HIGH-STRENGTH BOLTS AS A MEANS OF FABRICATING STEEL STRUCTURES

WILBUR M. WILSON, *Research Professor Structural Engineering, Emeritus, University of Illinois*

SYNOPSIS

Riveted joints connecting structural members are designed on the basis that the shear on the joint is resisted by the shearing strength of the rivets. Actually, especially for joints with long rivets, the rivets do not fill the holes, but, due to shrinkage when they cool, the rivets develop a tension equal, approximately, to the yield-point strength of the steel. This tension in the rivets produces pressure between the connected parts so great that it enables the joint to resist the shear to which it is subjected, by the friction between the connected parts.

If the tension in the rivets due to their cooling will enable a riveted joint to function by virtue of the friction between the connected parts, it would seem that the same results could be obtained if the rivets were replaced by bolts in which tension is induced by tightening the nuts. The resulting bolted joint would function by virtue of the friction between the parts that are clamped together by the bolts. Moreover, the carbon-steel rivets could be replaced by high-strength bolts, thus increasing the possible clamping force, and also increasing the resistance to shear of the bolted joint to a value considerably greater than the resistance to shear of the similar riveted joint.

Laboratory tests at the University of Illinois show that, by using high-strength bolts, it is possible to fabricate bolted structural joints that are actually much superior to similar riveted joints, particularly for situations where the joint is subjected to repeated or reversed loads.

Field observations under service conditions show that, under the very severe service conditions common to railway bridges, joints fabricated with high-strength bolts last longer and require less maintenance than similar joints fabricated with hot-driven rivets.

Thus it would seem from, (1) rationalization, (2) laboratory tests, and (3) observations on field installations; that structural joints fabricated with high-strength bolts not only should be, but actually are, superior to similar joints fabricated with hot-driven rivets.

The facts should be noted, however, that in order to obtain the good results described from the use of bolted joints, it is necessary that: (1) the bolts be made of a steel having a minimum yield point of the order of 85,000 psi. and an ultimate strength of at least 105,000 psi.; (2) the nuts must be tightened so as to produce a tension at the root of the threads of the bolts equal, approximately, to the yield point of the steel; (3) hardened steel washers of ample thickness must be used under both the heads and the nuts; (4) suitable bearing conditions must be provided for the washers under both the head and the nut; and (5) there must be a small fillet under the heads of the bolts.

These conditions, however, are easily complied with, and the service records of railway bridges indicate that the bolted joints fabricated with high-strength bolts are superior to riveted joints.

Until recently, structural engineers did not consider bolted joints to be suitable for fabricating permanent steel structures. Today, the structural engineers of some railroads are using high-strength bolts to replace rivets in the fabricated joints of bridges for which the rivets had either failed or loosened in service. The writer knows of no other instance in which the status of an engineering process has changed so completely and in so short a time, from one in which the process was considered unsuitable to one in which it was considered to be superior to the process previously used.

It is believed that the basis for this change in the attitude of structural engineers relative to the acceptability of structural joints fabricated with high-strength bolts, is of interest to the members of the engineering profession,

especially those who are responsible for the design, fabrication, and maintenance of steel bridges.

MANNER IN WHICH RIVETED AND BOLTED STRUCTURAL JOINTS FUNCTION

Riveted joints resist the shear between their various parts by virtue of the friction between those parts due to the tension in the rivets. This tension is due to the shrinkage of the rivets when they cool, and is limited in magnitude to the yield-point strength of the rivet, approximately 28,000 × 0.7854, or 22,000 lb. for a 1-in. carbon-steel rivet.

In considering the manner in which joints fabricated with high-strength bolts function, it should be realized that the bolts being considered are made of high-strength steel having a yield point of the order of 85,000 p.s.i., and an ultimate strength of 105,000 p.s.i.

A bolted joint, like a riveted joint, resists the shear to which it is subjected by the friction between the connected parts, but, in the case of the bolted joints, this friction is due to the tension in the bolts. And the clamping action of a 1-in., high-strength bolt, corresponding to the yield-point stress on the section at the root of the thread, 0.55 sq. in. for a 1-in. bolt, is 0.55 × 85,000, or 46,750 lb. per bolt, a value somewhat more than twice as great as the corresponding value for a 1-in. carbon-steel rivet.

