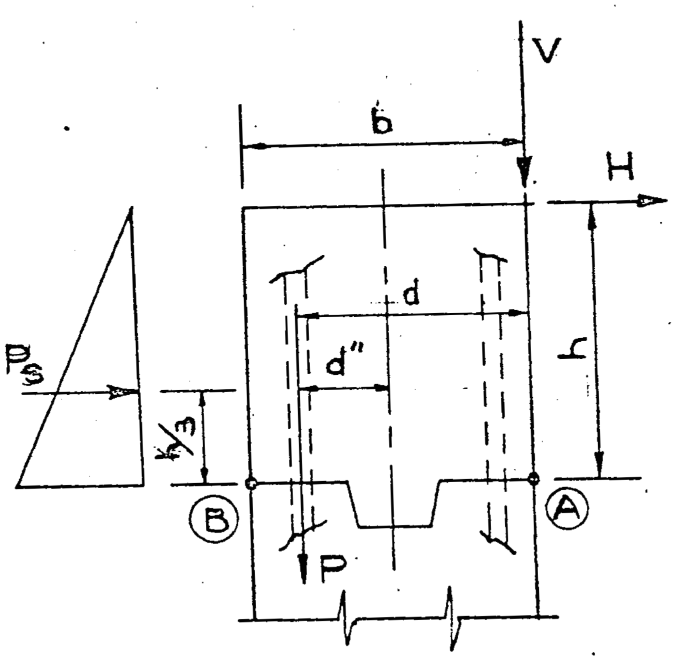
Before concluding this memorandum, I feel it is imperative that problems and design associated with headers should be at least aired here.

Failure of headers is not uncommon and has been personnaly observed. I believe that causes for their failure are as follows:

1. Loading, such as indicated in armor design.
2. Inadequate preparation of the backfill.
3. Concrete approach slabs directly supported by headers.

For the second problem, obviously I can suggest only one remedy - improvement of quality control in construction.

DESIGN (suggested approach):



Again as before:

Concentrated loads: 26.0 kips

Wheel loads for horizontal shear: 16.0 kips

Impact fraction: 30%

Friction factor for tire against concrete: 1.0

CANTILEVER SLABS :

MOMENT PER FOOT OF SLAB $= \frac{P}{E} X$;

WHERE $E = 0.35 X + 3.2$;

THUS : $V = 33.8 \; ^{K}/_{E}$

$H = 16.0 \times 1.0 = 16.0 \; ^{K}/_{E}$.

LOAD DISTRIBUTION :

$E = 0.35 \times 1.5 + 3.2 = 3.725$ FT.

MOMENT ABOUT PLANE "A-B" :

$$M = \frac{33.8}{3.725} \times \frac{b}{2} + \frac{16.0}{3.725} \times h + P_S \times \frac{h}{3} ;$$

TOTAL VERTICAL LOAD IS :

$$N = \frac{33.8}{3.725} + 0.15 \times 1.5^2 \times 1.0 ;$$

REINFORCEMENT DESIGN (10):

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

$$P = \frac{N(e - jd)}{jd} \quad (^{K}/_{\text{FT. WIDTH}})$$

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

A few words about approach slabs seem warranted before concluding this analysis. The problem of approach slabs is a very complex one, especially if a rigid slab is supported on one end elastically and on another end off a vertically rigid but horizontally flimsy support, such as a header would be.

In such a case the effect on a header would be vertically an eccentrically located static load and a distinctly possible substantial horizontal static force, and of course dynamic reactions in addition to those already discussed.

In this paper I shall not further discuss the problem of approach slabs nor suggest remedies since the prime purpose of this work is the sealing of joints. However, it must be said that obviously a joint which is not permitted to function as designed cannot be sealed. Even a perfect solution of the joint sealing problem shall be useless if a header failure, from whatever cause, disallows proper functioning of the joint.

In the experimental bridges approach slabs were removed, but also grossly inadequate preparation of the backfill had to be combated.

