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EVALUATION OF IMPROVED TRACK STRUCTURAL COMPONENTS UNDER SUB-ARTIC CONDITIONS

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FINAL REPORT

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16. Abstract

One area of concern to railroads in the northern third of the United States is the effect of frost heave and subsidence on track geometry. The purpose of this study was to evaluate two methods of improving track geometry and reducing maintenance in sub-arctic environments.

Test track sections were installed in frost heave areas of the Alaska Railroad. One installation was designed to evaluate concrete ties with adjustable fasteners. The other installation was designed to evaluate the effect of including an elastic polymer stabilizer in the ballast.

The concrete tie adjustable fastener section performed satisfactorily. Ties withstood bending stresses induced by the unfavorable frost heaving support conditions. The fasteners provided an acceptable means of adjusting track during frost heave.

Stabilized ballast effectively reduced track subsidence due to weak foundation support. However, other problems such as hardening of the binding material and migration of the unstabilized ballast layer, made the particular system used unacceptable at this time.

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SUMMARY HIGHLIGHTS

One area of concern to railroads in the northern third of the United States is the effect of frost heave and subsidence on track geometry. The purpose of this study was to evaluate two methods of improving track geometry and reducing maintenance in sub-artic environments.

Test track sections were installed in subsidence or frost heave areas of the Alaska Railroad. Sections located in frost heave areas were designed to evaluate concrete ties with adjustable fasteners. The installation at Houston was designed to evaluate the effect of including an elastic polymer stabilizer in the ballast at a location where subsidence often occurs.

The concrete tie adjustable fastener section performed satisfactorily. Ties withstood bending stresses induced by the unfavorable frost heaving support conditions. The fasteners provided an acceptable means of adjusting track during frost heave.

Stabilized ballast effectively reduced track subsidence due to weak foundation support. However, other problems such as hardening of the binding material and migration of the unstabilized ballast layer, made the particular system used unacceptable at this time.

EVALUATION OF IMPROVED TRACK STRUCTURAL COMPONENTS UNDER SUB-ARCTIC CONDITIONS

INTRODUCTION

The problem of heaving and subsidence of track has long been of concern to railroads operating in the northern third of the United States. Although frost heave is normally limited to northern areas, subsidence can occur in any portion of a track where foundation support is weak. Thus, heaving and subsidence can be considered as two separate problems.

Heaving due to frost action is generally caused by the formation of ice lenses within the supporting structure. Quite often, the location of the lens with respect to the centerline of track may cause a considerable degree of disparity in cross-level of the rails. The net result is both vertical and lateral displacement of the rails.

Severe heaving due to frost action requires track adjustment to maintain normal railroad operations. Conventional methods of tamping and lining cannot be used if ties are frozen in the ballast. When such a situation arises in wood-tie track the rails are repositioned vertically by using wood shims, and horizontally by re-spiking fasteners to a new wood surface. When warmer weather comes and the frost thaws in the supporting foundation, the track returns to its original position necessitating removal of the shims and repositioning of the rails to their original alignment. Repeated use of this procedure leads to early wood tie deterioration, a phenomenon called "spike-killing".

Use of concrete ties in this application have merit only if fasteners having a considerable degree of vertical and lateral adjustment can be made compatible with the ties. In addition, the ties must be structurally adequate to survive significant rearrangement in the normal ballast-tie support heaving.

Prior to this project, adjustable rail fasteners had not been used in concrete tie track. Adjustable fasteners presently available were designed for direct rail-to-concrete slab or tunnel invert fixation under transit loading. These fasteners are generally capable of $\pm$ 1 in. (25 mm) lateral adjustment and a moderate amount of vertical adjustment by use of shims. The adjustability of these fasteners is intended primarily to correct deviations in line and surface caused by construction tolerances, and to extend the periods between rail transpositions necessitated by rail wear.

Ideally, the adjustability of a concrete tie fastening system should be sufficient to handle the maximum anticipated vertical deformations of 4 to 6 in. (102 to 152 mm) and lateral deformations of up to $\pm$ 2 in. (51 mm). A fastener incapable of accommodating such deformations will prohibit full track adjustment. This causes reduced train speeds and results in increased operating costs.

Additionally, a fastener used in track subject to foundation heaving will have to withstand greater than usual forces. These forces would occur in the interim between heaving and the arrival of the maintenance crew. Also, when a fastener has been adjusted vertically, say 4 in. (102 mm), by placing shims between the fastener and the tie, the hold-down elements and associated tie anchorages will be subject to greater than normal stresses. Thus, even with off-the-shelf fasteners, modification of some fastener components should be anticipated.

Finally, to perform adequately in track an adjustable fastener should be simple in construction. It must be adjustable with hand tools under adverse conditions of cold, snow, and ice. Once adjusted, it must maintain the adjustment.

Track subsidence is generally associated with weak foundation support. This weakness can be caused by the presence of soft unstable material beneath the embankment. Under the dead weight of the embankment, ballast, and track the foundation deforms laterally into areas of lower pressure. Superimposed loads and attendant vibrations from passing trains can accelerate this movement causing track subsidence.

Track subsidence also can be caused by weak unstable material within the embankment, especially if the live load is a large percentage of the total load carried by the unstable material. If the ballast layer is made cohesive by inserting an elastic polymer to bind the ballast particles, subsidence may be reduced. This may also produce a more favorable distribution of live load over the embankment thereby reducing unit pressures in the unstable material. Also, the cohesive ballast layer may tend to reduce traffic-induced vibrations transmitted to the unstable material.

To gain knowledge about track heave and track subsidence, and to evaluate the effects of modifications to track structural components under sub-arctic conditions, the Alaska Railroad initiated a limited research program. The program was conducted in cooperation with the Office of Research and Development of the Federal Railroad Administration.

The program was done by the Portland Cement Association, Construction Technology Laboratories, under Contract No. 69-25-0003-4144.

Objectives

The objectives of the program, that encompass three winter seasons, are:

1. Determine the feasibility of using concrete ties and adjustable rail fasteners in situations where rail profile and alignment are disturbed by track support displacement due to frost heaving;
2. Determine the adequacy of available concrete ties, produced in conformance with current design criteria, in situations where foundation heaving has altered the tie bearing support pattern substantially from that assumed in design;
3. Determine the response of a stabilized granular ballast layer and the treating medium to sub-freezing temperatures;
4. Determine the contribution of a stabilized granular ballast layer to improved dynamic track performance in the presence of weak foundation support.

To implement these objectives a two-part program was developed. Part 1 dealt with Objectives 1 and 2 and required the design and construction of a concrete tie test track in a known frost heaving area. Part 2 dealt with Objectives 3 and 4 and required the design and construction of a wood tie test track supported on a stabilized granular ballast layer.

Selection of Test Sites

The Alaska Railroad trackage between Anchorage and Fairbanks consists of a single track mainline carrying traffic in both directions. Numerous sidings are located along the mainline track. Because the test installations would require removal of existing track, suitable test locations opposite a siding were used to prevent traffic disruption.

Alaska Railroad records indicated a track subsidence area opposite the Houston siding at Mile Post (MP) 175, and several areas of severe frost heave opposite the Montana siding at MP 209. These locations, shown on Map 1, were inspected and determined suitable for the test installations. Both locations are on tangent track with 115-lb RE rail jointed in the conventional manner. Access to the test sites is by means of gravel side roads from the all-weather Anchorage to Fairbanks highway that roughly parallels the railroad.

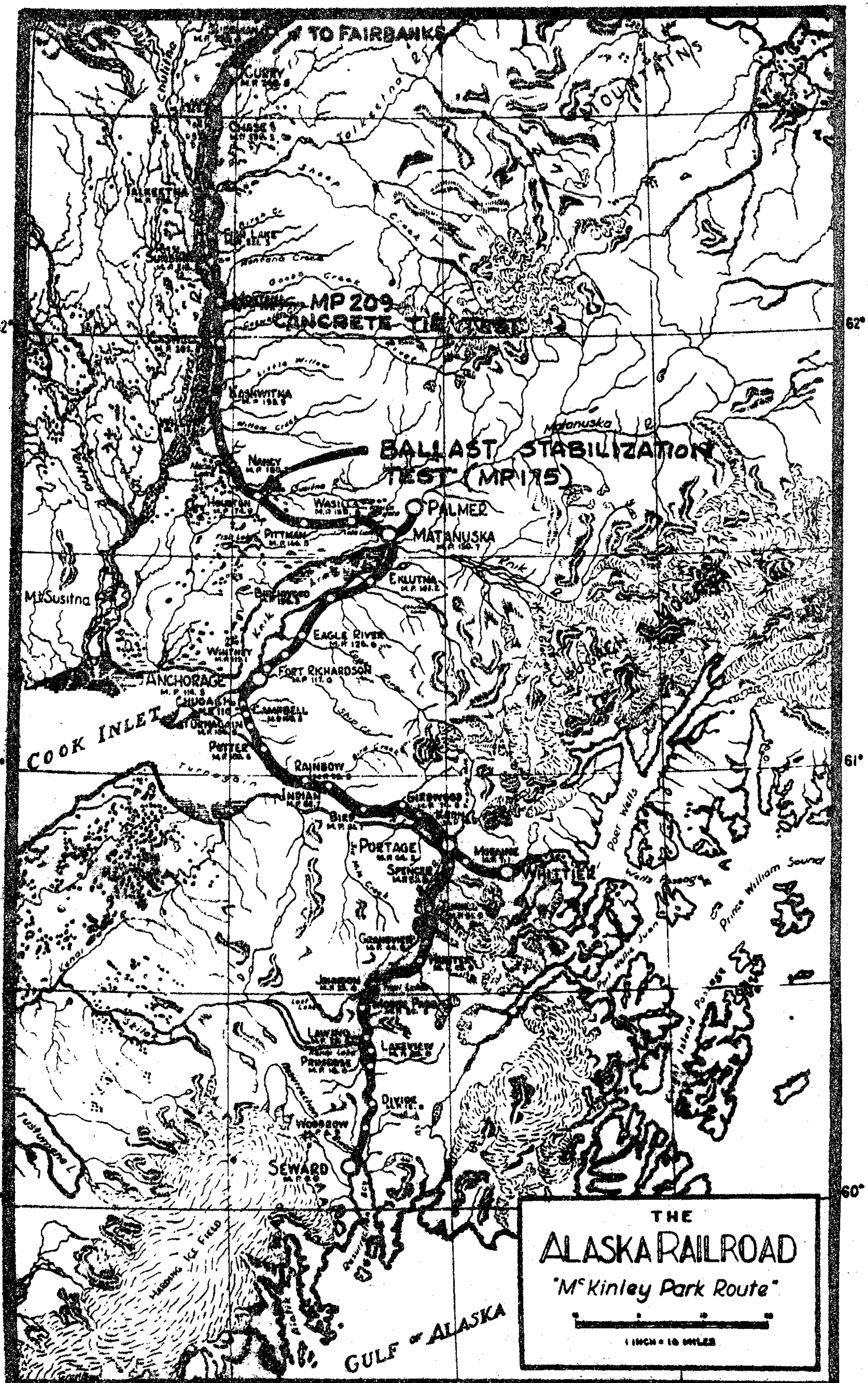
Average gross tons on the line was 2.21 million during 1971-76 and 2.83 million during 1974-76. The normal maximum wheel load was 32,500 lb. with approximately 10% occurrence.

MONTANA TEST SITE

Research Approach

To accomplish Objective 1, a concrete tie test section using ties with adjustable rail fasteners was designed and installed. The adjustment range for the fasteners was established at 6 in. (152 mm) vertically and ± 1-3/4 in. (44 mm) laterally per rail. Railroad personnel indicated that up to 3 in. (76 mm) of heaving could occur at the Montana test site. Thus, conventional surveying methods were used to provide verification of track performance.

Top of rail profiles were taken once in the fall, twice in winter, and once during early summer after frost left the



MAP 1 - LOCATION OF TEST SITES

ground. These profiles provided a record of the heaving that occurred in the test section during each winter. In addition, the degree of surface correction achieved by track adjustment (shimming) was recorded.

To compare the relative performance of wood and concrete tie track, an adjacent section of wood tie track, also subjected to frost heave, was designated as a control section. Thus, top of rail profiles, consisting of elevations taken at 10-ft (3.05-m) intervals along the test and control sections, were plotted and compared.

Measurements of the total vertical track displacement due to heaving were referred to a permanent bench mark near the track. Since frost penetration near the Montana test site can be over 10 ft (3.05 m), a permanent bench mark was installed. The bench mark was installed by driving a casing pipe over 20 ft (6.10 m) into the ground, placing a reference rod inside the pipe, and then driving the rod into the soil beneath the pipe.

An important consideration in determining the feasibility of concrete ties and adjustable fastenings was the relative ease or difficulty of making track adjustment as compared with wood ties. Thus, a record of man hours spent adjusting the test and control sections was kept.

Data for Objective 1 was gathered by Alaska Railroad personnel and analyzed by the Portland Cement Association.

To achieve Objective 2, the test section was inspected once each summer to determine the effect of the previous winter's exposure on the ties and fastenings. The ties were examined for flexural cracking caused by unfavorable support conditions associated with frost heave. Ties were examined also for signs of deterioration due to extremely low temperatures, -40 F (-40 C). Fastenings were examined for any distress due to service loads, corrosion, or any deficiency resulting from adjustments.

Two concrete ties were tested by the Portland Cement Association to determine strengths at critical cross sections (rail seat and tie center). All inspections and data analysis in connection with Objective 2 were conducted by the Construction Technology Laboratories staff.

Design

The Montana test site was inspected during the first week of July 1973. Three areas of major track adjustment were identified by the presence of shims used during the preceding winter.

After careful consideration, it was decided that the concrete-tie test section would be 10 rail-lengths long, approximately 390 ft (119 m). The section selected contained two 40 to 60 ft (12 to 18 m) areas of major track adjustment. These were centered within the first 4 rail lengths from each end of the test section.

The first 390 ft (119 m) of wood tie track immediately north of the test track was designated the control section. This section contained one 40 ft (12 m) length of major track adjustment.

Concrete ties were spaced at 26 in. (660 mm) on centers. This spacing was selected because of the 39-ft (11.9-m) rail-length between joints, and the need to center a tie directly beneath each joint. The 26-in. (660-mm) spacing was the only value that would position a tie directly under every joint. Thus, 181 concrete ties were required for the test section.

Limits of the test and control sections were staked prior to leaving the test site. Subsequently, Alaska Railroad personnel established a suitable bench mark and took top of rail profiles prior to installation of the concrete tie test section.

Selection and Evaluation of Track Components

Authorization to proceed with the project was given July 5, 1973. To avoid a one-year delay, it was necessary to install the concrete ties before freeze-up, which usually occurs in mid-October at the Montana site. This severe constraint necessitated several actions to conserve time. Thus, the following four work items were completed within a 10-week time span.

Establishment of Design and Performance Criteria

Because of unfavorable support conditions, the strongest concrete tie being manufactured in the United States at the time of installation was selected for the Montana test site. This was the prestressed Gerwick RT-7S tie being produced in limited quantities by Santa Fe-Pomeroy at Petaluma, California. A drawing of this tie, as modified to accept the special adjustable rail fastening system, is presented in Appendix A.

Adjustment needs of the rail fastening system exceeded the capability of any off-the-shelf fastener. Therefore, a new uncanted fastening system was designed. This fastening consists of a plate with two welded Pandrol-type shoulders for locating the rail on the plate. Heavy duty Pandrol spring clips were used for securing the rail to the plate. The plate has two slotted holes with serrations. Adjustment blocks, having matching serrations on the bottom surface and a 1-in. (25 mm) diameter coil bolt, were used for fastening the plate to the tie. The coil bolts are 10-in. (254-mm) long and thread into special coil-type anchors embedded in the tie. Cant was omitted for convenience of manufacture as it was not considered essential for a short duration test.

Vertical adjustment is made by loosening the coil bolts and inserting plywood shims between the plate and the tie. Lateral adjustment is made by loosening the coil bolts and sliding the plate laterally in either direction. When the plate is in the proper position, the coil bolts are tightened to secure the plate. The slotted holes allow $\pm$ 1-3/4 in. (44 mm) of lateral movement. Coil bolts and anchors are dimensioned to allow up to 6 in. (152 mm) of shims to be placed beneath the plate and still provide 2 in. (51 mm) of thread engagement.

In practice, fastener disassembly is not needed for adjustment. Because the clips secure the rail to the plate, all that has to be done is to loosen the coil bolts, jack the rails to the proper position, insert the shims between rails and plate, and retighten the bolts. Design torque for the coil bolts was 250-ft-lbs (339 N m). A drawing of the adjustable fastening system is shown in Appendix B.

Specifications for concrete ties and adjustable fastening system were prepared. These included dimensions and tolerances, basis for acceptance, and performance tests. Specifications are presented in Appendix C. Test procedures are similar to those specified in the AREA Preliminary Specifications for Concrete Ties (and Fastenings)(1)*. Loads proposed in AREA specifications for tie tests were used. However, loads proposed for fastening tests were changed, in some cases, in consideration of test track conditions. Principal change was a reduction of load in the "Lateral Load Restraint Test". This reduction was considered appropriate because of the low volume and speed of traffic and the straight alignment of test track.

Performance Tests of Adjustable Rail Fastenings

Performance testing required the fabrication of two prototype fastenings and special coil-type inserts to accept 1-in. (25-mm) diameter coil bolts.

The prototype fastenings were made at Petaluma, California and shipped to Portland Cement Association. Coil-type inserts were fabricated by Superior Products Co. of Chicago, Illinois. These were cast into a concrete test block. A right- and a left-hand fastening was secured to the test block with 2-1/4 in. (57 mm) of plywood shims. A 36-in. (914-mm) length of rail was secured to the fastenings using Pandrol heavy-duty clips. The tests were conducted by applying loads directly to the rail over one fastening, the second fastening being used only to prevent the rail from twisting in the fastening during the test. After completion of the tests, authorization was given to a machine shop in Petaluma, California to fabricate the fastenings for the Montana site.

Performance Tests of Concrete Ties

Production of concrete ties began September 11, 1973. Production could not begin earlier because the existing forms had to be modified for the new fastenings. Also, production of the special coil-type inserts could not begin until completion of tests on the prototype fastenings at the Portland Cement Association Laboratory. Production of concrete ties was completed September 25, 1973.

*Numbers in raised parentheses refer to references at end of report.

Performance tests were conducted by the Portland Cement Association at the Santa Fe-Pomeroy plant on September 25 and 26, 1973. The tie successfully met all of the specification requirements, except the Rail Seat Positive Moment requirement of 325,000 in.-lbs (36,720 N m), without cracking (see Appendix C - Specifications for Prestressed Concrete Ties and Adjustable Fastening System). The five ties selected for testing cracked in the range of 300,000 to 315,000 in.-lbs (33,895 to 35,590 N m). Previous tests on RT-7S ties with Pandrol shoulder had shown strengths considerably higher than the required bending moment of 325,000 in.-lbs (36,720 N m). Compression tests on cylinders showed concrete strengths somewhat lower than expected. This decrease in concrete strength and the resulting strength reduction in the ties was judged to be due to the use of the air-entrained concrete needed for the harsh Alaskan environment.