In view of the above computations which show that a 1-in. high-strength bolt has more than twice the clamping action of a 1-in. carbon-steel rivet, it is reasonable to expect a structural joint subjected to a repeated or a reversed shear to be more satisfactory in service if fabricated with high-strength bolts than if fabricated with carbon-steel rivets.

The foregoing discussion applies to structural joints for which the rivets or bolts have a fairly long grip. A reduction in the grip will reduce the clamping action of the rivets but need not affect the clamping action of bolts.

It would seem, therefore, that, purely as a result of rationalization, structural joints fabricated with high-strength bolts may be expected to withstand a greater number of reversed-load cycles than a similar joint fabricated with carbon-steel rivets.

To supplement the rationalization, fatigue tests[1] were made to determine whether or not structural joints fabricated with high-strength bolts will function in the manner described and, if so, to determine the relation between the tension in the bolts and the maximum shear in the cycle to which the joint can be subjected for a large number of cycles without loosening or breaking the bolts.

These tests are described briefly in the following section.

LABORATORY FATIGUE TESTS OF STRUCTURAL JOINTS FABRICATED WITH HIGH-STRENGTH BOLTS

Test of Specimen 1X:—Specimen 1X consisted of one 12 by 2-in. inside plate connected by means of four 1-in. high-strength bolts in a square pattern, to two 12 by 1-in. outside plates. The surfaces of the plates were in the as-rolled condition with the mill-scale on. The test was planned so that the shear on the joint would be resisted by the friction between the plates. This required that the tension in the bolts be made large relative to the shear on the joint.

After the nuts on the bolts of the joints had been tightened so as to produce the desired tension for all bolts, in this instance, 45,000 lb. per bolt, the specimen was bolted to the heads of the testing machine. The testing machine was then adjusted so as to produce a load-cycle on the specimen such that the shear on the joint would vary from 0 to 15,300 psi. of bolt section during a cycle. Actually, this shear was resisted by the friction between the plates due to the 45,000 lb. tension in each of the bolts.

The test was run continuously night and day for 15 days during which time the specimen was subjected to 2,829,100 cycles. The load on the specimen was checked twice daily, the necessary daily adjustments in the load being of the order of 2 or 3 percent in each instance. The slip between the plates during a cycle and the tension in the bolts were also read twice daily but no adjustment was made in the bolt tension. The decrease in the bolt tension, the average for the four bolts, was of the order of 3,000 lb. per bolt for the entire

[1] These tests were made at the University of Illinois for the Research Council on Riveted and Bolted Structural Joints.

time of the test, a value slightly less than 7 percent of the initial bolt tension.

The slip between the plates during a cycle, and also the accumulated slip during the life of the test, was less than 0.001 in. There was no indication that the specimen had been affected in any manner when the test was discontinued after 2,829,100 repetitions of the cycle in which the shear on the joint varied from a force corresponding to a shear on the bolts of 15,300 psi. to zero.

There was no indication that the joint of Specimen 1X had deteriorated in any manner because of this large number of repetitions of the load cycle.

Specimen 3X:—Specimen 3X was similar to Specimen 1X. The bolts were given an initial tension of 45,000 lb. per bolt, and the nuts were not turned and did not rotate during the test. At the beginning of the test, the throw of the eccentric was adjusted so as to subject the specimen to a cycle in which the axial load on the specimen varied from approximately 128,000 lb. tension to an equal compression. On the basis that the shear on the specimen is resisted by the shear in the bolts, the corresponding shear cycle for the bolts would be 20,450 psi. shear in one direction to an equal shear in the opposite direction. At the beginning of the test, this produced a slip between the plates, from maximum tension to maximum compression, of the order of 0.05 in. The load decreased somewhat as the test progressed until, at 1,500 cycles from the beginning of the test, the shear on the specimen varied from a value corresponding to a shear on the bolts of 15,500 psi. in one direction to an equal shear in the opposite direction. Beginning at about this time, and without any change in the adjustment of the machine, of the specimen, or of the bolts, the slip between the plates began to decrease and the load on the specimen began to increase until, after 11,500 cycles from the beginning of the test, the slip during a cycle, from maximum tension to maximum compression, was of the order of 0.001 in. to 0.002 in. By this time, the shear in the shear cycle, expressed in terms of shear on the bolts, had come to vary from 19,270 psi. in one direction to 21,200 psi. in the opposite direction. The machine was then adjusted so as to produce a cycle in which the shear varied from 20,250 psi. of the rivet section in one direction to 20,250 psi. in the opposite direction.