In conclusion it should be stated that obviously the design and construction of armor anchorage and headers is a very serious problem which to date apparently is being sometimes taken too lightly. The above offered suggestions should serve to air the related problems and perhaps help to alleviate them.

APPENDIX "D"

DYE TESTS ON BRIDGES #1 & #5 OF ROUTE #29

On March 16, 1970, dye tests were performed on the two experimental bridges of Route 29, designated as Bridges #1 and #5. The purpose of the tests is to study the sealing performance of bridge joint sealing techniques.

Specifically, various colors of dye are used to locate the origin and determine the cause of leaks in bridge joint sealers. In addition, the dyes may be traced by observation of their destination. In order to add to the completeness of these initial tests which were performed during a day of clear, dry, freezing weather, observation of the bridge joints and abutments was repeated on March 18 during light rain and sleet. Additional dye was also poured on March 18.

For the purpose of clarification, schematic diagrams of each bridge, Figs. C1 and C2, together with explanatory notes follow.

NOTES FOR BRIDGE #1
(FIG. C1)

1. Blue dye was poured on the safety walk sealer; no leaks were observed.

2. Pink dye was poured on the main sealer in the gutter area. The dye accumulated and ran along the gutter. No leaks were observed either down through the sealer or through the junction of the main sealer and the walkway sealer.

3. Blue dye was poured directly into a hole in the top surface of the sealer. The dye was not observed to have traveled to any other point. It is recommended that a large amount of dye be poured in this hole during the next dye tests in order to determine its final destination.

4. Yellow dye was poured on most of the length of the joint sealer. No leaks were observed. The dye simply flowed to the gutter of the bridge.

5. Blue dye was poured on the walkway sealer and on the main sealer in the gutter area; no leaks were observed.

6. Blue dye was poured on the walkway sealer and on the main sealer in the gutter area; no leaks were observed.

7. Maroon dye was poured on the main sealer in the gutter area; no leaks were observed. The dye did not run off the end of the main sealer, showing that the junction of the main sealer and the walkway sealer is, to date, sealed.

(Approximate place for Fig. C1)

NOTES FOR BRIDGE #5
(FIG. C2)

1. Blue dye was poured on the sealer in the gutter area. None of this dye was observed to have leaked through the joint; the excess dye flowed down the gutter.

2. Maroon dye was poured on the sealers in the gutter area and in the safety walk. This dye did not seep down through either the main joint sealer or the safety walk sealer. Excess dye flowed through the junction of the main sealer and the walkway sealer to the outer end of the main sealer and then down the abutment.

