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CONCRETE CROSSTIES IN THE UNITED STATES

by

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I. Introduction

The research laboratories of the Association of American Railroads began work in 1957 on the design and development of prestressed concrete crossties. Today, nearly one million such ties are in service in the United States. Their field performance has justified continuing research, development, and in-service testing.

To the author's knowledge, no one before has attempted to tell the concrete crosstie story as it has evolved in the United States since 1957. This paper tells that story.

II. History and Development

The first recorded use of concrete crossties in the United States was in 1893 when 200 were installed by the Reading Company in Germantown, Pa. Between 1893 and 1930, some 150 types of conventionally reinforced concrete crossties were designed and patented in this country. More than 60 different experimental installations of concrete crossties were made by about as many railroads.

Most of the ties performed unsatisfactorily because of improper design or inadequate rail fastenings. Others were removed after several years of service because design features prevented adaptation to other necessary changes in track construction. In addition, they were expensive in comparison with readily available timber crossties.

From 1930 to 1957 there was little if any concrete crosstie activity in this country. This was because of an adequate supply of suitable timber and improved methods of pressure treatment with preservatives to extend timber crosstie service life. Thus, during the years when European countries were developing and installing concrete crossties in their railroads, nothing along this line was being done in the United States. When work on concrete crossties began again in this country, in 1957, several European countries had already developed satisfactory concrete crossties designed specifically for prevailing loads and conditions.

Work of the AAR

The vast majority of work on concrete crossties in the United States was done by, or in cooperation with, the Association of American Railroads (AAR). In 1957, the research staff of AAR decided that even though the supply of timber ties was ample for the immediate future, some prestressed concrete ties should be designed, manufactured, tested and installed on several railroads to secure in-service performance records over a period of years in the event our timber supply or economic conditions changed. Accordingly, the AAR began a laboratory investigation of the static and repeated load strength of prestressed concrete crossties.

The first ties were designed on the assumption that prestressed concrete ties would replace timber ties at the same spacing. Three tie designs--called A, B, and C--were developed. These designs used the strength of an oak tie as a general criterion. Since a moment of about 400,000 in.-lb. is required to break a center-bound oak tie, this was used as one design requirement. Field measurements had shown that under service loading of end-bound ties, the bending moment under the rails is about 100,000 in.-lb. for a 20-in. tie spacing. This was used as the other design requirement.

Tie design A was rectangular and used both of the above criteria. Ties B and C used only the end-bound 100,000 in.-lb. criterion because the center-bound condition was considered to be relieved by the wedge shape of the center section.

Thirty-six ties of designs A, B, and C were tested at the AAR laboratories. Results of the static and repeated load tests were satisfactory and the wedge-shaped center section was found to be desirable. However, further consideration of the various factors involved indicated a change in spacing would be desirable. It was found that a tie 12 in. wide could be spaced at 30-in. centers with only a 10 percent increase in rail stresses and with no increase in the bearing pressure of the tie on the ballast. Consequently, designs D and E were developed early in 1959, using a tie width of 12 in. at the bottom and an increase in the bending moment under the rail.

Results of static and repeated load tests were satisfactory. Tests conducted by AAR to determine if the wedge-shaped tie would displace the ballast and prevent a center-bound condition indicated that the wedge shape reduced center-binding but does not completely eliminate it. These tests also determined that the Type E tie spaced at 30-in. centers should be suitable for mainline track.

Simultaneous with development of the Type E tie, the AAR did considerable work on rail fastening systems. A machine was designed which subjects an end section of a concrete crosstie--complete with rail, fastening system, and tie pad--to repetitive vertical loads combined with lateral loads alternately inward and outward of the rail. To simulate service conditions, sand and water could be introduced during the test. The loads applied were representative of field loads on curves as determined by field tests, the vertical load being 20,000 lb. and the lateral loads 3,750 lb. applied against the field side and 7,500 lb. applied against the gauge side of the rail head. Two and one-half million cycles of these loads were applied in determining the acceptability of the rail fastening systems tested.