The test continued without further adjustment of the machine or of the nuts on the bolts until, at 233,900 cycles, the average shear on the specimen varied from 19,360 psi. of bolt section in one direction to 21,050 psi. of bolt section in the opposite direction. The machine was then adjusted so as to produce a cycle in which the shear on the specimen varied from 20,250 psi. of bolt section in one direction to 20,650 psi. in the opposite direction. During all of this latter period, the slip from maximum tension to maximum compression in the specimen was of the order of 0.002 to 0.003 in.

The tension in the bolts, which at the beginning of the test was 45,000 lb. per bolt, decreased approximately 11,000 lb. per bolt during the first 100 cycles. This was during the time that the joint was slipping 0.05 in. while the load varied from maximum tension to maximum compression. At the end of 1,700 cycles, the slip during a cycle had diminished to 0.01 in., and the tension in the bolts had increased to a value only 7,400 lb. per bolt less than the value at the beginning of the test. From this time on, there was a very slow decrease in bolt tension until, at 26,100 cycles, it averaged 15,500 lb. per bolt less than at the beginning of the test. From this time on, the average stress per bolt did not change a significant amount and, at 1,585,000 cycles had an average value of 30,200 lb., 14,800 lb. per bolt less than at the beginning of the test. The slip, plus the strain in the plate, during a cycle continued to be of the order of 0.002 in. to 0.003 in. At the end of 1,585,000 cycles, the total accumulated slip was 0.027 in. Of this latter amount, 0.023 in. occurred during the first 87,900 cycles and 0.004 in. occurred during the last 1,497,100 cycles.

The tests that have just been described were of structural joints fabricated with high-strength bolts and the surfaces in contact of the connected plates were in the as-rolled condition with the mill-scale on. The tension in the bolts at the root of the threads was equal to the yield-point stress for the steel at the beginning of the tests. Of all the specimens tested, there was no evidence of impending failure in any of the bolts. Moreover, for specimens for which there was a slight slip between the plates at the beginning of the

tests, there was not a single instance in which the magnitude of the slip increased as the test continued beyond a few thousand cycles.

The fact should be realized, however, that the ability of a bolted joint to function when subjected to a reversal of stress depends on the friction between the plates. For this reason, in fabricating bolted joints that are to be subjected to repetitions or reversals of shear, the nuts should be turned up so as to produce an initial bolt tension at the root of the thread equal, approximately, to the yield point of the steel.[2]

BEARING FAILURE. PLATES CONNECTED WITH HIGH-STRENGTH BOLTS

No tests were made to determine the maximum allowable unit bearing of the bolts on the plates for bolted structural joints. And none would seem to be needed, for the following reason.

Bolted joints connecting plates are designed on the basis that the shear on the joint is resisted by the friction between the plates induced by the clamping action of the bolts. This being true, no appreciable pressure will be developed between the body of the bolts and the plates. Moreover, no indication of excessive bearing pressures nor of low fatigue values of the connected plates was noted during the tests such as would have developed if excessive bearing pressures had actually existed between the bolts and the plates.

SERVICE TESTS OF HIGH-STRENGTH BOLTS IN RAILWAY BRIDGES

The Committee on Iron and Steel Structures (Committee 15) of the American Railway Engineering Association, working through a sub-committee[3] entitled, The Committee on the Use of High-Strength Structural Bolts in Railway Bridges, supervised the installation of high-strength bolts to replace rivets in the connections of railway bridges for which the rivets had either become loose or failed in service. The following discussion of the behavior of high-strength bolted joints of steel railway bridges in service is based on the report[4] of this sub-committee.