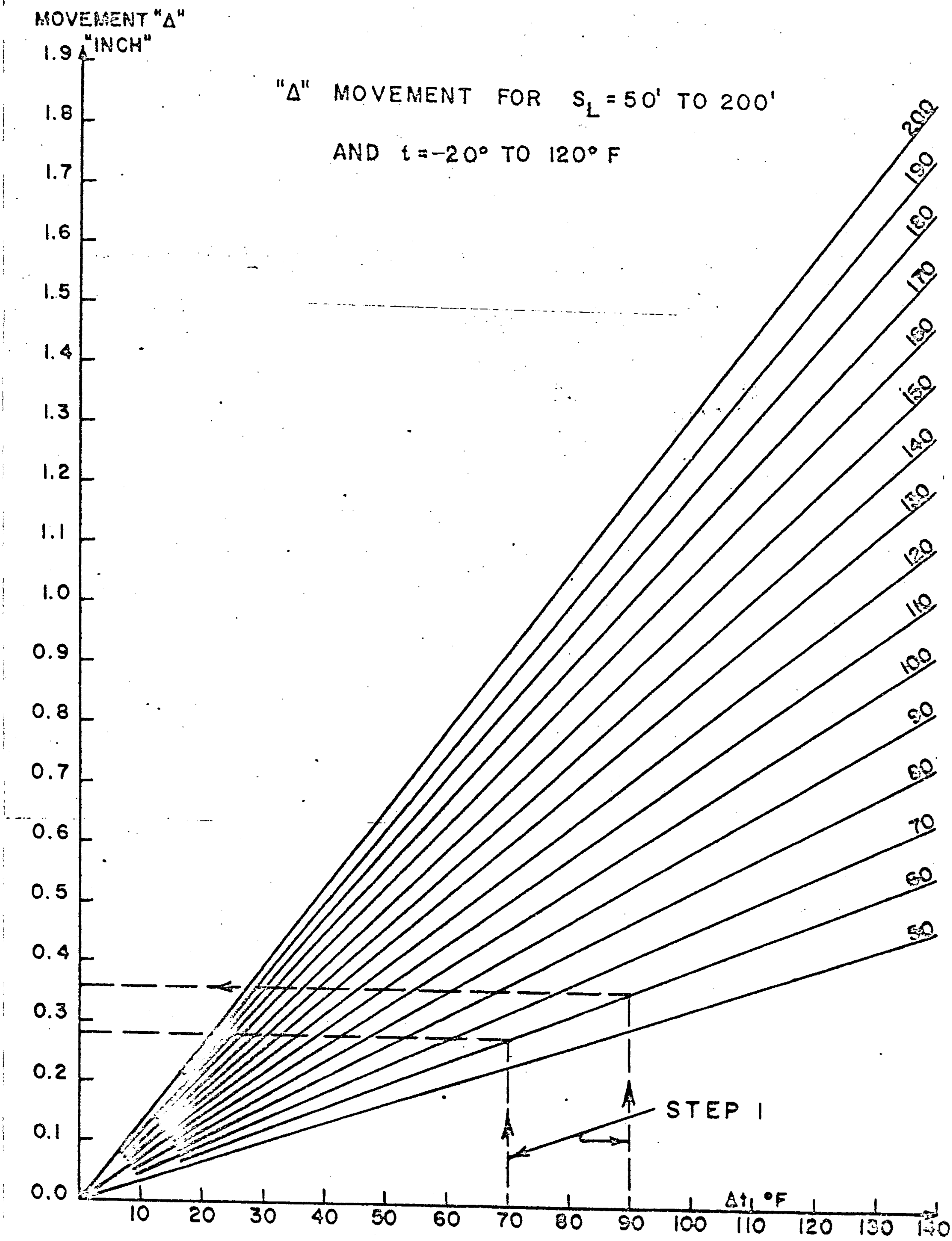
3. Maroon dye was poured on the main sealer in the gutter area and on the safety walk sealer. A small trace of dye leaked down through the walkway sealer; the remainder accumulated in the gutter.

4. Blue dye was poured where indicated on the sealer of the divider, on the walkway sealer, and on the main sealer in the gutter area. One leak was observed at the junction of the divider sealer and the walkway sealer.

5. Yellow dye was poured along the length of the sealer indicated. The dye was observed to flow to the hole at the junction of the divider sealers, through which it proceeded downward to the bridge substructures.

6. Red dye was poured where indicated on the sealer of the divider, on the Bridge #4 walkway sealer and on the hot poured main joint sealer of Bridge #4. Only one small leak was observed; it was not possible to accurately locate the leak on the topside of the bridge. It was determined that this leak does not contribute to, or interfere with the other leaks in the area (notes 4 and 5).

(Approximate place for Fig. C2)



MOVEMENT "Δ" "INCH"
"Δ" MOVEMENT FOR S_L = 50' TO 200'
AND t = -20° TO 120° F
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
STEP 1
Δt, °F
10
20
30
40
50
60
70
80
90
100
110
120
130
140