Five factors were considered in determining the acceptance of the ties:

a. Because special coil-type anchors embedded in the ties were non-retrievable, fabrication of new ones would cost additional funds and delay the project one year.
b. The 325,000 in.-lbs (36,720 N m) bending moment requirement was based on 27-in. (686-mm) tie spacing but 26-in. (660-mm) spacing would be used in the test track. The appropriate moment for 26-in. (660-mm) spacing is 316,750 in.-lbs (35,788 N m). This value is closer to the moment capacity of the tie.
c. The bending moment requirement for the 9-ft (2.74-m) long RT-7S tie is greater than that of an 8-ft, 6-in. (2.59-m) tie. The value for the latter at 26-in. (660-mm) spacing being 283,500 in.-lbs (32,031 N m). Thus, the RT-7S ties are stronger than the requirement for commercially produced 8-ft, 6-in. (2.59-m) ties.
d. The possibility of premature tie failure is remote in view of the light traffic density over the test section.
e. As a final consideration, a strength test was conducted on a tie to assure adequate bond performance.

A 100-kip (444,822-N) proof load was applied through a fastening and no failure occurred.

Based on the above considerations the ties were accepted. The ties and fastenings were placed on two Alaska Railroad flatcars on September 27, 1973 and shipped, by rail and seagoing ferry to Anchorage, Alaska.

Establishment of an Installation Plan

An installation plan was prepared and submitted to the Alaska Railroad so that preparations could be made well in advance of field installation of the test section at the Montana test site.

Installation

Two flatcars carrying the ties and fastenings arrived in Anchorage on October 8, 1973. Installation was planned for the 9th, however, due to heavy snowfall on that day installation began on October 10, 1973.

In view of steadily falling temperatures and 8 in. (203 mm) of snow on the ground at the Montana test site, it was decided not to totally "open up" the mainline track. If freeze-up took place before installation was complete, all rail traffic would have to operate over the siding track for the entire winter.

On the first day of installation, a ballast regulator was used to remove snow from the track. Then the regulator cut down the ballast on the east side of the track so the timber ties could be removed and concrete ties installed from that side of the track. Timber ties were removed so that the concrete ties could be placed in sets of two with a timber tie between them to hold rail gage during installation.

The tie inserter worked well in positioning concrete ties under the rails from the east shoulder, where they had been placed by an on track crane during unloading. The ballast was manually removed, using shovels, to make room for the concrete ties.

Fastening plates, adjustment blocks, and the basic 1/4-in. (6.4-mm) thick wood shims were assembled on the rail seats of the ties. The coil bolts were then inserted and tightened. Initially, one of the most time-consuming tasks was screwing the coil bolts down prior to applying the specified torque. This was due partly to a shortage of proper wrenches on the job site. More wrenches were obtained to speed up this operation.

The crew experienced some difficulty in installing the Pandrol clips. This was due primarily to a combination of inexperience, and the clips gouging the steel base plate while they were being driven into the shoulder holes. Also, it was found that the plastic plugs, removed from the coil bolt holes during the unloading of the ties, had to be reinstalled to prevent ballast particles from entering the bolt holes during tie installation. Torque applied to the coil bolts ranged from 0 to 250 ft-lbs (0-339 N m).

A total of twenty-three (23) concrete ties were installed the first day. During the next four days, the crews installed thirty-eight (38), forty (40), forty-one (41), and thirty-nine (39) concrete ties, respectively. After the first day's experience, the crew became familiar with the procedures and they increased their daily production by about 70%.

After completing the tie installation, an Electromatic tamper was used to raise the track. Blocks had to be placed under the jacking feet to clear the ends of the concrete ties. After raising, the section was tamped with electric vibratory hand tampers.

Early the following week, an inspection of the test section disclosed that some spots were out of crosslevel. The ends of most ties under the rail joints required raising to support the rail at the joints. This occurred because installation of Pandrol clips at the joints could not be completed during initial installation. This work was completed within the week.

A permanent bench mark made of a 4-in. (102-mm) pipe with 1-in. (25 mm) rod inside was driven east of Station 0+00. The postconstruction top of rail level readings on the test section and the pre-winter levels on the control section were taken on October 24, 1973. On that date, it was discovered that the section crew had made two errors while numbering the individual ties. There were two adjacent ties numbered 122 and none numbered 171. On October 26, 1973, the ties were numbered correctly with a hand marker. Measurements were taken to secure a beginning record of the bolt positions relative to the slots in the base plate. This was necessary since the tie centerlines did not coincide exactly with the centerline of the track.

When construction was started, there was 8 in. (203 mm) of snow on the ground at the Montana test site. During the last two days of tie installation (October 13 and 14), the temperature at 7:00 a.m. in Sunshine, Alaska, six miles to the North, was 18 F (-8 C) and 5 F (-15 C), respectively. The crews reported the ballast was frozen to a depth of about two inches on both of these mornings. The ties were installed without breaking the rail or closing the mainline to traffic.

Post-Installation Comments

Some Pandrol spring clips were more difficult to install than expected. This was due to a gouging by the sharp edge of the end of the clip into the flat steel base plate within the shoulder recess. The problem was caused by misalignment. Design of the clip was such that as it was being driven into position, a much greater resistance existed at the contact point between the clip and heel of the shoulder than at the contact point between clip and rail base. This caused the clip to rotate in the direction of least resistance causing misalignment. Heavy blows on a clip in the misaligned position produced the aforementioned gouging that further impeded driving of the clip into position. Once the clip was over the heel of the shoulder it becomes self aligning within the shoulder recess. The clip was then easily driven into its proper position.

Aside from the time lost in the installation of some clips, there was no apparent damage other than the gouging of the plate. In fact, the new fasteners proved to be extremely durable as demonstrated by their ability to absorb heavy blows from sledge hammers and track mauls without visible damage. Pandrol clips used in future installations of this type should be modified to eliminate the difficulties described.

Once installed there should be no reason for removing clips. If track adjustment is required, the fastener bolts should be loosened and the rail jacked to the proper position. The clips will hold the fastener to the rail so that when the rail is relocated, all that need be done is to insert shims and retorque the bolts. A similar procedure should be followed when removing shims in the spring except, in this case, the rail will have to be raised. After removing the required number of shims, the bolts should be retorqued.

A part of this program was a comparison of man hours expended in track adjustment at the Montana installation per unit length of track for the concrete tie test section versus the wood tie control section. The validity of this comparison is dependent, to some degree, on the capability of the tools used to adjust the concrete tie track.

The operation likely to require the most time is loosening and retightening the fastener adjusting bolts. The torque wrench used during fastener installation required four revolutions of the lever arm for each revolution of the bolt. Although this was an excellent torque wrench and required a small effort to torque the bolts, it took much longer than with a direct drive, ratchet-type torque wrench.

The fasteners at the Montana test site are ideally suited for the use of electric- or air-powered impact wrenches. Their use would greatly speed up the concrete tie track adjustment. If a portable source of electric or air power is unavailable for operating impact wrenches, then a direct drive, ratchet-type torque wrench should be used. This should be supplemented with shorter handled ratchet-type wrenches for running the bolts up and down after the long-handled wrench has been used to remove bolt tension.

On the subject of bolt torque, the specified value of 250 ft-lbs (339 N m) is not necessarily critical. A torque value between 225 to 275 ft-lbs (305 to 373 N m) may provide adequate fastener performance. Although, if vertical adjustment requires 3 to 5 in. (76 to 127 mm) of shims, bolt torques should be kept near the high end of the range and certainly not less than 250 ft-lbs (339 N m).

The concrete tie track was tamped manually to final grade, because the tamping tools of the Electromatic tamper would not extend deep enough to compact the ballast beneath the ties.

Data Acquisition

On October 24, 1973, following completion of the Montana test section, Alaska Railroad personnel obtained top of rail profile for both rails of the test and control sections. This was done to establish a basis for comparison of top of rail profiles taken subsequently during the winter months.

Standard surveying methods were used to obtain top of rail profiles. The test and control sections were stationed at 10-ft (3.05 m) intervals. Station 0+00 marked the south end of the test section, while Station 3+90 was the north end of the test section and south end of the control section. The north end of the control section was Station 7+80. Profiles were obtained by taking top of rail elevations at each 10-ft (3.05 m) intervals from south to north for both rails. Elevations were taken to the nearest 0.01 of a foot (3.0 mm).

Track adjustment data consisted of a record of shim thicknesses placed or removed at each tie on the date that adjustments were made. A positive number denotes an added shim thickness and a negative number is shown when a shim is removed. These data for the concrete tie test section and the wood tie control section are presented in Tables 1-16 and 17-19 respectively, in Appendix D.

Temperature data were obtained by the Alaska Railroad from weather stations at Sunshine and Willow, 6 miles north and about 30 miles south of the test site, respectively. These data consisted of weekly average temperatures at each reporting station and the lowest temperature during each of the reported weeks. Sunshine station data includes the period from the week ending October 14, 1973 to the week ending June 2, 1974. Willow station data includes the period from the week ending October 10, 1973 to the week ending June 2, 1974. These data are presented in Table 20, Appendix D.

Data gathering events are summarized in Table 21, Appendix D.

Data Reduction

Top of rail profile data taken periodically by the Alaska Railroad were submitted in the form of level notes. To reduce these data to a usable form, for comparison of the various top of rail profiles taken during the project, the following procedure was used:

1. Top of rail profiles for the as-built test and control sections were plotted as straight lines. Subsequent profiles were plotted against the as-built profiles as differences in elevation measured at each 10-ft (3.05 m) interval. Figures 1 and 2 show these comparisons for all the top of rail profiles taken during the project.
2. Examination of Figures 1 and 2 shows that both the test and control sections moved upward (heaved) during the winters and moved downward during frost free periods. However, November 19, 1974 profiles show that the test and control sections were considerably lower than the as-built elevation prior to the 1974 winter heaving. This is evident also in the November 6, 1975 profiles, although to a lesser degree than during the frost free period prior to the 1975 winter heaving. The July 1976 profiles show that the south end of the test section and the north end of the control section were lower than "as built" while the north end of the test section and the south end of the control section were higher than "as built". An examination of the data did not indicate any particular reason for these trends.
3. To better examine the individual rail profiles, each top of rail profile was also plotted separately. Because the test and control sections were not on a perfectly level grade, the exaggerated vertical scale caused the profile lines to ascend sharply from left to right as the profile was plotted. This tended to distort the profile. To eliminate this problem, each

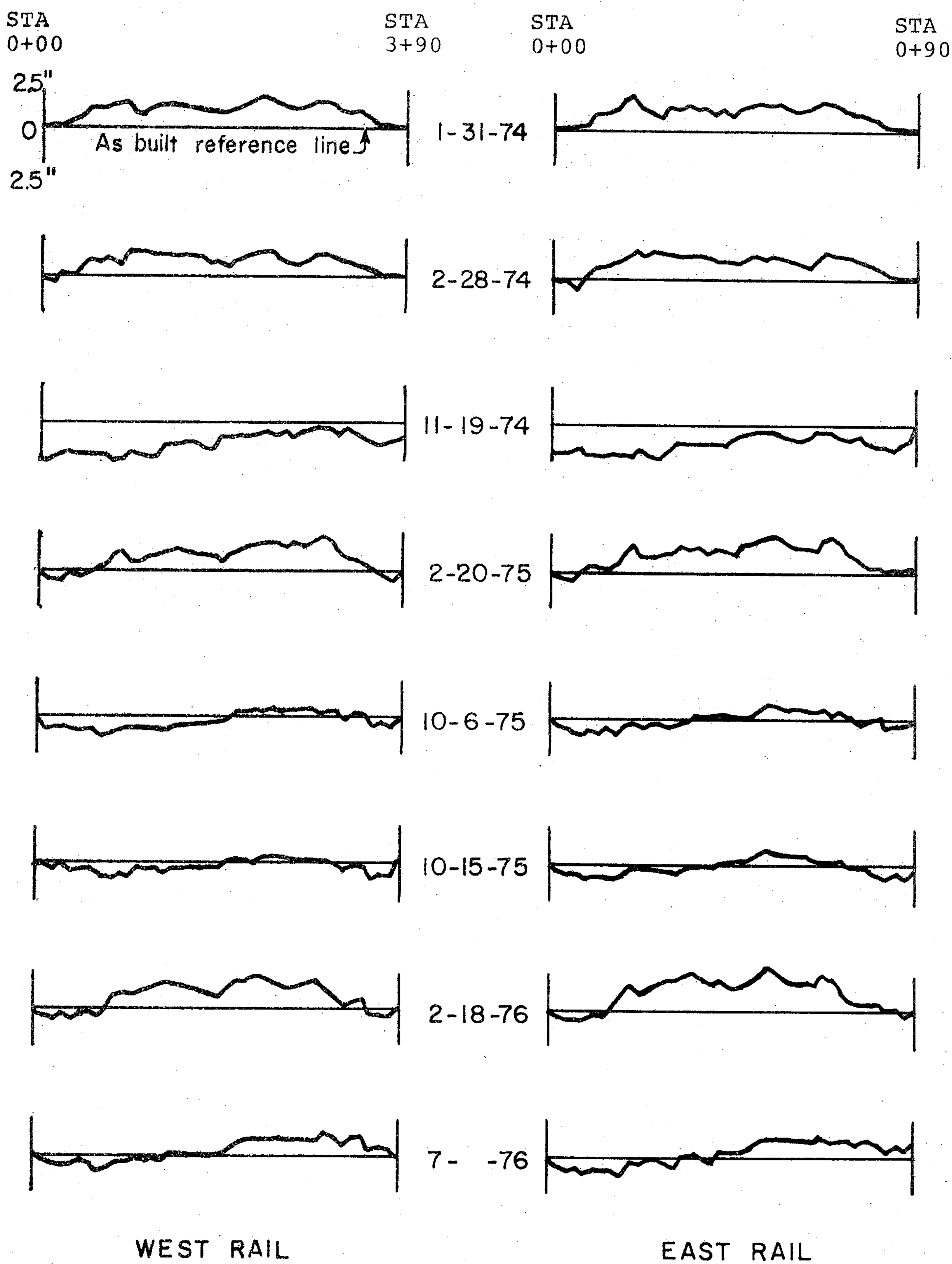


FIG. 1 - RAIL PROFILE CHANGE
MONTANA CONCRETE TIE SECTION

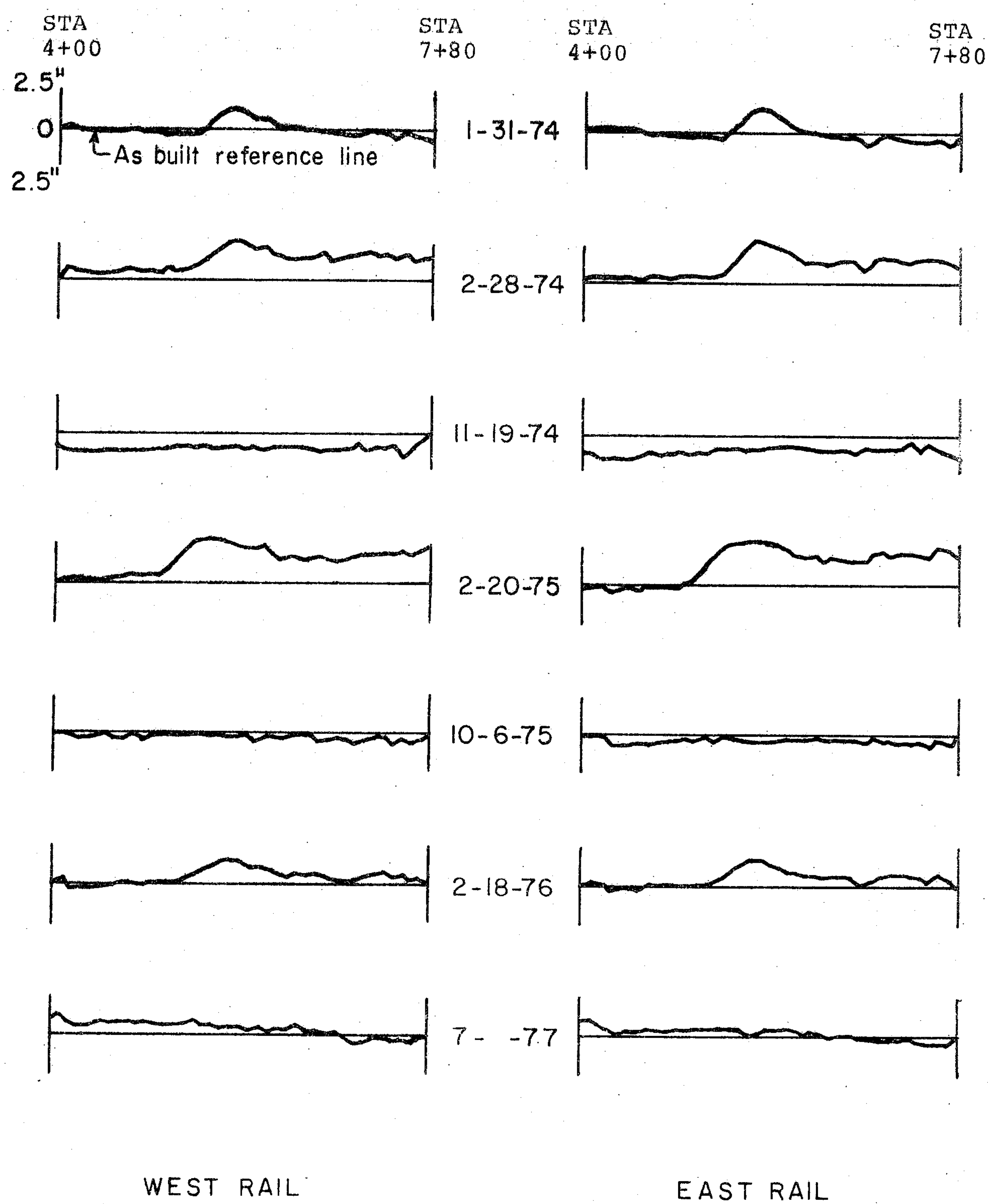


FIG. 2 - RAIL PROFILE CHANGE
MONTANA WOODEN TIE SECTION

profile was replotted as though both ends of the test and control sections, respectively, were at the same elevation. The top of rail profiles taken during the project are plotted in Figures 3 and 4.

Data Analysis

The "as built" rail profiles of the test and control sections are shown at the top of Figures 3 and 4, respectively. The test section was constructed with a high spot of 0.48 in. (12.2 mm) near the south end and a sag of about 2.76 in. (70.1 mm) near the two-thirds point. The control section also contained a sag of 1.44 in. (36.6 mm) near its midpoint.

Test and control section profiles taken on January 31, 1974 show that several heaves developed in the test section and most of the track moved upward. Shims were installed on either side of the heaves to correct the track surface.

Profiles of the test and control sections taken on February 28, 1974 show the smoother contour of the test section resulting from shims placed on February 26, 1974.

The November 19, 1974 profiles of the test and control section were taken when there was no frost. Although somewhat rougher, the "as built" sags could still be seen.

February 20, 1975 and April 18, 1975 profiles for the test and control sections show heaving similar to the first winter. Although numerous shims were placed in the test and control sections, it would appear that additional shims should have been placed to smooth out observed rough spots, particularly in the test section.

October 6, 1975 profiles were taken immediately prior to field welding the 21 rail joints in the test section. The original sags were evident though somewhat modified in both the test and control sections.

Test and control section profiles taken February 18, 1976 show that heaving, similar to that observed on February 20, 1975 and April 18, 1975, was evident, though not quite as severe as observed during the first winter.