In carrying out the work of the sub-committee, the research staff of the Association of American Railroads supervised the installation of over 1000 bolts of various sizes and lengths in about twenty different types of joints on twelve different bridges. The greater part of these installations were made in bridge joints where considerable trouble had been encountered in keeping rivets tight on account of the reversals of stress and vibration in the members.

The results of these field installations are of particular interest to the bridge engineer because, in the words of the sub-committee, "... the bolting of a joint is more economical than the riveting, especially in field construction, and more particularly in small or remote structures where the necessary riveting equipment is not readily available. Further, it obviously would be more economical to use high-strength bolts as erection or fitting up bolts and leave them in place during field erection of new structures rather than incur the expense of their removal and replacement with rivets."

The reader is referred to the work of the Committee on the Use of High-Strength Structural Bolts in Railway Bridges, for a detailed report of the large number of field installations in which high-strength bolts replace rivets that loosened or failed in service. However, the results of the tests as summarized by the A.A.R. Committee were as follows.[5]

[2] At first glance, this practice might seem to be objectionable, but, when it is realized that the external load does not act directly to produce tension in the bolt, and when it is further realized that a slight inelastic elongation of the bolt in service does not increase the bolt tension by an appreciable amount, it is apparent that the high initial bolt tension is not a serious potential weakness.

[3] The personnel of this sub-committee consisted of:

A. G. Rankin (Chairman), Frank Baron, J. E. Bernhardt, W. E. Dowling, R. B. Hennessey, N. E. Hueni, C. T. Looney, and H. C. Tammen.

[4] Bulletin 485, American Railway Engineering Association, pages 506 to 540, inclusive.

The test installations of high-strength bolts described in this report were conducted under the general direction of G. M. Magee, Research Engineer, Engineering Division, A.A.R. The bolts were installed and inspected under the direct supervision of H. H. West, research staff. The conduct of the tests and preparation of the report were in charge of E. J. Ruble, Structural Engineer, research staff, A.A.R.

[5] A.R.E.A. Bulletin 485, bottom of page 506.

The test installations of the high-strength bolts have only been in service for about one year but the following comments can be stated as a result of a recent inspection of all bolts:

1. The high-strength bolts installed with common washers in place of hardened washers in the ore docks at Ashtabula, Ohio, did not stay tight. The same bolts installed with hardened washers in the same holes have stayed tight.

2. The bolts installed in the lateral bracing connection of the Hulett ore unloader at Cleveland, Ohio, have remained tight but the new *rivets* in the connection at the other end of the same member have worked loose.

3. The bolts installed in the enlarged and jagged holes in the Pennsylvania bridge at Bellevue, Md., with *two* hardened washers under both the head and nut retained their full clamping force.

4. The bolts installed in the enlarged and dished out holes in the Pennsylvania bridge at Perryville, Md., with only *one* washer under the head and nut could not be kept tight as the washers deflected excessively under the high clamping force.

5. The bolts in the diaphragms of the Pennsylvania beam-span bridge at Naamen, Del., remained tight but some of the rivets at the opposite end of the diaphragms had worked loose during the year.

6. A visual inspection of all bolts did not reveal any evidence of cracked paint, rust or slippage of the members, except at those locations where the bolts had not been properly installed.

7. A check on the tightness of a representative number of bolts with the torque wrench revealed that the majority of the bolts have retained their original clamping force. Some of the bolts appeared to have lost some of their clamping force but it is not known whether the bolts were not properly tightened when installed or whether the bolted members are reseating themselves.

The Sub-Committee of Committe XV closes its report with the following conclusions.

The installation of high-strength bolts in structural joints of bridges carrying moving loads has afforded an opportunity to compare their behavior with the past behavior of rivets in the same holes and also with the behavior of new rivets in similar joints.

The behavior of high-strength bolts and hardened washers purchased in compliance with the previously stated specifications,[6] has been very satisfactory during a service period of over a year and it seems logical to make the following conclusions from a study of these test installations.