JOINT SEALER EFFICIENCY CHART

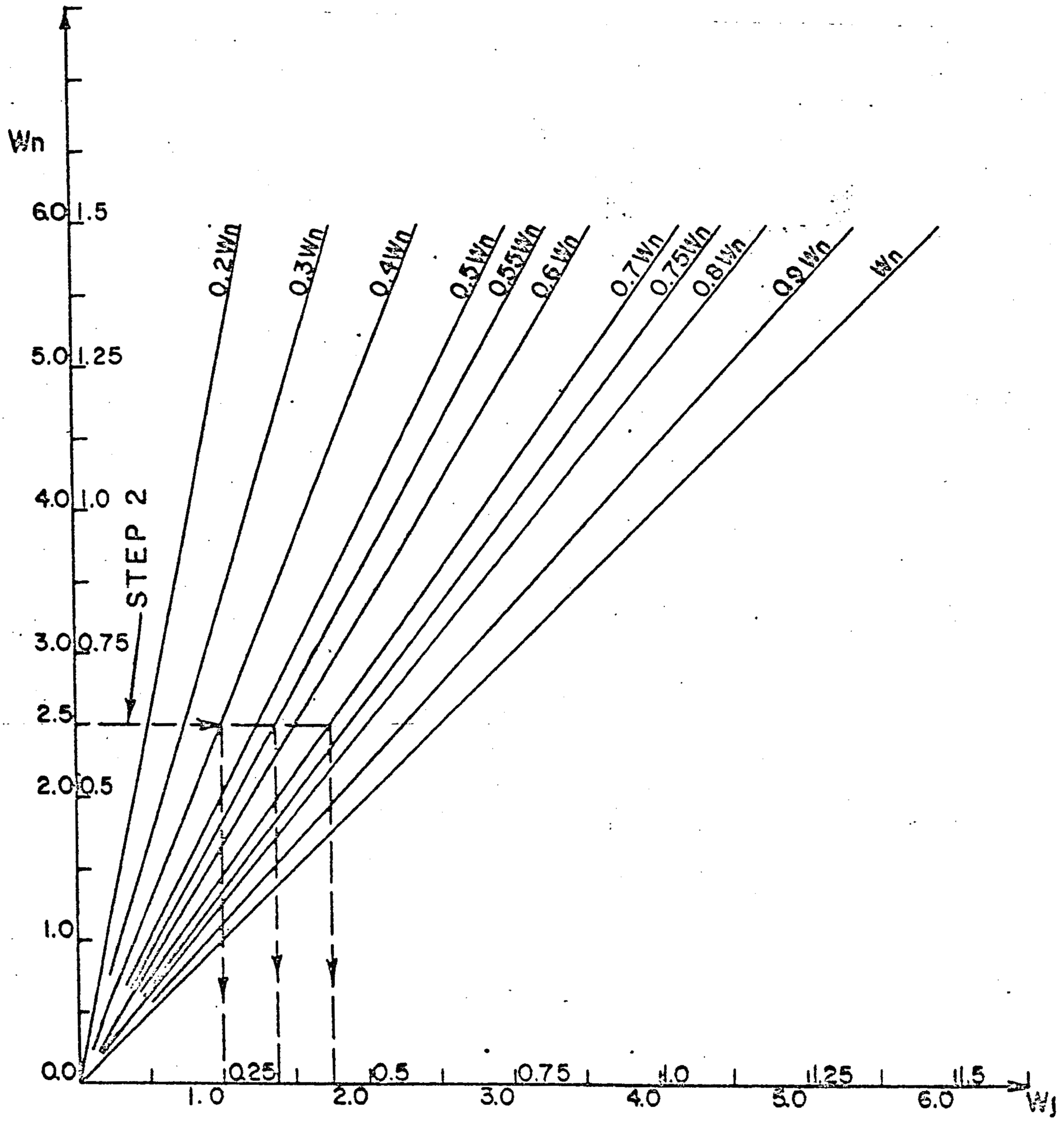


FIG. A2

Table A 3
Guide to the design of sealers

Temperature Range: 0° to 100°F
Construction Temperature: 30° to 90°F
Installation Temperature: 30° to 90°F
Degrees of Efficiency: $Z_{min} = \pm 0.50 W_n$
$Y_{average} = \pm 0.60 \text{ to } 0.65 W_n$
$X_{max} = \pm 0.80 W_n$