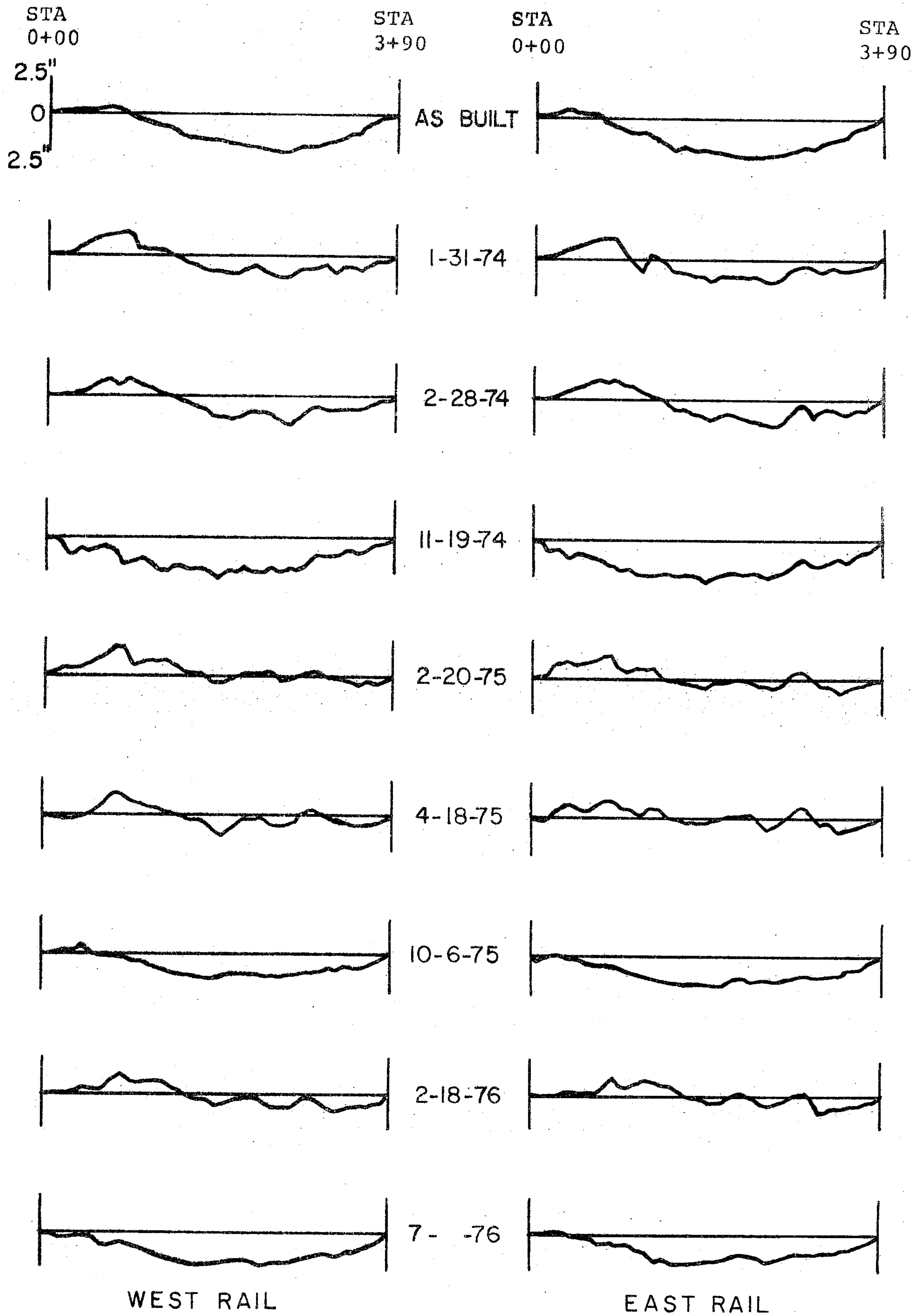


FIG. 3 - RAIL PROFILES

MONTANA CONCRETE TIE SECTION

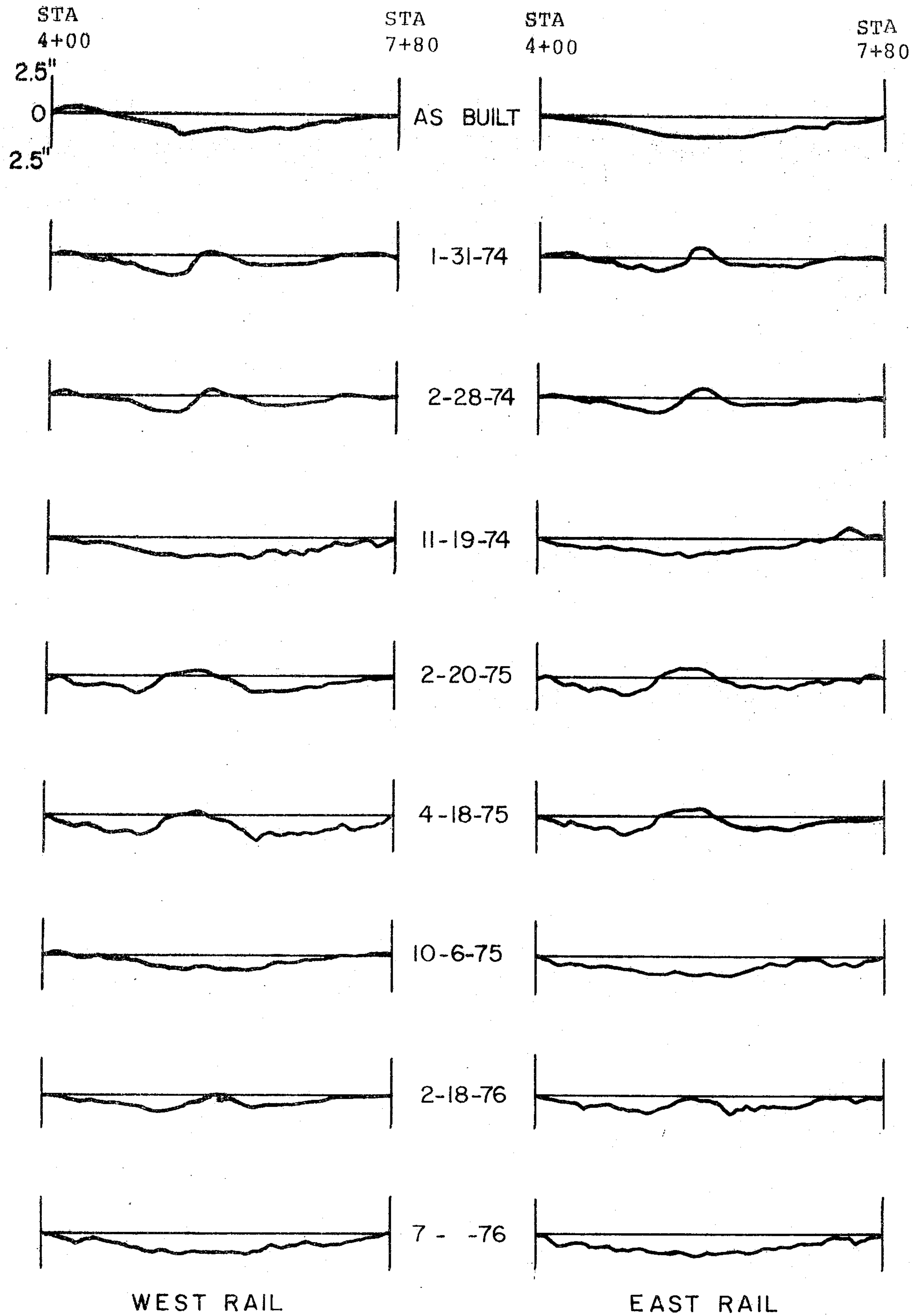


FIG. 4 - RAIL PROFILES

MONTANA WOODEN TIE SECTION

Final profiles of the test and control section were taken during July 1976. The original "as-built" sags, though somewhat modified, were still evident.

Track Adjustments

During the first winter shims were added to the test section on four separate occasions, compared to once for the control section. Table 17 shows that not only were shims added, but tie plates were removed in the control section. This greatly reduced the number of shims needed to correct track surface on either side of the heave in the control section.

As previously noted two major heaves were located in the test section area while one major heave was located in the control section area. The heave in the control section did not change significantly during the project. The two major heaves in the test section appeared to have developed into one major heave plus two, or possibly three, minor heaves. This trend appeared to hold during the remaining two months.

There is no sure way to compare the ease of adjustment in the test and control sections. Amounts of adjustment performed on each section was decided by railroad field crews. Top of rail profiles taken during the three winters indicated that additional adjustments could have been made in both sections. Additional shims could have been installed to improve the test section surface.

Tee headed wrenches were used to loosen and tighten the hold down bolts in the test section. The November 28, 1975 adjustment required 4 man hours to place shims at 34 fastenings. On March 16, 1976 4 man hours were required to place shims at 26 fastenings. The time required to remove snow from the fastenings was about the same, but the majority of the time on adjustments was spent in loosening and tightening the bolts. Use of an electric impact wrench in place of the tee headed wrenches would have greatly reduced the time needed to adjust the test section. The impact wrench could be powered by a portable gasoline generator mounted on a handcar towed by a motorcar.

The adjustable fasteners used in the concrete tie test section was a viable system, fully capable of providing adequate track adjustment in areas subject to frost heave. Although the fastenings are capable of providing up to 1-3/4 in. (44.5 mm) of lateral adjustment in either direction, the need for this degree of adjustment was not demonstrated. Future fastenings of this type should be modified by reducing the lateral adjustment, in either direction, to 1/2 in. (12.7 mm). This would reduce the length of tie plates by about 5 in. (127 mm). However, in heavy trackage areas the larger plates should probably be retained to prevent overloading of the wood shims.

Track Inspections

The test and control sections were inspected during early July of 1974, 1975, and 1976. During these inspections the concrete ties were checked for flexural cracking at the rail seats and tie center cross sections. The fastenings were inspected for distress such as bent or broken hold down bolts, broken or displaced spring clips, broken shoulders, or displaced pads.

Inspection in July 1974 showed the ties to be crack-free and the fastening to be undamaged, except for a slight migration of some rail seat pads. This was particularly evident at rail joints where spring clips were not installed due to insufficient space around the joint bars. Tie ends directly under the joints had begun to settle leaving a small gap between the rail-seat pad and the base of the rail. Several hold down bolts were removed and found to be bright and unbent. Wood shims that had been removed were found to be undamaged and reusable.

Inspection in July 1975 showed the ties were still uncracked. Fastenings were undamaged but the pad migration was worse. Pads under the joints and for several ties either side of the joints were displaced, probably due to the action of rail traffic over the low joints. Several pads were badly mashed. The low joints and associated pad problems were considered to be unacceptable for the future integrity of the test section. To eliminate this problem the 21 joints in the test section were field welded and spring clips were installed at the joint locations. This was done during October 1975, at which time some of the displaced pads located on either side of the joint were repositioned.

When inspected in July 1976, the concrete ties were still crack-free with the fastenings in excellent condition. All the pads that had been repositioned during October 1975, including the ones placed beneath the welded joints, had remained in place. Any remaining displaced pads were repositioned.

Thus, the study indicates that the RT-7S concrete ties were capable of surviving in a frost heave environment and the unfavorable support conditions associated with frost heave. In addition, the inspection indicated that the use of welded rail with concrete ties provides a good solution to joint problems. Photographs of the Montana Test Section are presented in Appendix E.

HOUSTON TEST SITE

Research Approach

To satisfy the requirements of Objective 3, a new wood tie track test section supported on a stabilized ballast layer was designed and installed. Periodic inspections were conducted to determine the response of the stabilized layer and the stabilizing medium to sub-freezing temperatures. Breakdown of the stabilized layer would be evidenced by loosening of the ballast particles.

A careful inspection of the completed track was made prior to the first winter and observations recorded as a basis of comparison for future inspections. Inspections were made by Portland Cement Association following each of the three winters. An additional inspection was made in mid-winter by the Alaska Railroad to determine the elasticity of the stabilized medium and identify any heaving or other conditions that may be damaging to the stabilized ballast. Any evidence of foundation heaving was noted and the affected area was checked for damage during subsequent inspections.

To determine the ability of the stabilized ballast layer to reduce track subsidence caused by weak foundation support, top of rail profiles were surveyed for both the test section and a control section. Because track subsidence takes place over a period of many months, top of rail profiles were taken by the Alaska Railroad immediately after each of the three winters, and sent to Portland Cement Association for analysis.

Inspection and Design

The Houston test site was inspected during the first week of July 1973. One major area of track subsidence was observed by sighting down the rails. By staking the extremities, this sag was measured at a length of 312 ft (95 m). Although Alaska Railroad personnel reported the existence of other areas of track subsidence in the mainline track, these were not clearly visible. Hence, it was not possible to identify any specific length of track as a control section. However, directly opposite the selected test section the siding track showed a similar sag. This indicated that the area of weak foundation support extended the full width of the double track embankment. Therefore, it was decided to locate the control section in the siding track directly opposite the test section in the mainline track.

Since the test section was to be supported on a stabilized ballast layer, the existing track structure had to be removed down to the subgrade. It was decided that the stabilized layer should be 9-ft (2.7-m) wide and 8-in. (203-mm) thick and should extend the full 312 ft (95 m) length of the test section.

To avoid damage to the stabilized ballast layer during future tamping, an additional 4 in. (102 mm) of ballast was placed between the top surface of the stabilized layer and the bottom surface of the ties. A 6-in. (152-mm) stabilized shoulder was placed along the top of both sides of the 9-ft (2.7-m) wide stabilized layer. Thus, the track was contained in a trough of stabilized ballast, and tamping would be possible without difficulty. New crushed gravel ballast was used throughout.

The control section was tamped to grade using ballast from the existing mainline track. Following installation and establishment of a permanent bench mark, top of rail profiles of the control and test sections were taken by the Alaska Railroad. These profiles would serve as a basis of comparison with future profiles.

Stabilizing Medium

The stabilizing medium used in the Houston test section was "Petroset RB", manufactured by the Phillips Petroleum Company of Bartlesville, Oklahoma. The material is an emulsion that is broken upon contact with anhydrous ammonia. The result is an elastic, rubberlike material that bonds rock surfaces at points of contact. When used to stabilize ballast the following procedure is used:

1. An ammonia solution is sprayed at a fixed rate on the top surface of ballast. The ammonia percolates through the ballast. At each point of contact between adjacent particles, the ammonia tends to collect in the form of a meniscus.
2. Within 5 to 15 minutes after the ammonia solution is applied, the Petroset emulsion is sprayed. The emulsion percolates through the ballast and also collects at the contact points between adjacent ballast particles. At these points, the emulsion merges with the ammonia and a reaction occurs that breaks the emulsion. Water is released and the result is an elastic, rubberlike material binding adjacent ballast particles together at their points of contact.

There are two constraints to the successful use of this rock binding emulsion, assuming all application procedures are properly followed. These are:

1. Ballast must be clean and open to allow proper penetration of the materials.
2. Temperatures must not be less than 40 F (4 C). The warmer the temperature, within reason, the better the results.

Humidity may have an influence on the materials after spraying. Following installation it was found that proper curing did not occur over the full depth of application. Whether this was caused by humidity or some other factor has not been identified.

Installation

All necessary equipment and materials were placed on the Houston siding September 10, 1973. The next day three temporary bench marks were placed, and top of rail elevations for the test and control sections were taken at 10-ft (3.05-m) intervals for the test sections and 50 ft (15 m) beyond each end limit.

On the 12th of September, the existing rails and ties were removed. Existing ballast was removed to a depth of 19 in. (483 mm) below the top of ties. Two hundred cubic yards of washed ballast were dumped on the test section. This was spread uniformly with a motor grader. Final grading and compaction of the 8-in. (203-mm) thick ballast layer was completed by noon on the 13th of September.

After calibrating the flow rates of the 5-nozzle spray bars for the ammonia solution and the Petroset, proper quantities of these materials needed to cover one-half 156 ft (47.5 m) of the test section were placed in tanks. These amounts were 12 gals (0.05 m^3) of 29% ammonia mixed with 203 gals (0.77 m^3) of water and 385 gals (1.46 m^3) of Petroset.

The ammonia application on the northern 156 ft (47.5 m) of the test section took 7 minutes and 45 seconds. Rail spikes were placed at 10-ft (3.05-m) intervals to serve as a guide for the rate of application. The rate of application was controlled by covering each 10-ft (3.05-m) interval in thirty seconds.

With the pump running at full capacity, pressure in excess of 30 psi (206.8 kPa), the Petroset application required 1-1/2 minutes per 10-ft (3.05-m) interval. Since this speed was difficult to control, a 45-second interval was used per 5-ft (1.52-m) distance. Thus, if a 5-ft (1.52-m) distance was covered too rapidly, the crew quickly returned to the start of the 5-ft (1.52-m) length to begin again at the proper speed. This worked well in holding the coverage per 5-ft (1.52-m) increment to 45 seconds.

Due to curling of the suction hose in the tank, the tank could not be emptied completely. This limited the Petroset coverage to 135 ft (41.2 m), the northern half of the test section. Because of cloudy weather and possible rain the treated portion was covered with plastic. By noon, on September 14th, the remaining 177 ft (54 m) was treated. Placement of ties and rails was started on the first 135 ft (41.2-m) of the test section which by that time had set to a depth of 2 in. (51 mm).

On September 15th, ties and rails were installed on the remaining 177 ft (54 m). By 10:30 a.m. the 135-ft (41.2-m) portion had set to a depth of 3 in. (76 mm) and the 177-ft (54-m) portion had set to a depth of 2 in. (51 mm). The coloration of the 135-ft (41.2-m) portion was brown but the 177-ft (54-m) portion, that had not been covered with plastic, had a light yellow appearance.

Two hopper cars of ballast were dumped on the test section. This was placed in the center to half the rail-depth with a ballast regulator. The ballast was hand-jacked and tamped the full length of the test section to 1/4 in. (6.35 mm) above calculated grade. An Electromatic tamper then tamped the entire test section. Following this, shoulders were dressed and shaped with a ballast regulator.

All work except shoulder spraying was completed by 3:00 p.m., spraying was done by 4:00 p.m. Due to the suction hose curling the northern 90 ft (27 m) of the right-hand shoulder did not receive the Petroset. Pumps and hoses were flushed, all material and outfit cars were switched to the siding, switches were lined, and the Dispatcher notified that the mainline was open at 4:45 p.m.

September 14th was a clear and sunny day. The noon temperature in the shade was 55 F (13 C) with the estimated daytime temperature of 60 to 65 F (16 to 18 C) in the sun.

Data Acquisition

On September 21, 1973, the Alaska Railroad obtained top of rail profiles for both sections.

The surveying technique used for obtaining top of rail profiles was similar to that at Montana. The test section was located in the mainline track. The control section was located in the parallel siding track directly opposite the test section. Because the exact length of the sag was not known, track limits were extended for 50 ft (15 m) at the east end and 62 ft (19 m) on the west end.

Profiles were obtained by taking top of rail elevations at each 10-ft (3.05 m) interval from east to west. Elevations were taken to the nearest 0.01-ft (3.05 mm). The data obtained were sent to Portland Cement Association in the form of surveying notes.

Top of rail profiles of the test and control sections were taken during July of 1974, 1975, and 1976. These were taken to observe the development of any trend in long term settlement.

The first visual inspection by Portland Cement Association, scheduled for October 1973, could not be made due to heavy snow cover at the test site. The test and control sections were inspected during July of 1974, 1975, and 1976. During these inspections numerous photographs were taken.

Data Reduction

Top of rail profiles for the "as built" test and control sections are plotted as straight lines. For all other profiles, the difference between the "as built" and subsequently measured elevations, at each of the 10-ft (3.05 m) stations along the track, is plotted against the straight lines. Figures 5 and 6 show these comparisons for top of rail profiles taken during the project.

Data Analysis

The rail profiles in Figures 5 and 6 show the preconstruction top of rail profiles plotted against the "as built" profiles for the test and control sections, respectively. Figure 5 shows that up to 2.76 in. (70 mm) of sag was removed from the track during construction of the test section.