1. To avoid the breakage of the bolts subjected to high bending stresses it appears that the bolts should have fillets at the junction of the head and shank.

2. Bolts should not be installed in enlarged or irregular shaped holes without providing proper bearing surface for the hardened washers.

3. All bolts in a joint should first be tightened to the approximate torque and then each individual bolt again checked for the proper torque.

4. High-strength bolts properly installed stayed tight longer than rivets in similar joints subjected to the same vibrational loads.

5. High-strength bolts have a definite use in the maintenance work of railroad bridges.

CONCLUSIONS

The relative merits of high-strength bolts and carbon-steel rivets, as a means for fabri-

[6] The following requirements were specified in ordering the bolts, nuts and washers used in the tests supervised by the A.R.E.A. Sub-Committee entitled, The Committee on the Use of High-Strength Structural Bolts in Railway Bridges. (A.R.E.A. Bulletin 485, page 508).

1. The bolts shall be of American Standard Regular proportions, hexagonal in shape, and semi-finished to provide a good bearing surface under the head.

2. The nuts shall be semi-finished American Standard heavy and hexagonal in shape. The nuts shall have a minimum hardness of 121 Brinell (70 Rockwell B).

3. The bolts shall be quenched and tempered and shall have a minimum yield point of 85,000 psi. and a minimum tensile strength of 105,000 psi.

4. The threads shall be of the coarse-thread series as specified in the American Standard for Screw Threads having a Class 2 tolerance for both bolt and nut. The length of thread on the bolt shall be such that when the bolts are installed the face of the nut is not closer than three threads to the beginning of the shank.

5. The nominal diameter of the bolts shall be not less than $\frac{1}{16}$ in. smaller than the diameter of the hole.

6. The washers shall be of a heavy flat type about $\frac{11}{64}$ in. in thickness and shall be hardened by carburizing (not cyanide) to a minimum depth of 0.015 in. and to a hardness of 65–70 Rockwell A. The inside diameter of the washer shall be not more than $\frac{1}{16}$ in. larger than the nominal bolt diameter.

cating steel structures, have been studied by three methods, as follows:

1. Rationalization
2. Laboratory tests
3. Observations on structures in service.

The results of all three studies indicate that, for structural joints subjected to repeated or reversed loads, joints properly fabricated with high-strength bolts which resist the shear to which the joint is subjected by virtue of the friction between connected parts, are superior to similar joints fabricated with carbon-steel rivets.

In order to obtain superior results by the use of joints fabricated with high-strength bolts, it is necessary for the fabrication to comply fully with the prescribed practices recommended by the A.A.R. Committee, except as the recommended practice may be amended by future research and experience.

The replacement of rivets that have loosened in service, is both cheaper and better if the replacement is made with high-strength bolts than if it is made with rivets.

It is of interest to note that, in the A.A.R. installations, lock-washers were not used and in no instance did the bolts become loose due to the counter rotation of the nuts.

There have been some suggestions to the effect that high-strength bolts might be used in the field erection of building frames in congested areas, thereby eliminating the infuriating noise of the air-hammers. The results of the investigations described above would seem to fully justify this suggested change in practice, thereby eliminating a long-standing nuisance in the congested areas of our cities, as well as profiting by a cheaper and a better type of construction.

DISCUSSION

G. S. Paxson, *Oregon State Highway*, We have a job where several plates are stacked together and have been using alloy steel and alloy steel rivets. We have had some difficulty in getting tight rivets, which we think may be due to the multiplicity of plates in the grip. About 40 percent of the joints are critical and the grips are long, about 4½ in. Would it be safe to use high tensile bolts?