W_n	@100°F $W_{j\,min.}$	Z	Δ @ Δt=70°	@30° to 90°F W_j	Y	Δ @ Δt=90°	@ 0°F $W_{j\,max.}$	X	Limits of Span
1 1/2"	0.875	0.58	0.00	7/8"	0.58	0.00	0.875	0.58	Up to 40'
	0.695	0.46	0.18			0.24	1.115	0.74	
1 3/4"	0.945	0.54	0.18	1 1/8"	0.64	0.24	1.365	0.78	40' to 45'
	0.915	0.52	0.21			0.27	1.395	0.80	
2"	1.04	0.52	0.21	1 1/4"	0.625	0.27	1.52	0.76	45' to 55'
	1.00	0.50	0.25			0.33	1.58	0.79	
2 1/2"	1.25	0.50	0.25	1 1/2"	0.60	0.33	1.83	0.73	55' to 70'
	1.18	0.47	0.32			0.42	1.92	0.77	
3"	1.555	0.52	0.32	1 7/8"	0.625	0.42	2.295	0.765	70' to 90'
	1.455	0.485	0.42			0.53	2.405	0.80	
4"	2.08	0.52	0.42	2 1/2"	0.625	0.53	3.03	0.76	90' to 120'
	1.95	0.49	0.55			0.71	3.21	0.80	
5"*	2.575	0.515	0.55	3 1/8"	0.625	0.71	3.835	0.77	120' to 150'
	2.435	0.49	0.69			0.89	4.015	0.80	
6"*	3.06	0.51	0.69	3 3/4"	0.625	0.89	4.64	0.77	150' to 180'
	2.92	0.49	0.83			1.07	4.82	0.80	

NOTE: All the above temperatures are those of the concrete. Since these temperatures cannot readily be measured, the daily average temperature of the air with a tolerance of +5 to +10°F would be presently acceptable.

* Because of the lack of experience with these two largest sizes, it is preferable not to use the sealer widths W_n = 5" and 6" until further data are available.

TABLE A4

GUIDE TO THE DESIGN OF SEALERS

Temperature Range: 0°to 100°F
Construction Temperature: 30° to 90°F
Installation Temperature: 30° to 90°F
Degrees of Efficiency: Z_{min} = 0.40 W_n±
$Y_{average}$ = 0.60 W_n±
X_{max} = 0.80 W_n±

W_n	@100°F $W_{j_{min.}}$	Z	Δ@ Δt=70°	@30° to 90°F W_j	Y	Δ@ Δt=90°	@ 0°F $W_{j_{max.}}$	X	Limits of Span
1½"	0.875	0.58	0.0	7/8"	0.58	0.0	0.875	0.58	Up to
	0.621	0.41	0.254			0.327	1.202	0.80	55.0'
1¾"	0.746	0.43	0.254	1"	0.57	0.327	1.327	0.75	55.0' to
	0.700	0.40	0.300			0.386	1.386	0.79	65'
2"	0.825	0.41	0.300	1-1/8"	0.56	0.386	1.511	0.76	65.0' to
	0.778	0.39	0.347			0.446	1.571	0.79	75'
2½"	1.153	0.46	0.347	1½"	0.60	0.446	1.946	0.78	75' to
	1.084	0.43	0.416			0.535	2.035	0.81	90'
3"	1.334	0.44	0.416	1-3/4"	0.58	0.535	2.285	0.76	90' to
	1.242	0.41	0.508			0.653	2.403	0.80	110'
4"	1.867	0.47	0.508	2-3/8'"	0.59	0.653	3.028	0.75	110' to
	1.682	0.42	0.693			0.891	3.266	0.81	150'
5"*	2.182	0.44	0.693	2-7/8"	0.57	0.891	3.766	0.75	150' to
	1.951	0.39	0.924			1.188	4.063	0.81	200'
6"*	2.576	0.42	0.924	3½"	0.58	1.188	4.688	0.78	200' to
	2.484	0.41	1.016			1.307	4.807	0.80	220'

NOTE: All the above temperatures are those of the concrete. Since these temperatures cannot readily be measured, the daily average temperature of the air with a tolerance of +5 to +10°F would be presently acceptable.

* Because of the lack of experience with these two largest sizes, it is preferable not to use the sealer widths W_n = 5" and 6" until further data are available.