Figure 6 shows that little or no sag was taken out of the control section. Actually, the only effort on the control section was to add some ballast removed from the mainline track and tamp it.

The July 1974 profile plotted against the "as built" profile show that settlement up to 0.84 in. (21 mm) occurred

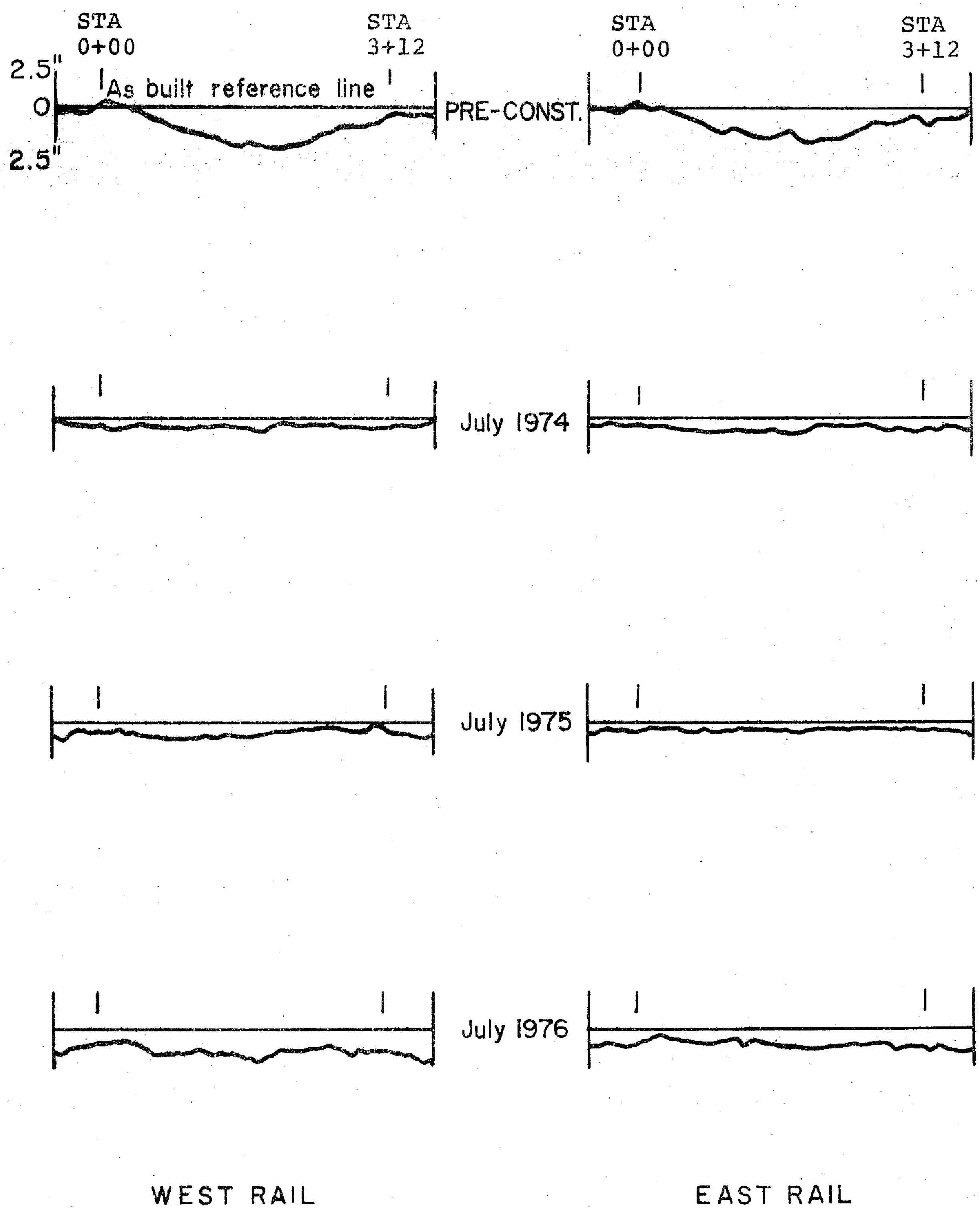


FIG. 5 - RAIL PROFILES
HOUSTON STABILIZED BALLAST SECTION

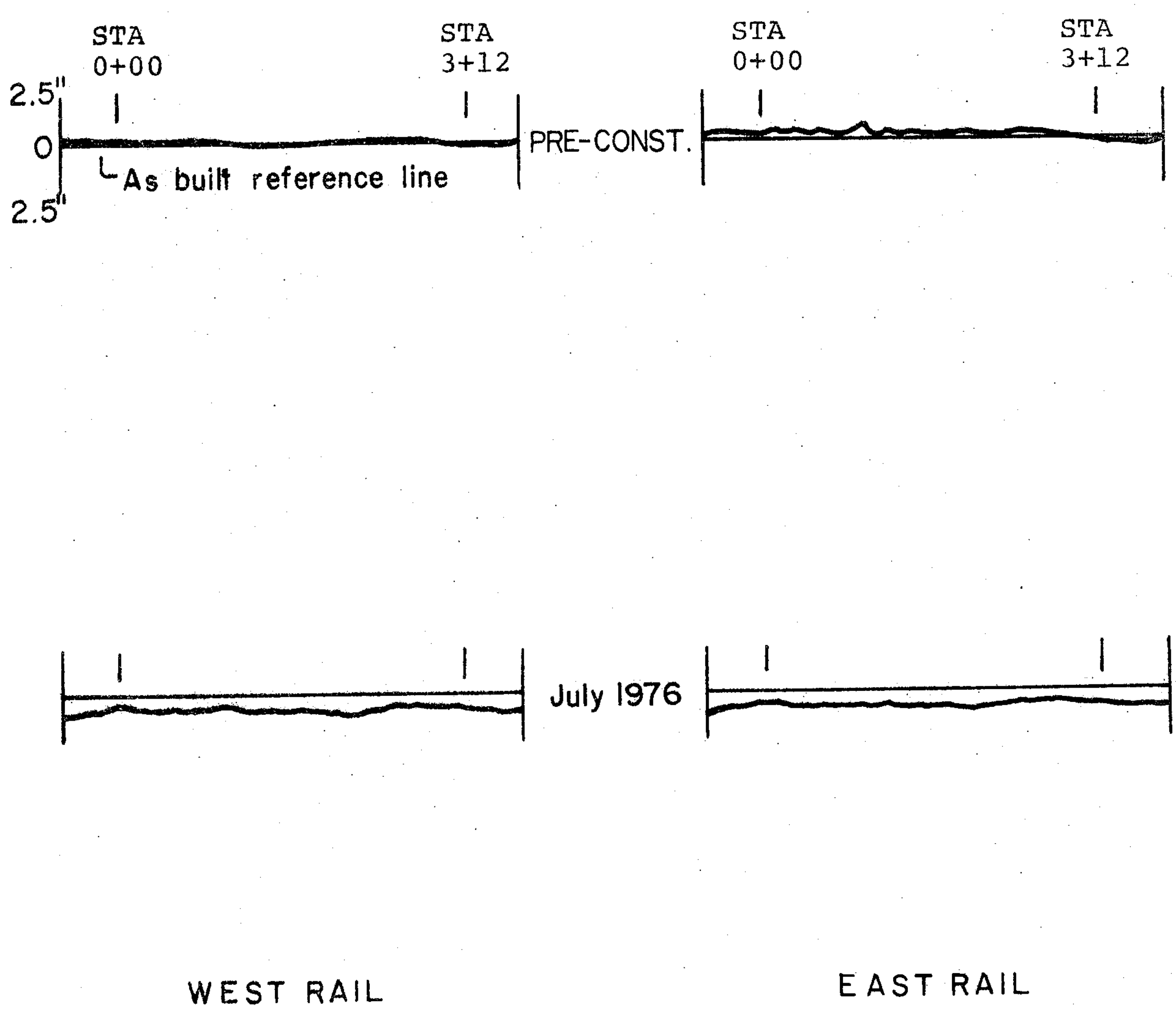


FIG. 6 - RAIL PROFILES
HOUSTON CONTROL SECTION

during the first winter. Settlement of new track is normal, particularly with gravel ballast. A good part of the first winters settlement can be attributed to resurfacing of the track.

The July 1975 profile plotted against the "as built" profile shows a maximum settlement of 0.96 in. (24 mm) as compared to 0.84 in. (21 mm) the previous year. In general, settlement, except for one or two locations, was substantially the same as the preceding year. However, beyond the end of the test section, the untreated track appeared to be settling with respect to the test section.

The shape of the July 1976 profile plotted against the "as built" profile was similar to the 1975 profile except the low spots had become worse (lower). In general, the settlement in the entire track (both rails) was uniform. The profile also shows the untreated track beyond the ends of the test section.

For the control section, the July 1976 profile is compared against the "as built" profile in Figure 6. This profile is similar to the original profile. It appears the entire track (both rails) had settled uniformly. This amount was about the same as the test section on the same date and was equal to 0.84 in. (21 mm). This indicates that, either the entire embankment settled this amount, or perhaps the reference rod moved upward by this amount during the winter of 1975-1976.

Judging from elevations in the untreated track, beyond the ends of the test section, it would appear that the stabilized ballast layer was effective in reducing track subsidence.

Track Inspections

The July 1974 inspection revealed the following:

1. Ballast outside the stabilized shoulders showed evidence of slumping downward and out away from the top of the shoulders. A maximum slump of about 1 in. (25 mm) occurred on the outside of the left shoulder referenced to increasing mile posts, at several locations along the test section. The same condition, though to a lesser degree, existed outside the right shoulder.

2. A distinct gap had appeared between the inside of the shoulders and the ends of the ties. This gap was about 2 in. (51 mm) wide and at the worst locations it was as deep as 4 in. (102 mm). Evidence indicated that the ballast particles, in the cribs between the ties, had been in motion. There was no evidence to suggest that the shoulder had moved outward.
3. Each box-anchored tie showed evidence of longitudinal back and forth movement in the ballast. This movement had exposed the tie sides to about mid-depth of the tie. The displaced ballast was deposited along and, in some cases, upon the adjacent ties. It was evident that the lack of interlock between ballast particles in the trough, at the stabilized ballast interface, had reduced lateral and longitudinal restraint in the track.

The July 1975 inspection revealed that the conditions observed during the 1974 inspection had become worse. The maximum amount of ballast slump at one location outside the left shoulder was 7 in. (178-mm). Numerous locations outside both shoulders showed slumping of from 2 to 5 in. (51 to 127 mm). At the location of the 7-in. (178-mm) slump, some ballast was removed from under the stabilized portion of the shoulder. This ballast was loose and easily removable. Ballast particles in the stabilized portion of the shoulder were firmly bound together and the exposed binding material at the top of the shoulder had hardened due to weather. At this location the stabilizing material had cured to a depth of 7 to 8 in. (178 to 203 mm).

Gaps between shoulders and tie ends had widened up to 3 to 4 in. (76 to 102 mm) for the full tie depth at numerous locations. Also apparent was a reduction in the quantity of ballast in the cribs between the ties. The general ballast level dropped by 1 to 2 in. (25 to 51 mm). In an effort to determine how the loss occurred, ballast particles at several locations along the left shoulder were paint-tagged.

Ties did not appear to have settled appreciably. By sighting down the rails, several low spots were observed especially at the joints in the left rail.

Box-anchored ties for some distance along the track, beyond the ends of the test section, showed evidence of back and forth movement. This was not observed during the 1974 inspection. It is unlikely that these ties moved because of the lack of longitudinal restraint of the box anchored ties in the test section.

The July 1976 inspection revealed that the conditions observed during previous inspections had progressively gotten worse, though not as much as anticipated. Low spots at the joints were somewhat lower and there appeared to be less ballast in the cribs between the ties. A check of the paint-tagged ballast particles was inconclusive. It was not possible to tell if they had moved in an outward direction beneath the shoulders. At one location the ballast near the tie ends was removed down to the stabilized layer. This was still intact and very firm. Where the binding material had been directly exposed to the weather it was hard while the unexposed material was still elastic.

Incomplete curing of the shoulders down to the stabilized layer effected the performance of the test section adversely . Even though the stabilized layer appeared to reduce track subsidence, the poor performance of the shoulders and the lack of keying of ballast particles at the stabilized layer interface created an unsatisfactory track condition.

Photographs of the Houston Test Sections are presented in Appendix F.

MATANUSKA TEST SITE

During October 1975, 16 RT-7S ties with adjustable fastenings, similar to those at the Montana site were installed in trackage subject to severe heaving near Matanuska. Matanuska is located about 38 miles north of Anchorage.

Ties were spaced at 26 in. (660 mm) on center except in the middle of the test section where ties were placed on each side of the joint to allow the Pandrol clips to clear the joint bars. Center-to-center spacing of ties on either side of the joint was 36 in. (914 mm). The 1/4-in. (6.35 mm) wood shim was not placed between the tie plate and the concrete tie. No difficulty was encountered in shimming by as much as 4.25 in. (108 mm) during the winter of 1975-76 and 5 in. (127 mm) in the winter of 1977-78.

During July 1976, a detailed inspection of the test section was made. The ties were crack-free despite the 36-in. (914-mm) spacing. The 1-3/4 in. (44 mm) of shims in the ties, adjacent to the joint, produced a pronounced hump. These were removed and the rails settled to a good surface. A number of displaced rail-seat pads were repositioned during this inspection. Photographs of the Matanuska Test Section are presented in Appendix G.

CONCLUSIONS

Based on the data analyzed and conditions observed in track, the following conclusions have been reached:

1. The concrete ties with special adjustable fastenings provides a workable system, allowing extensive and repeated adjustments in tracks disturbed by frost heaving. With proper tools, adjustments can be made at least as easily as for conventional wood tie track. Plywood shims do not crush under the tie plates, and thus are reusable for several years. In the test installation, provision for lateral adjustment exceeded track requirements. Reducing this to plus or minus 1/2 in. (12 mm) would decrease tie plate size by up to 5 in. (127 mm). However, tracks carrying heavy traffic should have the larger plates to prevent overloading of the wood shims. Evidently, reliance on concrete ties with adjustable fasteners is a technically feasible way to avoid the spike-kill damage costs associated with timber tie track in certain locations.

2. The concrete ties are capable of withstanding the bending stresses induced by unfavorable support conditions associated with frost heaving. The ties were subjected to three years of service under sub-arctic conditions without any sign of distress. Other concrete ties with similar strength and modifications for the adjustable fastening systems should be expected to serve equally well.
3. If concrete ties are used in track having jointed rails, special clips must be used to secure the rails to the tie at joint bar locations. The practice of leaving the clips off at the joint is unacceptable. Alternately, special clips that will allow ties to be spaced immediately adjacent to the ends of the joint bars should be used.
4. Future installations of concrete ties and adjustable fastenings using a steel tie plate will require that special considerations be given to the pad placed between the rail base and the tie plate. Pads should not be eliminated unless special clips are designed to properly secure the rail to the fastener.
5. Sub-freezing temperatures and exposure to weather generally caused the binding material at the Houston test site to become hard and unyielding. However, the material remained rubbery when protected from direct exposure to the weather.
6. The stabilized ballast layer at the Houston test site was effective in reducing track subsidence due to weak foundation support. However, other details of the test section resulted in unacceptable track conditions.

REFERENCES

1. "Concrete Ties (And Fastenings)," Manual Recommendations, American Railway Engineering Association Bulletin 644, September-October, 1973.