W. C. Stewart, *Industrial Fasteners Institute:* I think it would be perfectly safe to use high tensile bolts in this application. Possibly the difficulty in getting tight rivets is because the rivets were of alloy steel. There has always been some difficulty in driving alloy steel and getting a tight job. Rivet steels as they cool go through a recalescence state at a certain temperature. Down to this point the steel contracts as it cools, but at the recalescence point the steel expands rather suddenly without much change in temperature, and below this temperature further contraction takes place. It is usually considered that a rivet should continue to be driven to a temperature below this recalescence point. In carbon steel the recalescence point is around 1200 or 1300 deg., and below this temperature the steel is still malleable enough for easy driving. In the case of most alloy steels the recalescence point is lower and may be as low as 900 deg. The alloy steel, therefore, should be driven below this temperature and at that point is quite difficult to drive. I think this partly explains the difficulty in getting tight alloy rivets. A discussion of this action was published in Volume One, Number Two, of our magazine, FASTENERS, in an article by Professor George A. Maney of Northwestern University entitled, "Clamping Force—The Silent Partner in Riveted Joint Dependability."[7]

One of the things that has been surprising to many people is the fact that the high tensile bolts described by Professor Wilson maintain their tension under very rigorous circumstances such as occur in railway bridges, ore bridges, Hulett unloaders, etc. Throughout all the field tests and laboratory fatigue tests the bolts have maintained their tension quite well. I think the reason for this is that the loss in tension, which may be fairly small, and stabilizes rather quickly, is still a considerable proportion of the tension that can be set up in a low strength bolt; therefore, the low strength bolt might come loose due to the loss in tension. In the case of the high strength bolt, the amount of loss is the same, since the modulus of steel is the same, but the high

[7] FASTENERS, published by Industrial Fsateners Institute, 3648 Euclid Avenue Cleveland 15, Ohio.

strength bolt is capable of a very high clamping force—about three times that of the low strength bolt, so the loss in tension becomes a small proportion of the original clamping force.

Another point that seems to bother some engineers is whether there is any danger in bver-tightening the high tensile bolt. Of course, you may break the bolt by over-tightening, out you know it immediately and can replace it. Anything short of this failure—and this may be well beyond the yield point—may not cause any damage. We have some laboratory tests of this in fatigue where bolts were set up with a plastic elongation as much as ⅛ in., which, of course, is beyond the yield point. Specimens have been stressed in reversals of loading several million cycles without difficulty. Also, some bolts recently have been installed in field applications and stressed beyond the yield point to see what effects may occur.

E. J. Ruble, *Association of American Railroads:* The high strength bolt installations on Bridge No. Z-312, Chicago, Milwaukee, St. Paul and Pacific Railroad at Byron, Illinois were inspected by H. H. West, AAR and L. L. Darnell, Bridge Inspector, C.M.St.P. and P., October 30, 1950.

The stresses in approximately 20 percent of the bolts were checked by means of a torque wrench. Of this 20 percent, half of them had been checked and tightened with the torque wrench during the inspection in September 1949 and the other half had not been disturbed since their installation in November 1948.

The bolts that had been checked for tension and retightened previously had not lost any of their tension. The torque at which the nut started turning on the bolt was equal to or greater than the 470-ft. lb. torque used in tightening them during the last inspection.

The bolts which were checked this year that had not been disturbed during the 1949 inspection showed the same results as was found during that inspection. The nuts started turning on these bolts when a torque of from 360 to 420 ft. lb. was applied. This is approximately 18 percent below the 470 ft. lb. torque with which they should have been tightened originally. It is interesting that this is the same percentage of loss in tension that was found during the first year of service, and if the bolts are actually losing their tension, the total relaxation evidently takes place in a short time and does not continue indefinitely. These bolts were all tightened to 470 ft. lb. torque and their locations recorded so that they may be checked at future inspections.

Three bolts were found that turned at very low torque. One at 120 ft. lb. and the other two at about 240 ft. lb. Close examination showed that this was due to poor bearing surfaces for the washers. In two of these places the original rivets had been loose enough to move about in their holes and a dished out surface was formed by the heads. The other place was caused by deforming the angle when the rivet was backed out. The washers at these locations deflected and cracked causing the bolts to loose their tension. Double washers were placed on these bolts and they were retightened. One thing of interest is the fact that even with low tension that was in these bolts the nuts had not backed off any and there was no evidence that they were loose.