Schematic Diagram of Bridge #1, Rt. 29

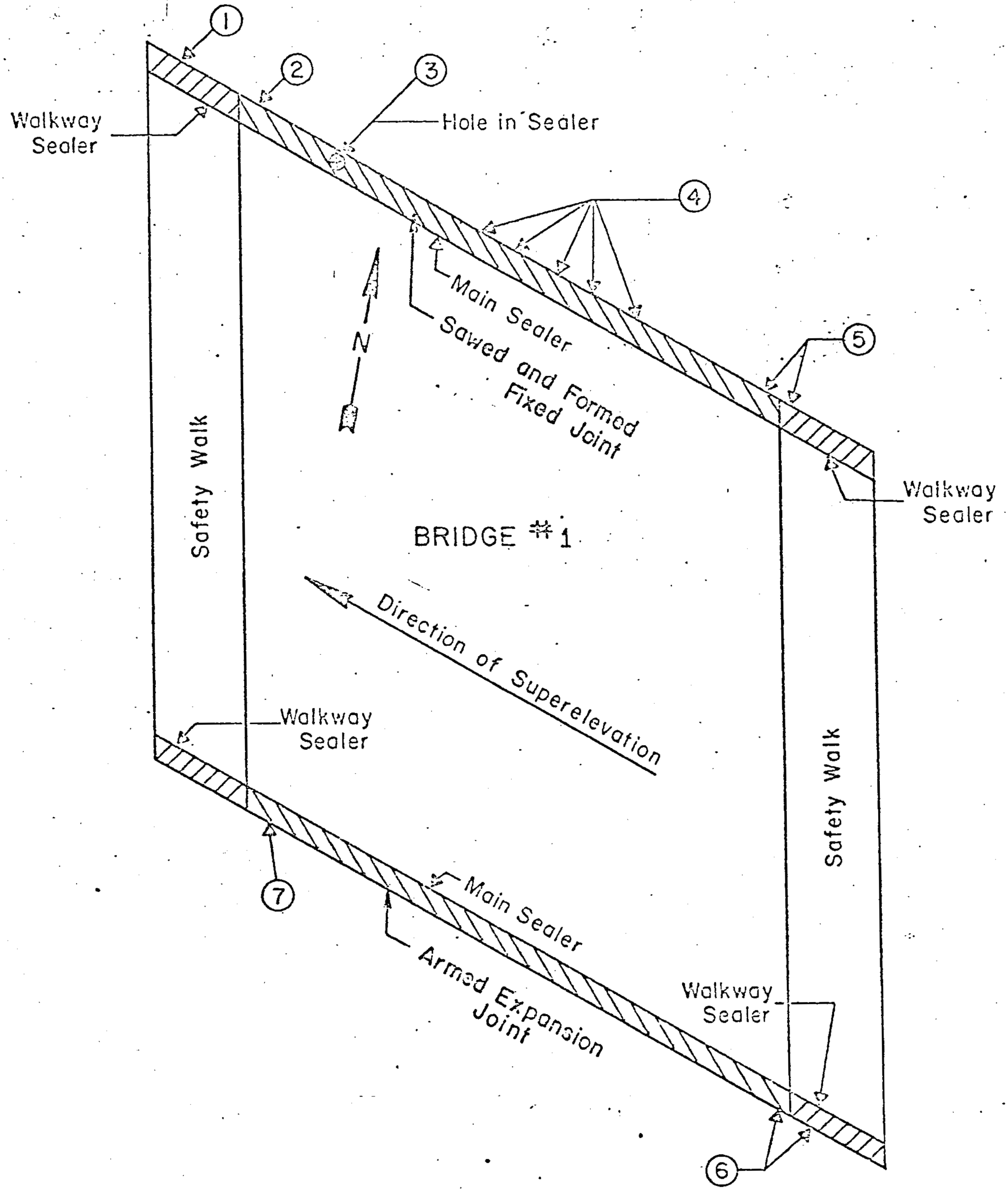


FIG. C1