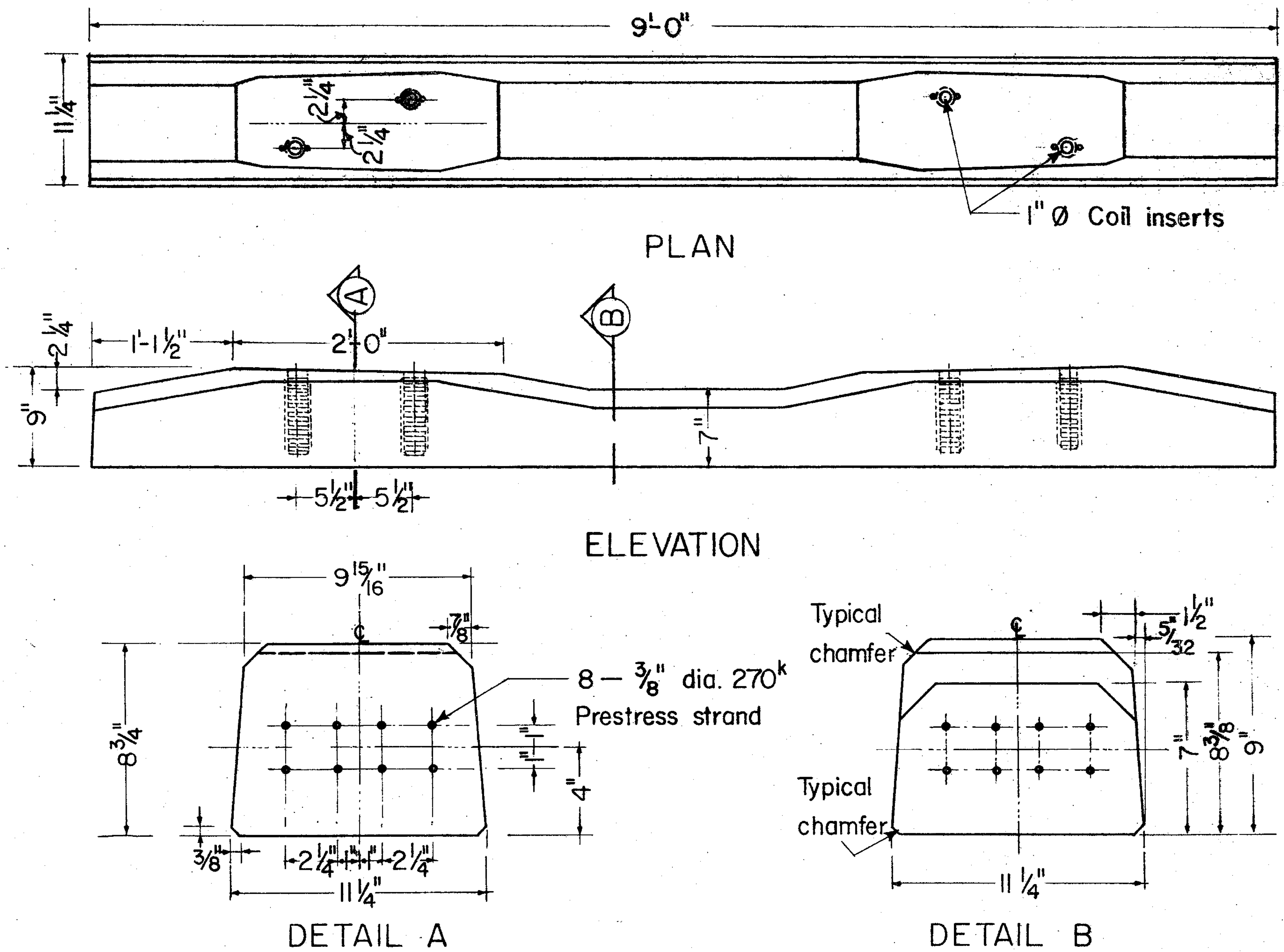


FIG. A-1 - GERWICK RT-7S CONCRETE TIE

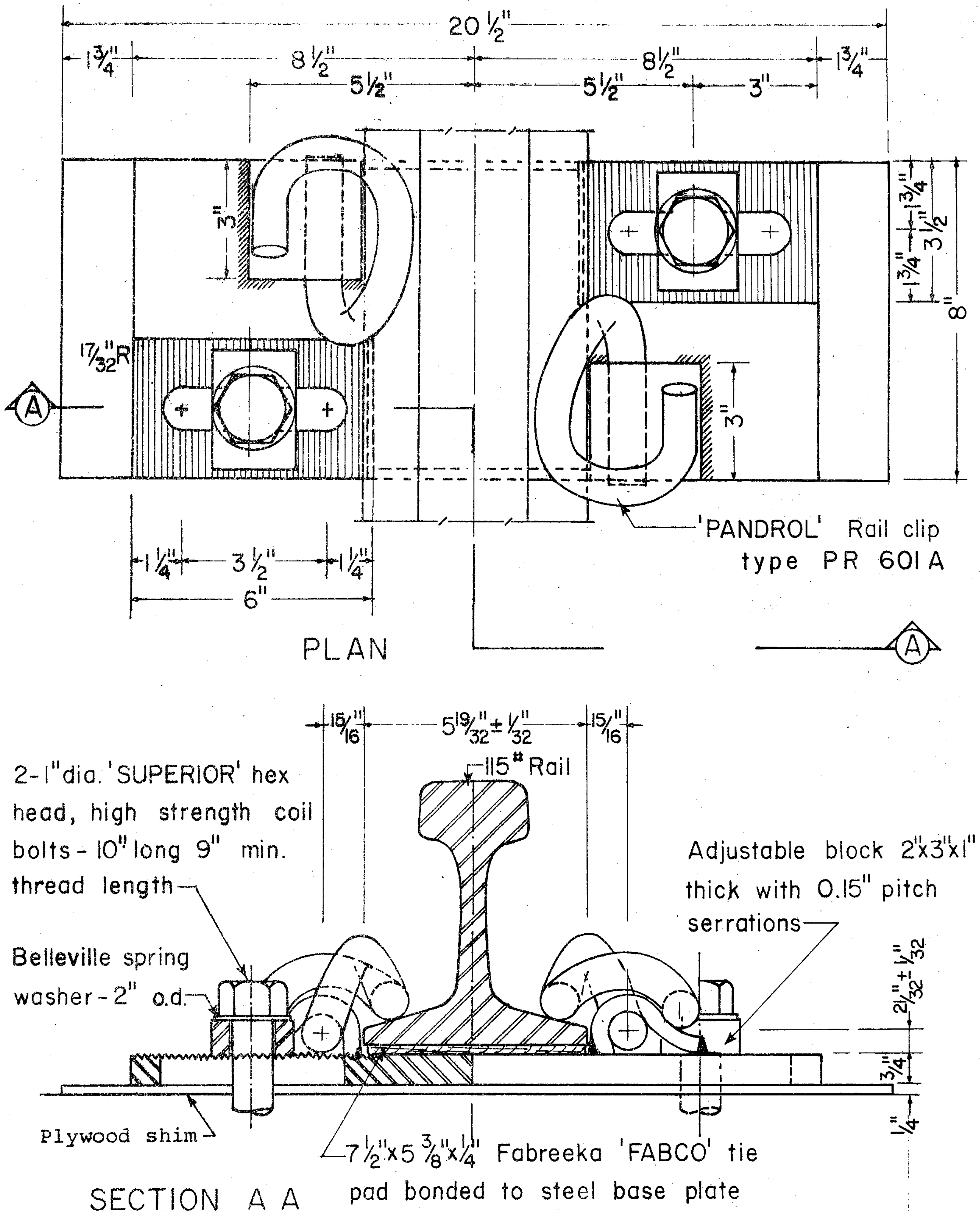


FIG. B-1 - ADJUSTABLE FASTENING SYSTEM

APPENDIX C

SPECIFICATIONS FOR PRESTRESSED CONCRETE TIES AND ADJUSTABLE FASTENING SYSTEM

General

Prestressed concrete ties shall be the Gerwick RT-7S, as manufactured by Santa Fe Engineering and Construction at Petaluma, California, and as shown in Figure A-1. The railseats and anchorage system of the ties shall be compatible with the adjustable rail fastener as shown in Figure B-1.

Concrete Tie Dimensions and Tolerances

Concrete tie dimensions shall conform to Figure A-1. Tolerances in length, width, depth, cant, and track gage shall be in accordance with AREA Preliminary Specifications for Concrete Ties and Fasteners of latest revision.

Quantity

200 prestressed concrete ties are required.

Acceptance

Acceptance of prestressed concrete ties will be based on the following:

1. Visual inspection at the point of manufacture and at destination.
2. Successful passage of performance tests to be conducted at the place of manufacture by personnel of the Portland Cement Association.
3. Seven day and 28 day concrete cylinder strengths shall be furnished for each days production.

Performance Tests

Five ties picked at random from the 200 ties produced shall be subjected to the following performance tests conducted at the plant at Petaluma, California.

1. Railseat Positive Moment Test
2. Center Negative Moment Test
3. Center Positive Moment Test
4. Insert Pullout Test

Should a test tie fail any of these tests, two additional ties shall be picked at random and tested. In the event either of these ties fails, the remainder of the ties shall be either tested or rejected at the option of the manufacturer.

Ties that pass these testing requirements and are not cracked or otherwise damaged after testing shall be considered acceptable for use in track.

Performance tests shall be conducted within 10 days following the last days production.

TEST DESCRIPTIONS

Test 1. Railseat Positive Moment Test

With the tie supported and loaded as shown in Figure C-1, apply a load "P" to produce a bending moment of 325,000 in lbs. The load shall be held for not less than 3 minutes during which time a careful inspection shall be made to determine if cracking has occurred. If cracking has not occurred the requirements of this test will have been met.

Test 2. Center Negative Moment Test

With the tie supported and loaded as shown in Figure C-2, apply a load "P" to produce a bending moment of 200,000 in lbs. The load shall be held for not less than 3 minutes during which time a careful inspection shall be made to determine if cracking has occurred. If cracking has not occurred the requirements of this test will have been met.

Test 3. Center Positive Moment Test

With the tie supported and loaded as shown in Figure C-3, apply a load "P" to produce a bending moment of 115,000 in lbs. The load shall be held for not less than 3 minutes during which time a careful inspection shall be made to determine if cracking has occurred. If cracking has not occurred the requirements of this test will have been met.

Test 4. Insert Pullout Test

An axial tensile load of 12,000 lbs. shall be applied to a rod threaded into the embedded insert. The load reaction points on the top surface of the tie shall not be less than

8 inches from the insert being tested. If the insert does not loosen and if cracking of the concrete around the insert does not occur the requirements of this test will have been met.

ADJUSTABLE FASTENERS

General

Adjustable fasteners shall be as shown in Figure B-1. The fasteners shall be compatible with the railseats and anchorage system provided in the prestressed concrete ties. Fastener hold down bolts shall engage the embedded coil type anchors in the ties without binding. Fasteners shall provide a nominal 4'-8 1/2" track gage with 115 lb. RE rail when each fastener is adjusted to the mid-point of its lateral adjustment range and with the serrations of the tie plates and hold down blocks fully mated.

Quantity

199 right-hand and 199 left-hand adjustable fasteners are required.

Shims

Shims shall be dimensioned as shown in Figure B-1. They shall be cut from high quality marine grade plywood or other wood capable of withstanding the loading and environmental conditions prevailing at the Montana test site on the Alaska Railroad.

The basic fastener shall utilize one 1/4-inch shim to act as a cushion between the fastener and the concrete tie.

Shims shall be furnished by the Alaska Railroad in packs for each fastener to consist of the following thicknesses:

2 @ 1-in. 2 @ 1/2-in., 1 @ 3/8-in., and 1 @ 1/4-in. These shims will allow a vertical adjustment from 1/4-in. through 3-5/8-in. in 1/8-in. increments for each fastener.

Acceptance

Acceptance of adjustable fasteners will be based on the following:

1. Visual inspection at the point of manufacture.
2. Successful passage of performance tests to be conducted at the Research and Development Laboratories of the Portland Cement Association at Skokie, Illinois.

Performance Tests

Two adjustable fasteners complete with shims and another bolts shall be fabricated by the manufacturer and shipped to PCA at Skokie, Illinois for performance tests. If a prestressed concrete tie containing coil anchors is available, the two fasteners shall be fastened to the tie for shipment. Otherwise, five coil anchors shall be shipped with the fasteners for casting into a concrete test block to be used for performance tests. Following successful passage of the performance tests and upon approval of the test results by represntatives of the Alaska Railroad, ORD&D, and PCA, production of 396 additional fasteners shall begin. Production fasteners shall incorporate any such modifications as may be necessary to assure adequate performance in track.

TEST DESCRIPTIONS

Test 1. Torque-Lateral Movement Test

This test is to establish a torque-tension relationship of the anchor bolts and the coil inserts so that a torque to produce optimum fastener performance can be used during the performance tests and during installation in track.

With the fasteners installed on the tie or test block complete with rail and shims, (if the tie is used it is to be cut into 2 pieces and secured at 27-in. on centers. If a test block is used, the coil inserts are to be cast into the block so that the fasteners are located 27-in. on centers), a series of pure lateral forces is to be applied to the edge of the rail base at a point midway between the fasteners. These forces are to be defined as the force necessary to just overcome the frictional resistance to lateral movement of the fasteners at each of the following bolt torques: -25, 50, 75, 100, 125, 150, 175, and 200 ft. lbs.

Note: We are looking for a frictional resistance to lateral slippage for each fastener of at least 5 kips or a total of at least 10 kips for the two fasteners set up. The bolt torque within the above range that will provide the desired resistance to lateral slippage will be used for all subsequent testing.

Test 2. Fastening Repeated Load Test

(a) With a section of new 115 lb. RE rail secured to a complete fastener assembly, the upward vertical load "P" to just cause separation of the rail from the railseat pad or the pad from the fastener base plate whichever occurs first shall be determined.

(b) A section of new 115 lb. RE rail is to be secured to two complete fastener assemblies, including shims. The fasteners are to be spaced 27-in. on centers on either the two tie blocks or on the concrete test block whichever was used in Test 1. Alternating downward and upward loads are to be applied to the head of the rail at an angle of 20 degrees to the vertical axis of the rail at the midpoint of one of the two fasteners. The second fastener serves only to stabilize the rail within the fastener to be tested. The alternating loads shall be applied at a rate not to consist of both a downward and an upward load. The magnitude of the upward load shall be 0.532 P where P is the load determined in part (a) of this test. The magnitude of the downward load shall be 30 kips. Both the upward and the downward loads are to be applied by a single double acting hydraulic ram.

Rupture failure of any component of the fastening system shall constitute failure of this test.

Test 3. The Fastening Longitudinal Restraint Test

This test as described in Article 6.9.1.12 of the AREA Preliminary Specifications for Concrete Ties andFastenings[1], is to be carried out on a single fastener. Loading arrangement is shown in Figure C-4.

Test 4. <u>Lateral Load Restraint Test</u>

With the test block and fasteners supported and loaded in accordance with Figure C-5, a load of 29 kips is to be applied in increments of 5 kips up to and including 25 kips, and in increments of 1 kip from 25 kips up to and including 29 kips. Dial gage readings for both gage widening and rail translation shall be recorded for each increment of load. In addition, lateral movement of the fastener base plate with respect to the test block shall be recorded.

Note: The AREA Specifications for Concrete Ties and Fasteners limits the allowable gage widening to 1/4-in. This test was designed to accommodate concrete ties and fasteners to be used in heavy duty main line service in curved track as well as tangent track.

The Montana test segment is on tangent track. In addition, traffic on the Alaska Railroad cannot be considered heavy duty in terms of speed or density. The requirements of this test are therefore less severe than those specified in the AREA Specifications.

For the purpose of this test, with the fastener supported on shims, a lateral movement of the fastener base plate of 1/4-in. with respect to the test block shall constitute failure to pass the test.

<u>Shipping</u>

Prestressed concrete ties shall be banded and/or otherwise protected from damage in a manner consistant with the mode of shipment selected. Threaded inserts shall be protected against entry of foreign materials.

Two prestressed concrete ties shall be shipped prepaid to the Portland Cement Association, Skokie, Illinois, for additional testing.

One hundred ninety-eight prestressed concrete ties and the same number of tie sets of fastening hardware are to be shipped to point of destination on the Alaska Railroad as designated on the Purchase Order.