Several bolts in the top lateral system of spans 2 and 5 showed new rust around the bolt and at the edges of the faying surfaces. These bolts were checked with the torque wrench and some were found which turned at a torque of about 240 ft. lb. These bolts, being awkward to work on, were not checked with the torque wrench when installed and it is felt they were not tightened enough at that time. Part of the rust is due to the fact that the bolts had not been painted after their installation.

A visual inspection was made of the remaining bolts and nothing irregular could be found. None of the bolts showed any evidence of the nuts turning off or of rust that might indicate their being loose. During the inspection several loose rivets were found that had become loose since the bolts were installed. These were in locations similar to those in which the bolts were placed

Two facts are pointed out by this inspection; one is that workmen on inspections will have to check the condition of the bearing surfaces of the washers so that corrective action may be taken at the time of installation, and second, bolts in locations where it is hard to get good leverage must be checked with a torque wrench.

Professor Wilson: *Closure.* In my rational approach to the possibilities of a structural

joint fabricated with high-strength bolts being able to resist reversals of stress, I used as the value for the maximum stress to which the bolt could be subjected, the product of the yield point of the steel and the section of the bolt at the root of the thread. On this basis, the computations indicated that the joint should be able to function properly when subjected to a reversed-load cycle.

If the tension to which a bolt can be subjected is based on an area equal to the average of the area of the section at the root of the thread and the area of a section through the body of the bolt, the ability of the joint to resist reversals of stress would be correspondingly greater.

In other words, my statement in the paper is quite conservative.

STUDIES ON THE HEATING OF BRIDGE DECKS AND CONCRETE PAVEMENTS

G. S. Paxson, *Bridge Engineer, Oregon State Highway Commission*

SYNOPSIS

In this paper are described a pavement heating project using heat from a well driven into a subsurface strata carrying hot water and the preliminary experiments and design of a project involving the heating of a concrete bridge deck by electricity.

Klamath Falls Pavement Project—At Klamath Falls, Oregon, a 400-ft. section of four-lane roadway including two spans of a bridge was built on an eight percent grade. To minimize the traffic hazard from frost and snow, the roadway was heated. The project is underlaid by a strata carrying natural hot water. A 12-in. well, 425 ft. deep, was drilled to this strata supplying hot water which rises to the ground surface. The water temperature at the bottom was approximately 180 deg. F. The water contains mineral salts in solution, and to guard against deposits in the pipes a heat exchanger was placed in the well and an unmineralized solution circulated through the pavement slabs. The circulating system consists of 0.75-in. wrought-iron pipes at 18-in. centers placed at mid-depth of the slabs.

The system was designed to melt 0.5-in. of snow or .26 lb. of ice per sq. ft. per hr. This required 37.4 BTU per sq. ft. per hr. The design was based on a flow of 116 gal. per hr. per 30-ft. length of pavement slab, entering at 160 F. and leaving at 70 F. In actual operation the fluid entered at 130 deg. and left at 65 deg. The change in initial temperature was compensated for by increasing the flow to 158 gal. per hr. The actual BTU per hr. per sq. ft. varied from 43 to 47.

The installation operated through the winter of 1949–50, which was the most severe on record. With snow falling at the rate of one-half inch per hr., the pavement surface was moist but clear, with a surface temperature of 37 deg. F. and air temperature of 32 deg. F. A storm with a snowfall of 6 in. in 5 hr. left 0.5 in. of soft slush on the pavement, which was cleared in 30 min. At no time did any ice form. Tests indicate that after the pavement and subsoil reach equilibrium temperature little if any heat is lost into the subsoil. In fact the heat stored in the subsoil helps to maintain the pavement heat during periods when the system is working to capacity.

Willamette River Bridge at Salem—Limited distance in which to rise from street level and overcross a railroad forced the use of a seven percent grade on the approach to a bridge over the Willamette River at Salem, Oregon. Heavy snowfall is infrequent in the Willamette Valley, but high humidity results in frequent frost. As a safety measure the concrete deck of the approach will be heated by electric heat.

The bridge deck will be a reinforced concrete slab supported by concrete gird-