Schematic Diagram of Bridge 5, Rt. 29

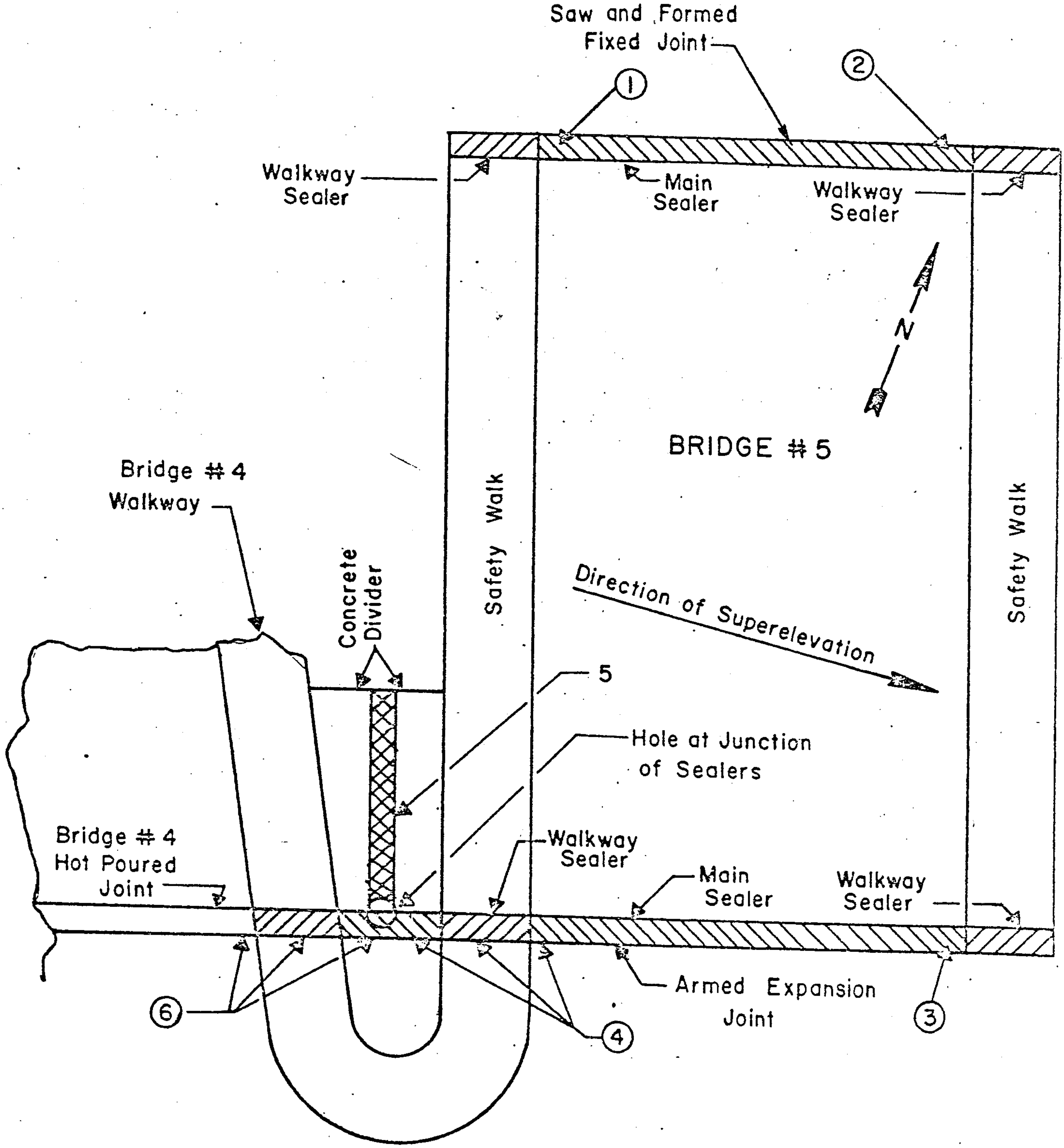


FIG. C2