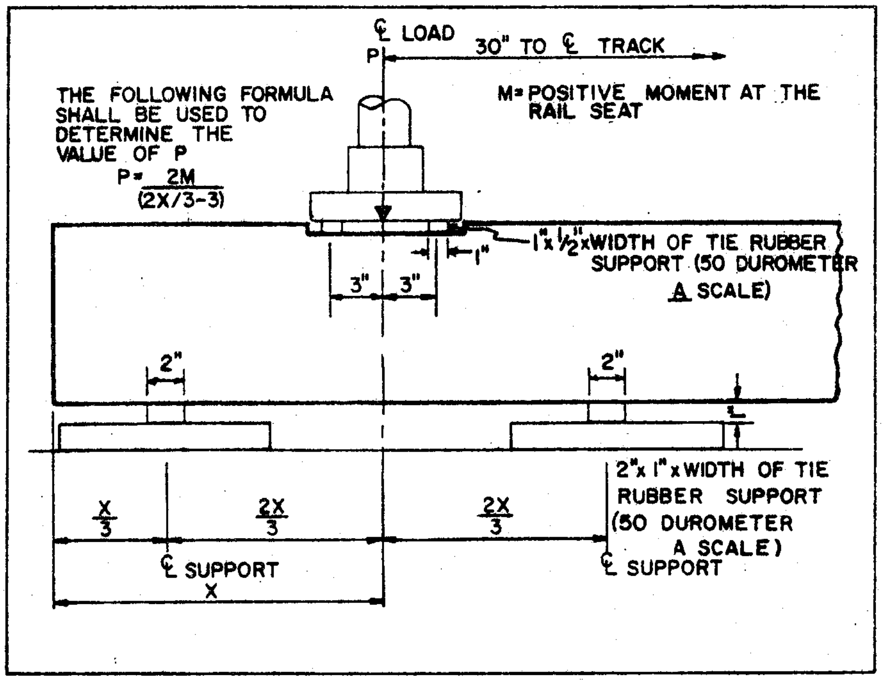


FIG. C-1 - RAIL SEAT POSITIVE MOMENT TEST

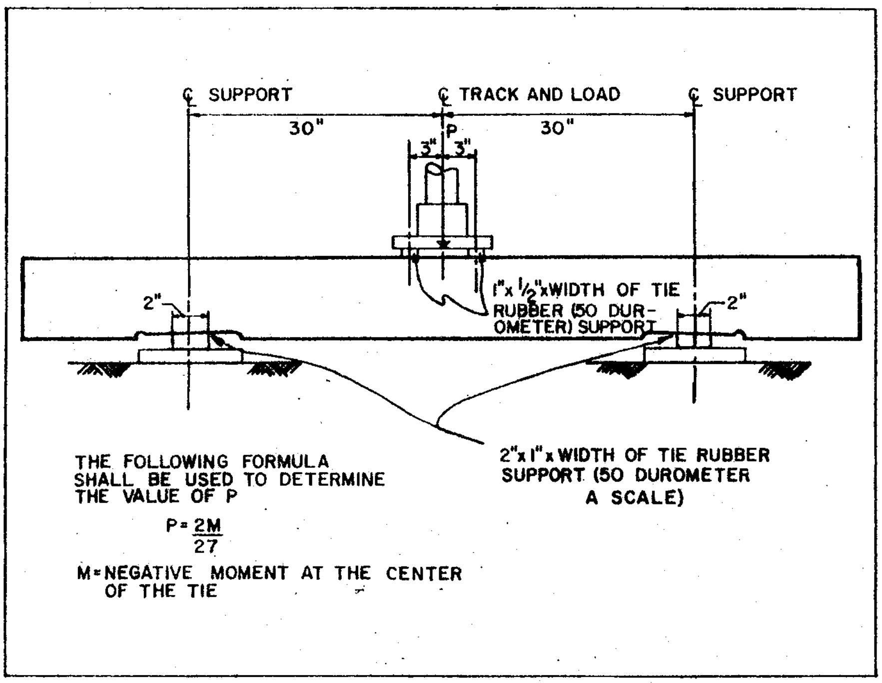


FIG. C-2 - CENTER NEGATIVE MOMENT TEST

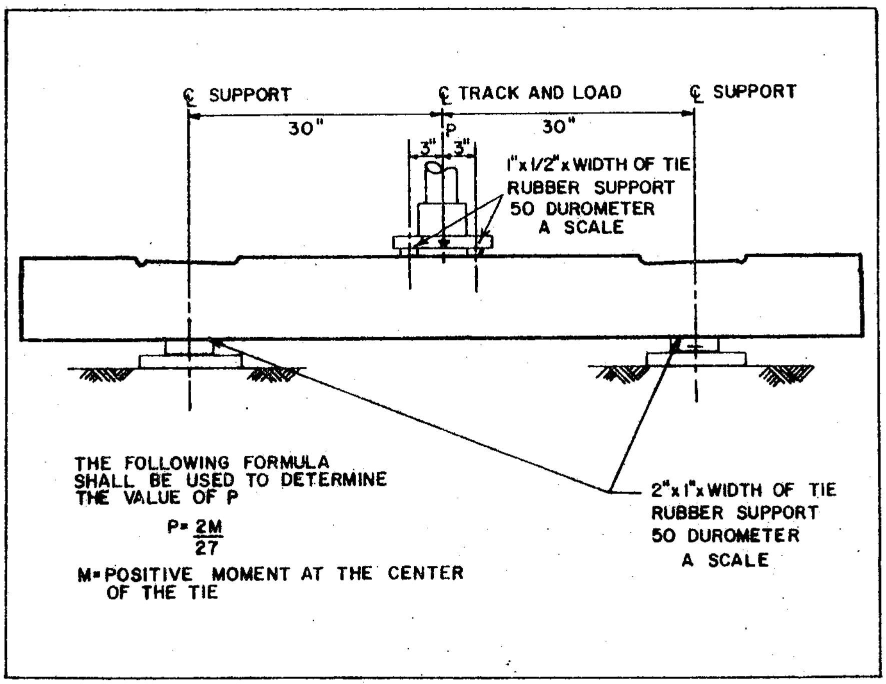


FIG. C-3 - CENTER POSITIVE MOMENT TEST

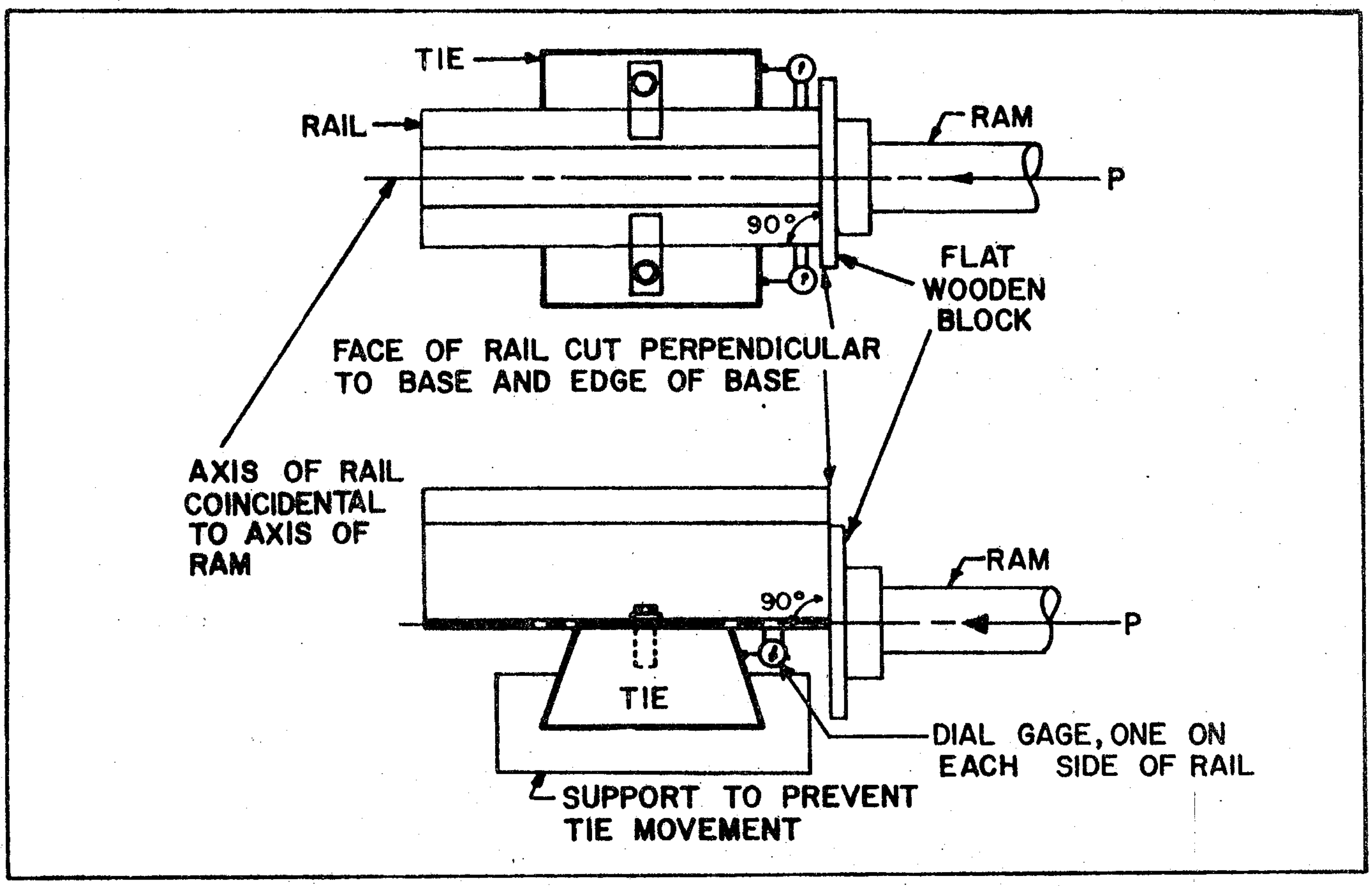


FIG. C-4 - FASTENING LONGITUDINAL RESTRAINT TEST

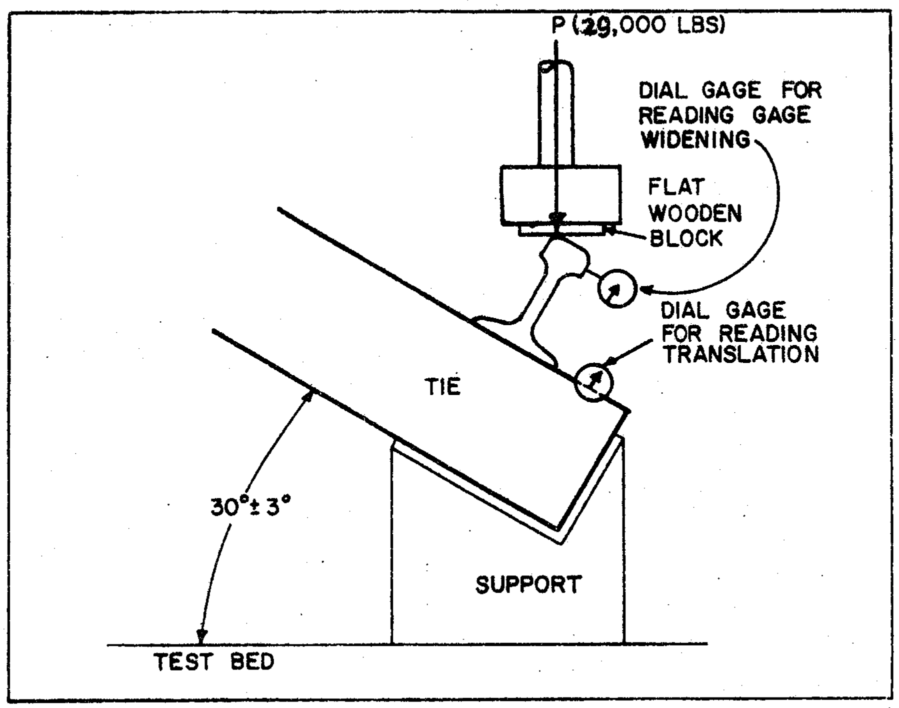


FIG. C-5 - LATERAL LOAD RESTRAINT TEST

APPENDIX D

TABLE D-1
MONTANA CONCRETE TIE SECTION
December 10, 1973 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
36	-	0.25
37	0.25	0.50
38	0.25	0.75
39	0.25	0.75
40	0.50	1.00
41	0.75	1.25
42	1.00	1.50
43	1.00	1.50
44	1.00	1.50
45	1.25	1.50
46	1.00	1.50
47	1.00	1.50
48	1.00	1.50
49	0.75	1.25
50	0.75	1.00
51	0.50	0.75
52	0.25	0.50
53	0.25	0.25
54	-	0.25
55	-	0.25

For SI units 1 in. = 25.4 mm

TABLE D-2
MONTANA CONCRETE TIE SECTION
January 29, 1974 - Track Adjustment

	Shim Adjustment, in.				
Tie No.	West Rail	East Rail	Tie No.	West Rail	East Rail
9	0.25	-	142	0.25	0.25
10	0.25	-	143	0.50	0.50
11	0.25	-	144	0.75	0.75
12	0.25	-	145	1.00	0.75
13	0.50	-	146	1.00	1.00
14	0.50	-	147	1.00	1.25
15	0.50	-	148	1.25	1.25
16	0.50	0.25	149	0.75	1.00
17	0.75	0.25	150	0.75	1.00
18	0.75	0.25	151	0.75	1.00
19	0.75	0.25	152	0.75	1.00
20	0.75	0.50	153	0.75	1.00
21	0.75	0.50	154	0.75	1.00
22	0.75	0.50	155	0.75	0.75
23	0.75	0.50	156	0.75	0.75
24	0.75	0.50	157	0.75	0.75
25	0.75	0.50	158	0.75	0.75
26	0.75	0.50	159	0.75	0.50
27	0.75	0.50	160	0.75	0.75
28	0.75	0.50	161	0.75	0.75
29	0.50	0.50	162	0.50	0.50
30	0.50	0.50	163	0.50	0.50
31	0.25	0.25	164	0.25	0.50
32	0.25	0.50			
33	-	0.50			
34	-	0.25			
35	-	0.25			

TABLE D-3
MONTANA CONCRETE TIE SECTION
February 26, 1974 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
40	0.25	0.25
41	0.50	0.25
42	0.50	0.50
43	0.75	0.50
44	0.75	0.75
45	0.75	0.75
46	0.75	0.75
47	0.75	0.75
48	0.75	0.75
49	0.50	0.75
50	0.50	0.50
51	0.25	0.50
52	0.25	0.50
53	0.25	0.25

TABLE D-4
MONTANA CONCRETE TIE SECTION
April 18, 1974 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
105	-	0.25
106	-	0.25
107	-	0.25
108	-	0.50
109	-	0.50
110	0.25	0.50
111	0.25	0.50
112	0.25	0.50
113	0.25	0.50
114	0.50	0.75
115	0.50	0.75
116	0.75	0.75
117	0.75	1.00
118	0.75	1.00
119	0.75	1.00
120	0.75	1.00
121	1.00	1.00
122	1.00	1.00
123	1.00	1.00
124	1.00	1.00
125	1.00	1.00
126	1.00	0.75
127	0.75	0.75
128	0.75	0.75
129	0.50	0.50
130	0.25	0.50
131	0.25	0.50
132	0.25	0.25
133	0.25	0.25
134	-	0.25
135	-	0.25
136	-	0.25

TABLE D-5
MONTANA CONCRETE TIE SECTION
June 28, 1974 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
142	-0.25*	-0.25
143	-0.50	-0.50
144	-0.75	-0.75
145	-1.00	-0.75
146	-1.00	-1.00
147	-1.00	-1.25
148	-1.25	-1.25
149	-0.75	-1.00
150	-0.75	-1.00
151	-0.75	-1.00
152	-0.75	-1.00
153	-0.75	-1.00
154	-0.75	-1.00
155	-0.75	-0.75
156	-0.75	-0.75
157	-0.75	-0.75
158	-0.75	-0.75
159	-0.75	-0.50
160	-0.75	-0.75
161	-0.75	-0.75
162	-0.50	-0.50
163	-0.50	-0.50
164	-0.25	-0.50

*Minus sign indicates shims removed

TABLE D-6
MONTANA CONCRETE TIE SECTION
August 27, 1974 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
105	-	-0.25*
106	-	-0.25
107	-	-0.25
108	-	-0.50
109	-	-0.50
110	-0.25	-0.50
111	-0.25	-0.50
112	-0.25	-0.50
113	-0.25	-0.50
114	-0.50	-0.75
115	-0.50	-0.75
116	-0.75	-0.75
117	-0.75	-1.00
118	-0.75	-1.00
119	-0.75	-1.00
120	-0.75	-1.00
121	-1.00	-1.00
122	-1.00	-1.00
123	-1.00	-1.00
124	-1.00	-1.00
125	-1.00	-1.00
126	-1.00	-0.75
127	-0.75	-0.75
128	-0.75	-0.75
129	-0.50	-0.50
130	-0.25	-0.50
131	-0.25	-0.50
132	-0.25	-0.25
133	-0.25	-0.25
134	-	-0.25
135	-	-0.25
136	-	-0.25

*Minus sign indicates shims removed

TABLE D-7
MONTANA CONCRETE TIE SECTION
August 28, 1974 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
9	-0.25*	-
10	-0.25	-
11	-0.25	-
12	-0.25	-
13	-0.50	-
14	-0.50	-
15	-0.50	-
16	-0.50	-0.25
17	-0.75	-0.25
18	-0.75	-0.25
19	-0.75	-0.25
20	-0.75	-0.50
21	-0.75	-0.50
22	-0.75	-0.50
23	-0.75	-0.50
24	-0.75	-0.50
25	-0.75	-0.50
26	-0.75	-0.50
27	-0.75	-0.50
28	-0.75	-0.50
29	-0.50	-0.50
30	-0.50	-0.50
31	-0.25	-0.25
32	-0.25	-0.50
33	-	-0.50
34	-	-0.25
35	-	-0.25
36	-	-0.25
37	-0.25	-0.50
38	-0.25	-0.75
39	-0.25	-0.75
40	-0.75	-1.25
41	-1.25	-1.50
42	-1.50	-2.00
43	-1.75	-2.00
44	-1.75	-2.25
45	-2.00	-2.25
46	-1.75	-2.25
47	-1.75	-2.25
48	-1.75	-2.25
49	-1.25	-2.00
50	-1.25	-1.50
51	-0.75	-1.25
52	-0.50	-1.00
53	-0.50	-0.50
54	-	-0.25
55	-	-0.25

*Minus sign indicates shims removed

TABLE D-8
MONTANA CONCRETE TIE SECTION
January 16, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
37	0.25	0.25
38	0.25	0.50
39	0.50	0.75
40	0.75	0.75
41	1.00	1.00
42	1.00	1.00
43	1.00	1.00
44	1.00	1.00
45	1.00	1.00
46	1.00	1.00
47	1.00	1.00
48	1.00	1.00
49	1.00	1.00
50	0.75	0.75
51	0.75	0.50
52	0.50	0.25
53	0.25	–
54	0.25	–

TABLE D-9
MONTANA CONCRETE TIE SECTION
February 6, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in.	
	West Rail	East Rail
140	0.25	0.25
141	0.50	0.50
142	0.75	0.75
143	0.75	0.75
144	0.75	0.75
145	0.75	0.75
146	0.75	0.75
147	0.75	0.75
148	0.75	0.75
149	0.75	0.75
150	0.75	0.75
151	0.75	0.75
152	0.25	0.25

TABLE D-10
MONTANA CONCRETE TIE SECTION
April 8, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
38	-	0.25
39	0.25	0.50
40	0.25	0.50
41	0.50	0.75
42	0.50	0.75
43	0.75	0.75
44	0.75	0.75
45	0.75	0.75
46	0.50	0.75
47	0.50	0.50
48	0.50	0.25
49	0.25	0.25
50	0.25	-

TABLE D-11
MONTANA CONCRETE TIE SECTION
April 22, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	East Rail
114	0.25	-
115	0.25	0.25
116	0.50	0.50
117	0.75	0.75
118	1.00	0.75
119	1.00	1.00
120	1.00	1.00
121	1.00	1.25
122	1.25	1.25
123	1.25	1.25
124	1.25	1.25
125	1.25	1.00
126	1.25	1.00
127	1.00	1.00
128	1.00	1.00
129	1.00	1.00
130	1.00	1.00
131	0.75	1.00
132	0.75	0.50
133	0.50	0.50
134	0.25	0.25

TABLE D-12
MONTANA CONCRETE TIE SECTION
May 22, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
38	-0.25	-0.25
39	-0.50	-0.50
40	-0.75	-1.00
41	-1.25	-1.25
42	-1.50	-1.50
43	-1.75	-1.75
44	-1.75	-1.75
45	-1.75	-1.75
46	-1.50	-1.75
47	-1.50	-2.00
48	-1.50	-1.50
49	-1.25	-1.25
50	-1.00	-1.00
51	-0.75	-0.75
52	-0.50	-0.50
53	-0.25	-0.25
54	-0.25	-0.25

TABLE D-13
MONTANA CONCRETE TIE SECTION
June 12, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	East Rail	Tie No.	West Rail	East Rail
114	-	-0.25	132	-0.50	-0.75
115	-0.25	-0.25	133	-0.50	-0.50
116	-0.50	-0.50	134	-0.25	-0.25
117	-0.75	-0.75	135	-0.25	-
118	-0.75	-1.00	142	-	-0.25
119	-1.00	-1.00	143	-0.25	-0.25
120	-1.00	-1.00	144	-0.50	-0.50
121	-1.25	-1.00	145	-0.75	-0.75
122	-1.25	-1.25	146	-0.75	-0.75
123	-1.25	-1.25	147	-0.75	-0.75
124	-1.25	-1.25	148	-0.75	-0.75
125	-1.00	-1.25	149	-0.75	-0.75
126	-1.00	-1.25	150	-0.75	-0.75
127	-1.00	-1.00	151	-0.75	-0.75
128	-1.00	-1.00	152	-0.75	-0.75
129	-1.00	-1.00	153	-0.75	-0.75
130	-1.00	-1.00	154	-0.50	-0.50
131	-0.75	-0.75	155	-0.25	-0.25
			156	-0.25	-

TABLE D-14
MONTANA CONCRETE TIE SECTION
November 28, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
29	0.25	-
30	0.25	-
31	0.50	-
32	0.75	0.25
33	0.75	0.25
34	0.75	0.50
35	1.00	0.50
36	1.00	0.50
37	1.00	0.75
38	1.00	1.00
39	1.00	0.75
40	1.00	0.75
41	1.00	0.75
42	0.75	0.75
43	0.50	1.00
44	0.50	0.75
45	0.25	0.75
46	-	0.50
47	-	0.50
48	-	0.50

TABLE D-15
MONTANA CONCRETE TIE SECTION
March 16, 1976 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
31	0.50	-
32	0.75	-
33	0.75	-
34	1.00	0.25
35	1.00	0.50
36	1.00	0.75
37	1.00	0.75
38	1.00	1.00
39	1.00	1.25
40	1.00	1.50
41	1.00	1.50
42	0.75	1.50
43	0.50	1.50
44	0.25	1.00
45	-	0.75

TABLE D-16
MONTANA CONCRETE TIE SECTION
May 24, 1976 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	Shim Adjustment, in. East Rail
29	-0.25*	-
30	-0.25	-
31	-1.00	-
32	-1.50	-0.25
33	-1.50	-0.25
34	-1.75	-0.75
35	-2.00	-1.00
36	-2.00	-1.25
37	-2.00	-1.50
38	-2.00	-2.00
39	-2.00	-2.00
40	-2.00	-2.25
41	-2.00	-2.25
42	-1.50	-2.25
43	-1.00	-2.50
44	-0.75	-1.75
45	-0.25	-1.50
46	-	-0.50
47	-	-0.50
48	-	-0.50

*Minus sign indicates shims removed

TABLE D-17
MONTANA WOOD TIE CONTROL SECTION
April 16, 1974 - Track Adjustment

	Shim Adjustment, in.				
Tie No.	West Rail	East Rail	Tie No.	West Rail	East Rail
1	-	0.25	28	-0.75*	-0.75*
2	-	0.25	29	-0.75*	-0.75*
3	0.25	0.25	30	-0.75*	-0.75*
4	0.25	0.50	31	-0.75*	-0.75*
5	0.25	0.50	32	-0.75*	-0.75*
6	0.25	0.25	33	-0.75*	-0.75*
7	0.25	0.25	34	-0.75*	-0.75*
8	0.25	0.25	35	-0.75*	-0.75*
9	0.25	0.25	36	-0.75*	-0.75*
10	0.25	0.25	37	-0.75*	-0.75*
11	0.25	0.25	38	-0.75*	-0.75*
12	0.25	0.25	39	-0.75*	-0.75*
13	0.50	0.25	40	-0.75*	-0.75*
14	0.50	0.25	41	-0.75*	-0.75*
15	0.50	0.25	42	-0.75*	-0.75*
16	0.50	0.25	43	-0.75*	-0.75*
17	0.50	0.25	44	-0.75*	-0.75*
18	0.50	0.25	45	-0.75*	-0.75*
19	0.25	-	46	-0.75*	-0.75*
20	0.25	-	47	-0.75*	-0.75*
21	0.25	-	48	-0.75*	-0.75*
22	-0.75*	-0.75*	49	0.25	-0.75*
	-0.25*	-0.25*			
23	-0.25*	-0.50*	50	0.25	-0.75*
24	-0.50*	-0.75*	51	0.25	-0.75*
25	-0.50*	-0.75*	52	0.25	0.25
26	-0.75*	-0.75*	53	0.25	0.25
27	-0.75*	-0.75*	54	0.50	0.50

TABLE D-18
MONTANA WOOD TIE CONTROL SECTION
February 11, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	East Rail
1	0.25	0.25
2	0.25	0.25
3	0.25	0.50
4	0.50	0.50
5	0.75	0.75
6	0.75	0.75
7	1.00	0.75
8	0.75	0.75
9	0.75	0.75
10	0.75	0.75
11	1.00	1.00
12	1.00	1.00
13	1.00	1.00
14	1.00	1.00
15	1.25	1.25
16	1.25	1.25
17	1.50	1.25
18	1.50	1.25
19	1.50	1.50
20	1.75	1.50
21	1.75	1.50
22	1.75	1.50
23	1.75	1.50
24	1.75	1.50
25	1.50	1.50
26	1.50	1.50
27	1.50	1.50
28	1.50	1.50
29	1.50	1.50
30	1.25	1.25
31	1.25	1.25
32	1.25	1.25
33	1.25	1.00
34	1.00	1.00
35	1.00	0.50
36	0.75	0.50
37	0.50	0.50
38	0.25	–
39	0.25	–

TABLE D-19
MONTANA WOOD TIE CONTROL SECTION
May 20, 1975 - Track Adjustment

Tie No.	Shim Adjustment, in. West Rail	East Rail	Tie No.	West Rail	East Rail
69	0.25	-	90	2.00	1.50
70	0.25	-	91	2.00	1.50
71	0.50	-	92	2.00	1.50
72	0.75	0.25	93	2.00	1.75
73	0.75	0.25	94	2.00	1.75
74	0.75	0.25	95	1.75	1.75
75	0.75	0.50	96	1.75	1.75
76	0.75	0.50	97	1.75	1.75
77	1.00	0.50	98	1.75	1.50
78	1.00	0.75	99	1.50	1.50
79	1.00	0.75	100	1.25	1.50
80	1.25	1.00	101	1.25	1.00
81	1.25	1.00	102	1.25	1.00
82	1.25	1.25	103	1.00	0.75
83	1.25	1.25	104	0.75	0.75
84	1.50	1.50	105	0.50	0.50
85	1.50	1.25	106	0.50	0.25
86	1.75	1.25	107	0.25	0.25
87	2.00	1.25			
88	2.00	1.25			
89	2.00	1.25			

TABLE D-20
MONTANA TEMPERATURES
7:00 a.m. Temperature, F

Week Ending	SUNSHINE WEATHER STATION Average	Max. Low	WILLOW WEATHER STATION Average	Max. Low
10/07/73			32	26
10/14/73	22	5	25	18
10/21/73	22	5	21	8
10/28/73	20	5	19	14
11/04/73	17	5	-4	-16
11/11/73	7	-5	1	-15
11/18/73	-16	-22	-15	-31
11/25/73	13	5	9	-24
12/02/73	9	-20	-23	-30
12/09/73	-22	-28	-3	-24
12/16/73	15	-10	-11	-22
12/23/73	25	23	5	-18
12/30/73	20	8	18	18
1/06/74	-3	-10	-14	-41
1/13/74	3	-10	-10	-36
1/20/74	-7	-22	-23	-36
1/27/74	-12	-34	-20	-34
2/03/74	-8	-22	-15	-30
2/10/74	22	15	20	20
2/17/74	-4	-24	-6	-14
2/24/74	-16	-25	-9	-10
3/03/74	17	10	0	-30
3/10/74	-15	-32	-21	-32
3/17/74	0	-20	3	-24
3/24/74	29	25	29	22
3/31/74	23	15	25	20
4/07/74	22	18	35	30
4/14/74	31	30	33	28
4/21/74	32	28	40	32
4/28/74	32	30	38	34
5/05/74	39	35	50	48
5/12/74	40	40	53	44
5/19/74	41	40	48	40
5/26/74	41	40	54	48
6/02/74	45	44	49	44

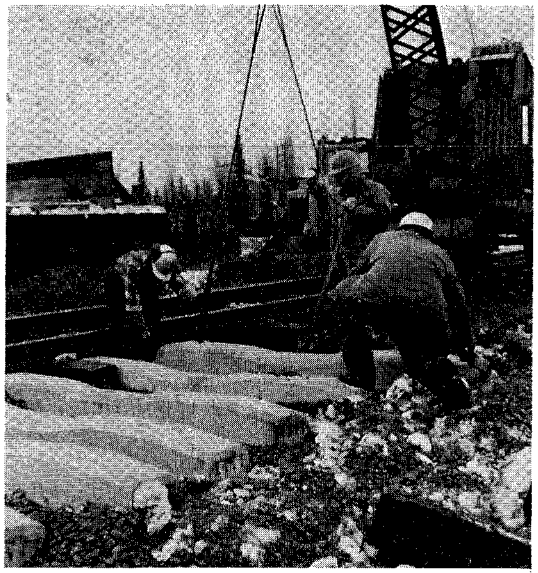
To convert temperature reading to Celsius
$t_C = (t_F - 32)/1.8$

TABLE D-21

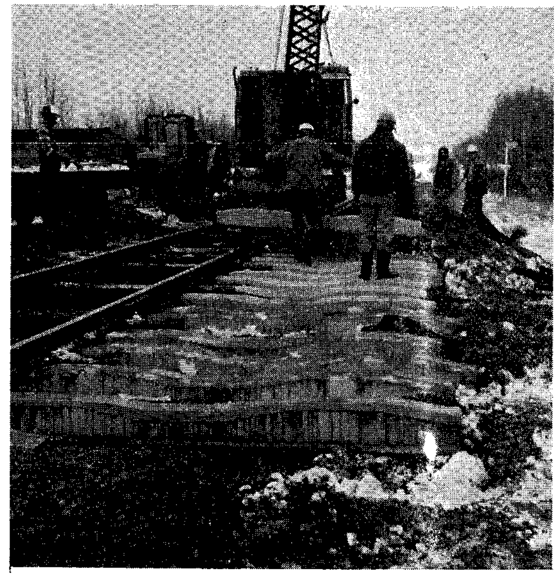
Event Summary Montana

Event		Date
1.	As built Profile	October 24, 1973
2.	Add Shims Test	December 10, 1973
3.	Add Shims Test	January 29 & 30, 1974
4.	Top of Rail Profile	January 31, 1974
5.	Add Shims Test	February 26, 1974
6.	Top of Rail Profile	February 28, 1974
7.	Add Shims Control	April 16, 1974
8.	Add Shims Test	April 18, 1974
9.	Remove Shims Test	June 28, 1974
10.	Visual Inspection	July, 1974
11.	Remove Shims Test	August 27 & 28, 1974
12.	Top of Rail Profile	November 19, 1974
13.	Add Shims Test	January 16, 1975
14.	Add Shims Test	February 6, 1975
15.	Add Shims Control	February 11, 1975
16.	Top of Rail Profile	February 20, 1975
17.	Add Shims Test	April 8, 1975
17a.	Top of Rail Profile	April 18, 1975
18.	Add Shims Test	April 22, 1975
19.	Remove Shims Control	May 20, 1975
20.	Remove Shims Test	May 22, 1975
21.	Remove Shims Test	June 12, 1975
22.	Visual Inspection	July, 1975
23.	Top of Rail Profile	October 6, 1975
24.	Top of Rail Profile	October 15, 1975
25.	Add Shims Test	November 28, 1975
26.	Top of Rail Profile	February 18, 1976
27.	Add Shims Test	March 16, 1976
28.	Remove all Shims	May 24, 1976
29.	Top of Rail Profile & Inspection	July 1976.

APPENDIX E



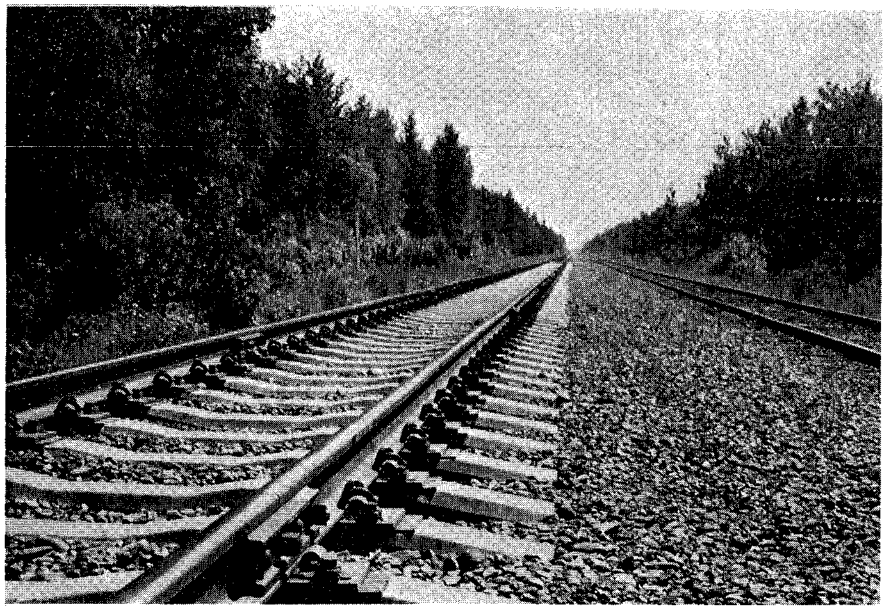
Positioning of Ties
Fig. E-1



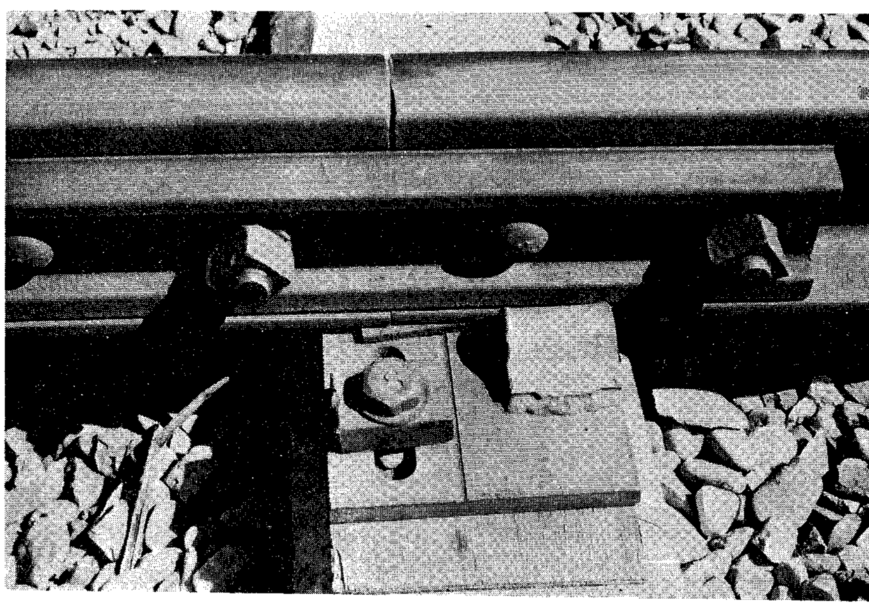
Ties Prior to Installation
Fig. E-2

Installation of Ties
Fig. E-3

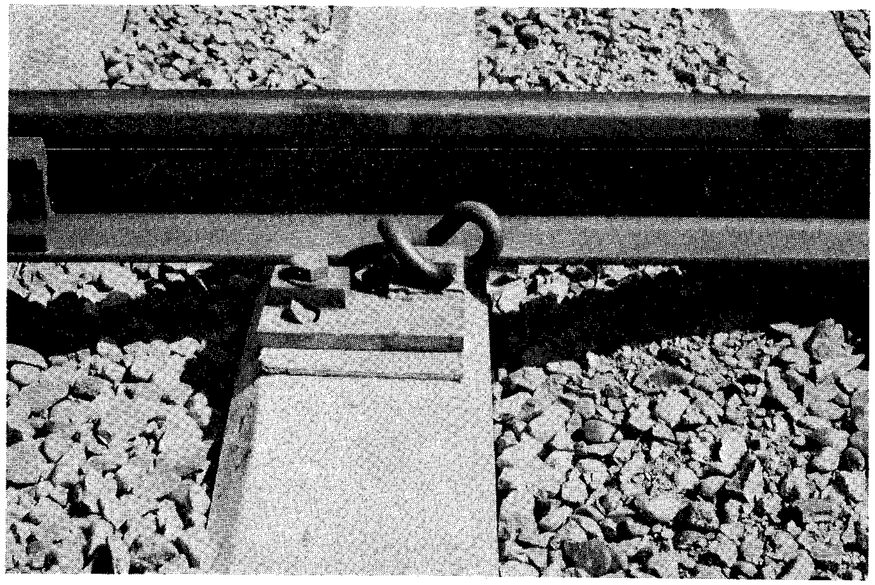
Test Section Following
Installation
Fig. E-4



Test Section - July 1975
Fig. E-5

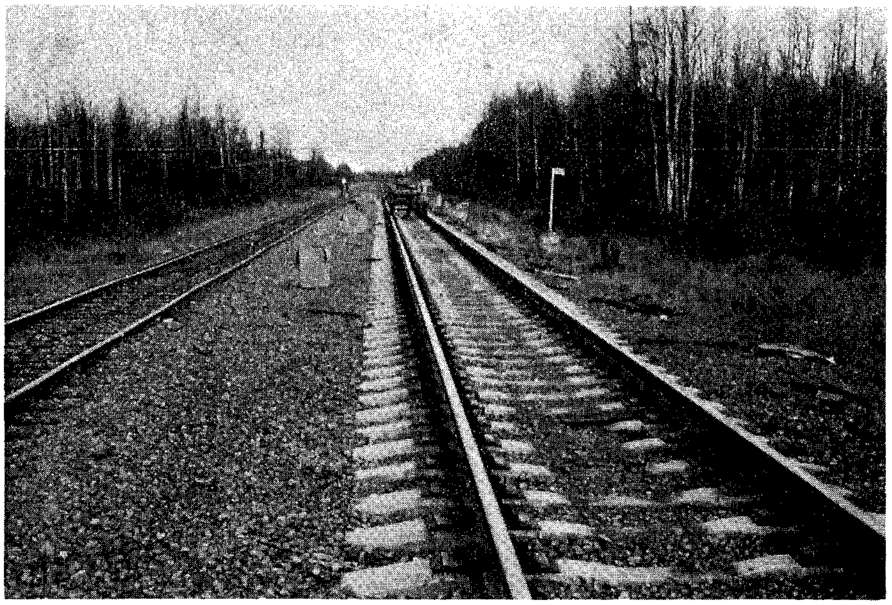


Pad Displacement Beneath Rail Joint
Fig. E-6

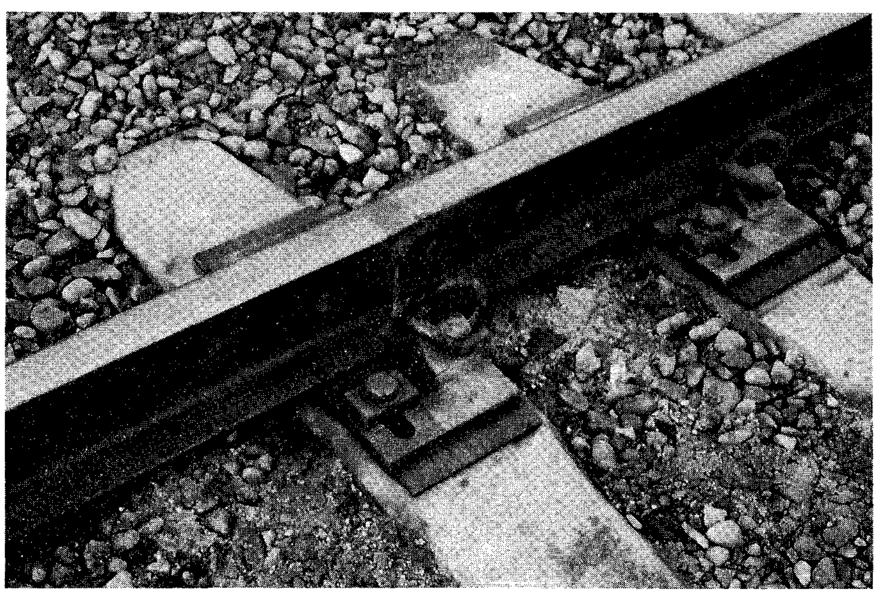


Pad Displacement on Tie Adjacent
to Rail Joint
Fig. E-7

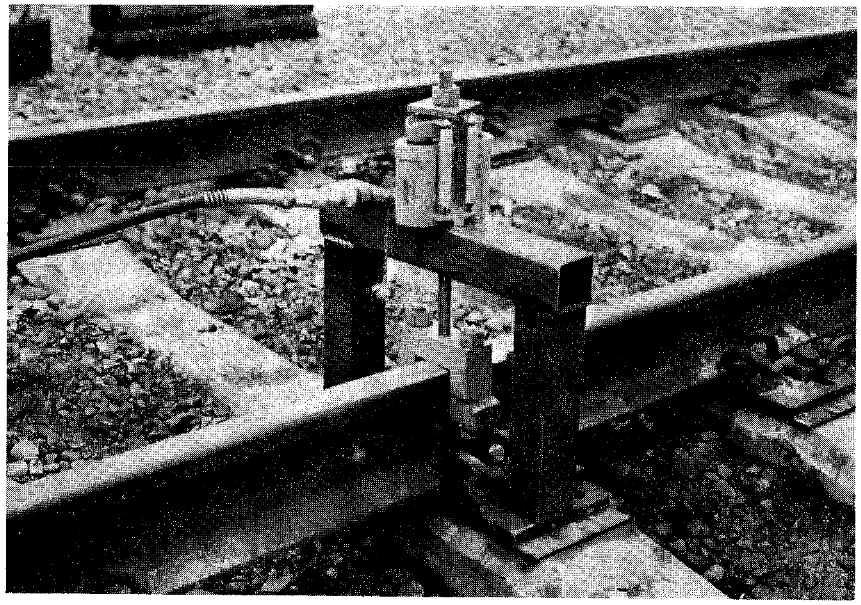
Wood Shims Removed After First Winter
Fig. E-8



Track View Following Welding of Rail
Joints - October 1975
Fig. E-9

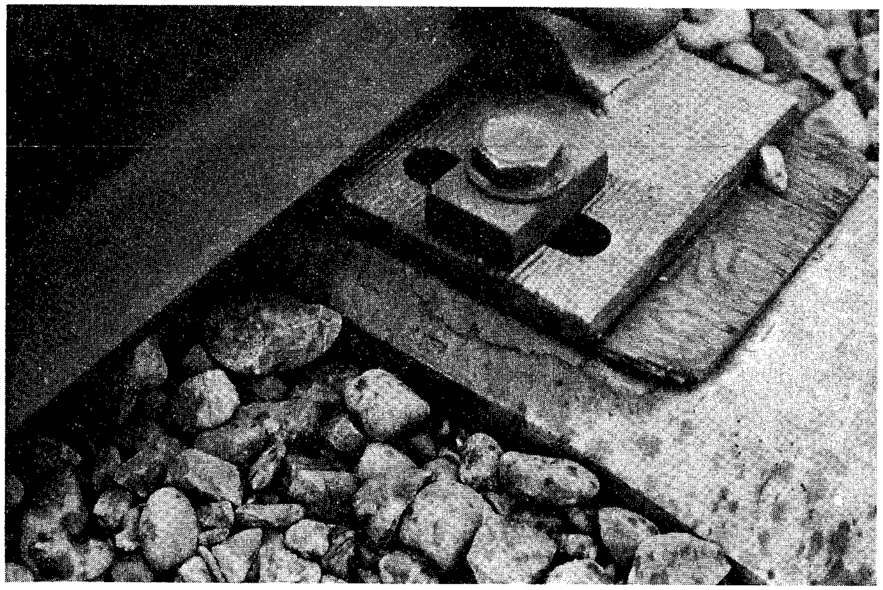


Fastener System at Welded Rail Joint
Fig. E-10

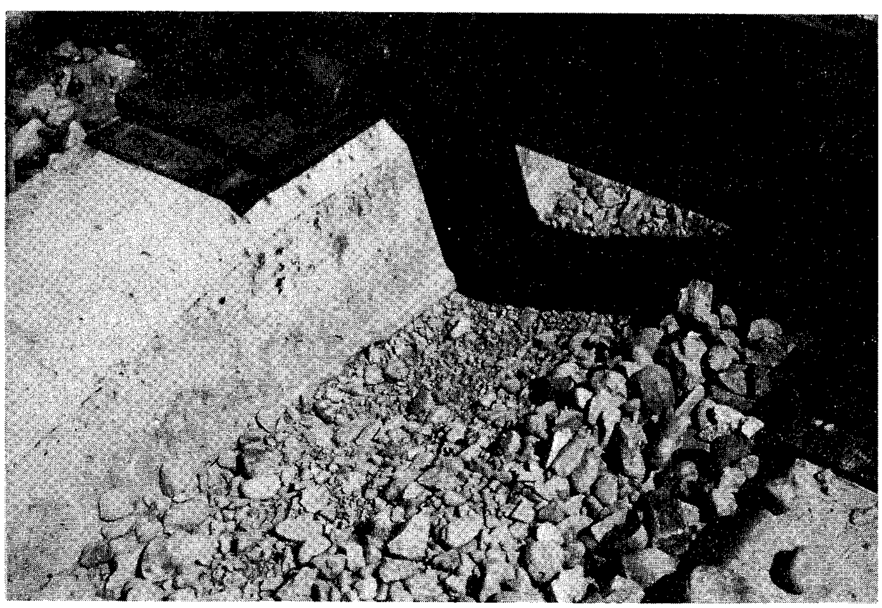


Lifting Rail for Pad Adjustment
Fig. E-11

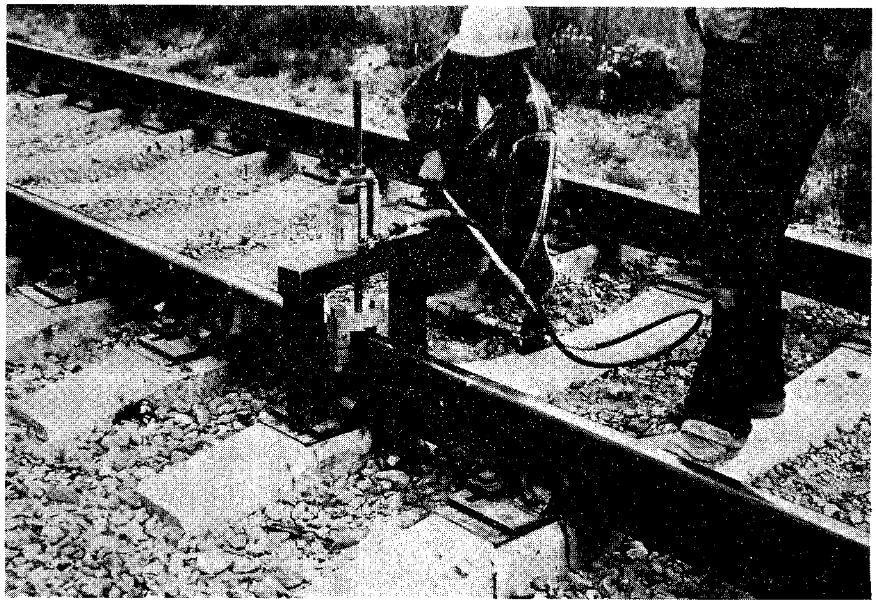
Spalled Concrete Caused by Over
Torqueing Anchor Bolts
Fig. E-12



Damage Caused by Excessive Anchor Bolt Torque
Fig. E-13



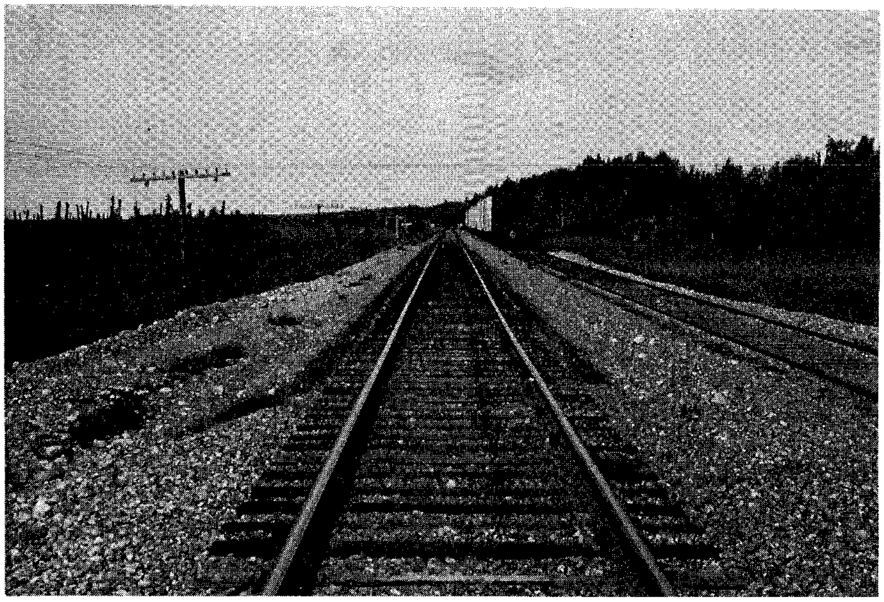
Tie Condition - 1976
Fig. E-14



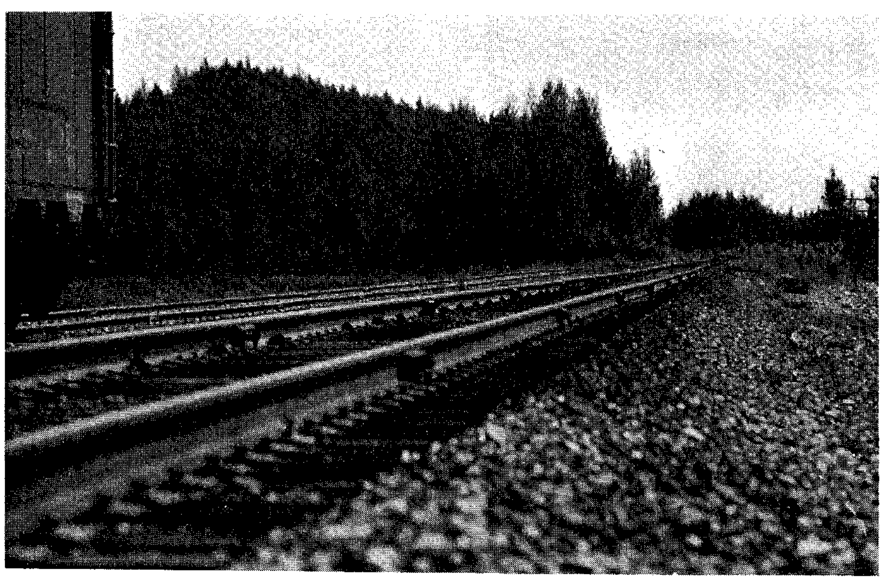
Pad Replacement - July 1976
Fig. E-15

Crushed Rail Seat Pad
Fig. E-16

Appendix F



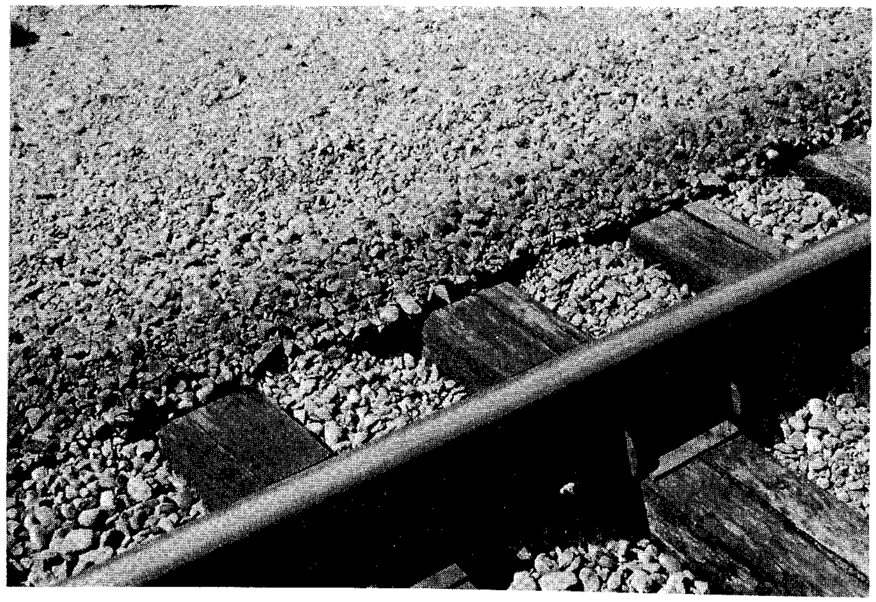
Test Section After Installation
Fig. F-1



Rail Profile of Test Section
Fig. F-2

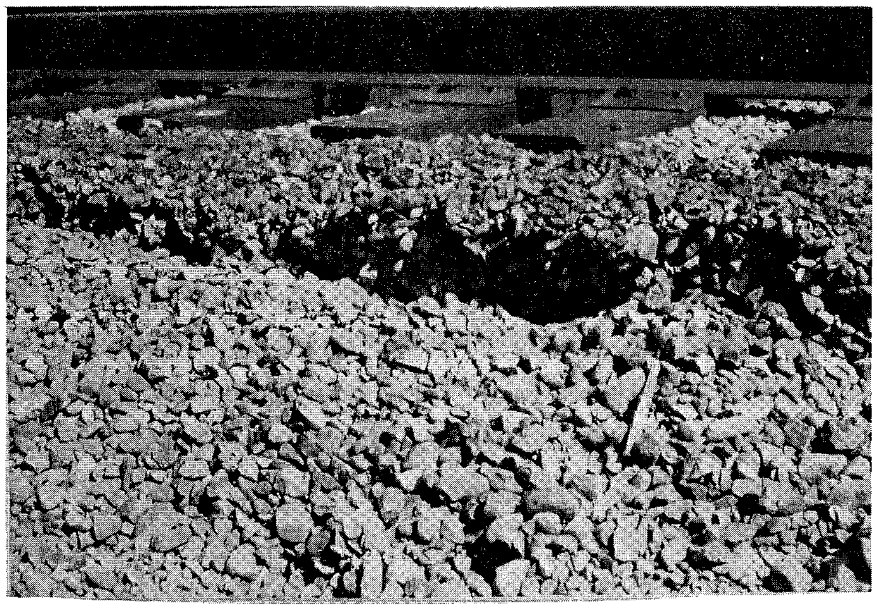
Reference Rod for Rail Profiles
Fig. F-3

Beginning of Ballast Settlement Outside
Stabilized Shoulder - July 1974
Fig. F-4

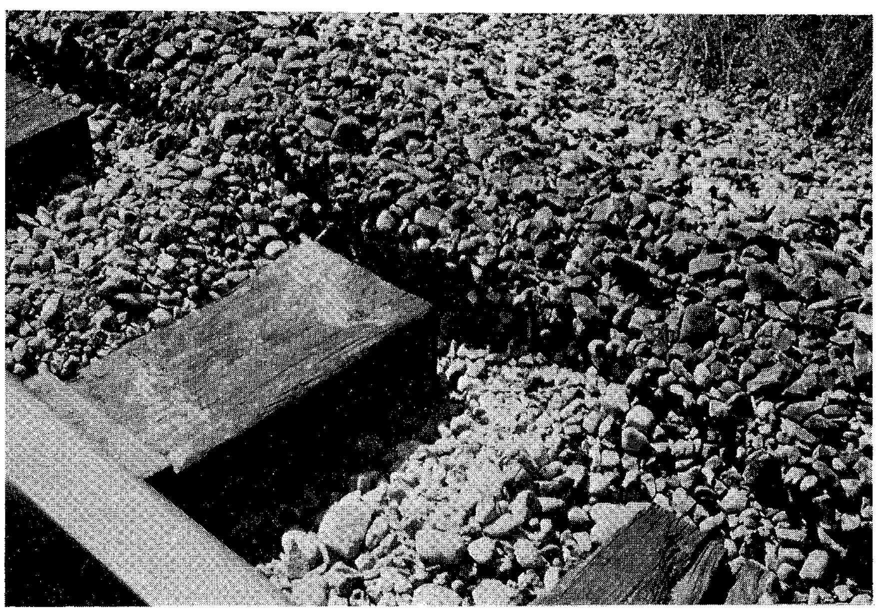


Ballast Migration Along Inside Edge of
Stabilized Shoulder - July 1974
Fig. F-5

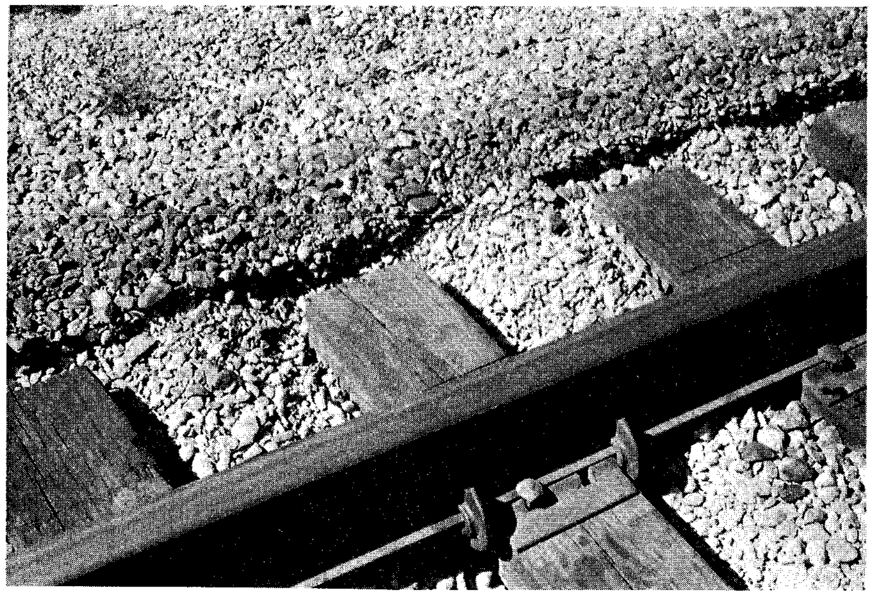
First Sign of Back and Forth Tie Movement
- July 1974
Fig. F-6



Settlement of Ballast Outside of Stabilized Shoulder - July 1975
Fig. F-7



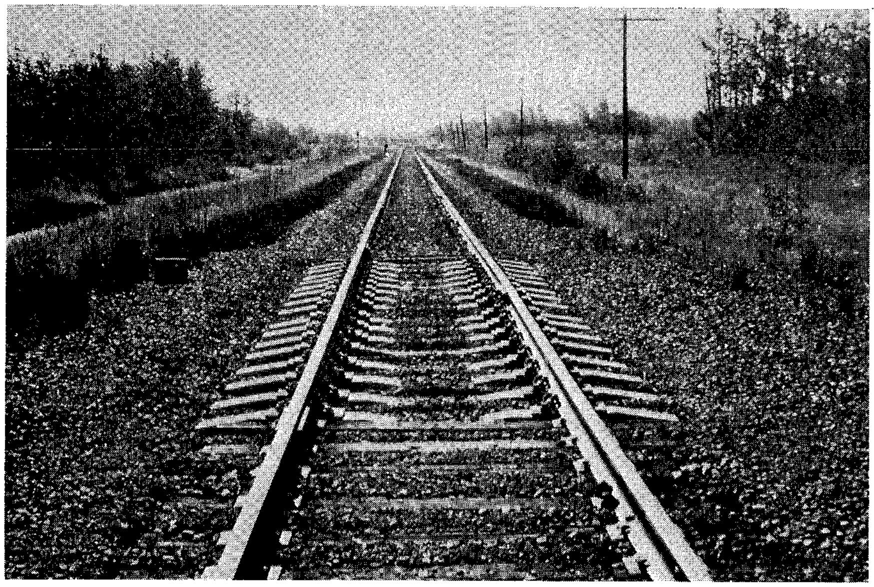
Migration of Ballast Inside of Stabilized Shoulder - July 1975
Fig. F-8



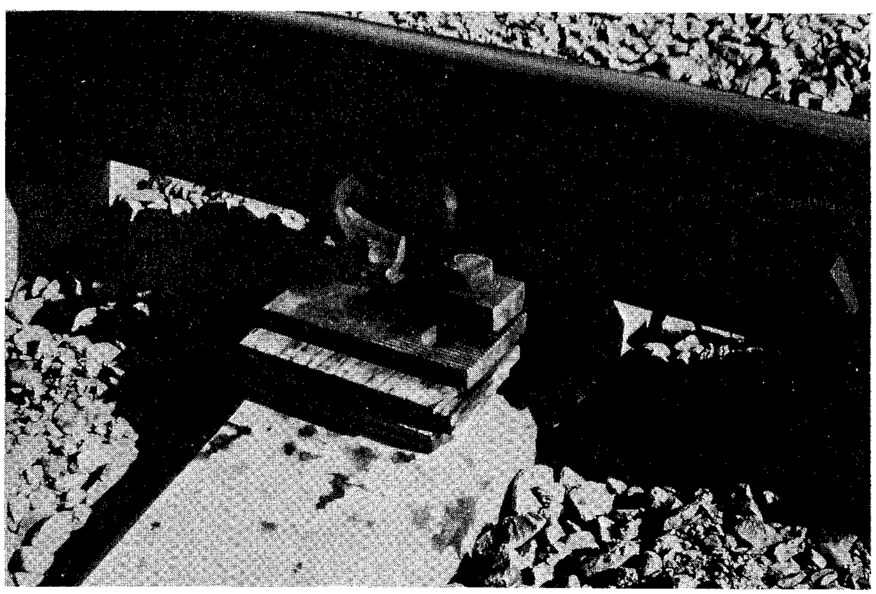
Ballast Movement Inside of Stabilized
Shoulder - July 1975
Fig. F-9

Signs of Back and Forth Tie Movement
- July 1975
Fig. F-10

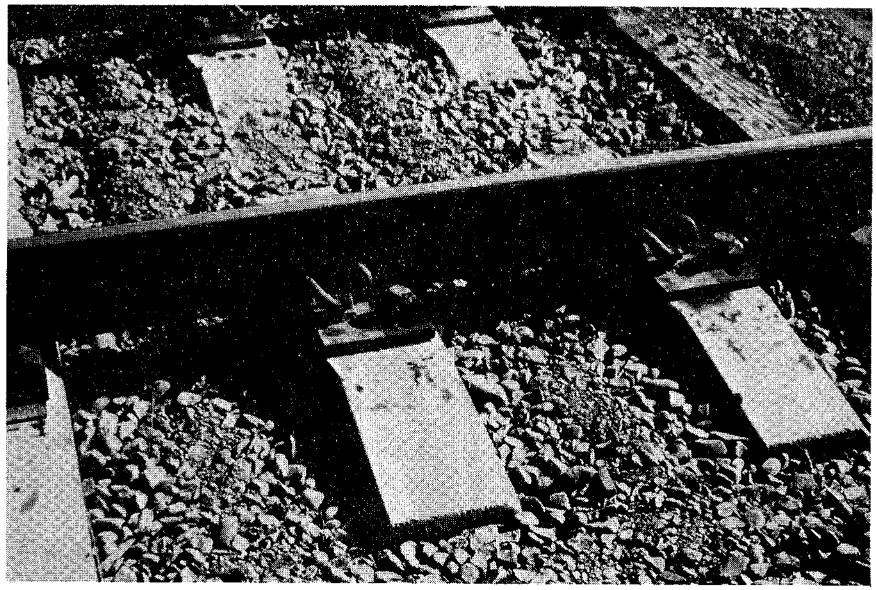
APPENDIX G



Test Section - July 1976
Fig. G-1



$2\frac{1}{4}$ in. Shimming Used During 1975-76 Winter
Fig. G-2



Fastener Without Wood Shims
Fig. G-3

Tie Spacing of 36-in. Used at Rail Joint
Fig. G-4