Two types of fastenings were developed--indirect and direct fixation. The "indirect fixation" fastener has a steel tie plate under the rail and a tie pad under the plate. The rail is held vertically by a clip while shoulders on the tie plate hold the rail transversely. Standard 1:40 rail cant is built into the tie plate, the top of the concrete tie being flat. Bolts, extending down through holes in the clips, tie plate and pad, are screwed into stainless steel threaded inserts embedded in the tie and hold the rail and fastening system securely to the tie. The "direct fixation" fastener eliminates the tie plate, the clips holding the rail both vertically and transversely. Rail cant is provided by a 1:40 slope of the concrete surface at the rail seat area.

The first installation of AAR Type E crossties was made on the Atlantic Coast Line RR (now Seaboard Coast Line) near Four Oaks, N. C., in March 1960, using 500 indirect fixation ties. This was followed shortly by 600 AAR Type E indirect fixation ties on an instrumented test installation on the Seaboard Airline RR (now Seaboard Coast Line) near Tampa, Fla. Both installations were at 30-in. spacing in 79 mph territory. They annually carry 10 million and 17 million tons of traffic respectively.

Other Type E ties installed for test and evaluation by AAR include 500 direct fixation ties on the Atlantic Coast Line in July 1961, 800 indirect-fixation and five direct-fixation ties on the Canadian National RR in October 1961, and 1,056 direct-fixation ties on the St. Louis-San Francisco Railway in October 1962. All installations were made in high-speed, high-tonnage territory.

During this period the AAR Type E tie was modified several times to improve its in-service performance. The vast majority of all concrete crossties in service in the United States are of a modified AAR direct-fixation Type E design and are commercially designated the MR2 tie. All concrete ties placed in service before 1965 were installed as short test sections of 10 to over 1,000 ties in main line tracks, or in sidings and industrial spur tracks in quantities of up to 10,000 ties. In 1965 the Norfolk Southern Railway installed a 31-1/2-mile industry track utilizing 70,528 MR2 ties and the Florida East Coast Railroad began installing the first of more than 70,000 MR2 ties on its main line north and south of St. Augustine, Fla. This was followed in 1966 by construction of a 32-1/2-mile industrial spur by the St. Louis-San Francisco Railway utilizing the MR2 tie. During this period a number of additional MR2 test installations were made in main line tracks by other railroads.

Continuing investigation and evaluation of in-service performance of MR2 tie installation prompted the AAR to recommend further modifications to that tie. These consisted basically of deepening the tie in the rail seat area and increasing the prestress by 25 percent. The resulting tie, known as the AAR Type 3, is commercially designated the MR3 tie. To date, only a few of these ties have been manufactured and installed in main line track.

III. Types of Concrete Crossties and Manufacturing Methods

Concrete crossties now in service can be classified as either prestressed or conventionally reinforced concrete. Many different types have been produced and installed for test and evaluation purposes. The only tie that has been installed in quantity in the United States is the prestressed AAR Type E (MR2) tie.

Railroad loadings in Canada are of the same magnitude as those in the United States. A number of types of concrete crossties have been installed for test and evaluation purposes on Canadian railroads, including ties of American design. Mention therefore will be made of concrete crossties installed in Canada.

The AAR Type E (MR2)

This tie was produced in at least three versions. Version one is shown in Fig. 1. The top surface of the tie is flat to accept the indirect-fixation rail fastening assembly. Version two is similar but has recesses in the top of the tie at the rail seat areas to accept the direct-fixation rail fastening assembly. Version three is similar to version two but the top surface of the center 3 ft. of the tie is recessed 1 in.

All MR2 ties are 8 ft. 6 in. long by 12 in. wide at the bottom and are characterized by a wedge shape at the bottom of the center three feet of the tie. Weight is about 620 lb. These ties are pretensioned with four 7/16-in. strands. Initial prestress force in the tie is 81,600 lb. Prestress force after losses is assumed to be 69,600 lb. Designed for a cracking moment of 150,000 in.-lb. under the rail, they must be able to withstand 200,000 in.-lb. of bending moment for 2 million cycles of load without failure.

While early AAR Type E ties were made by the long-bed method, this practice was soon discontinued in favor of special machines to boost daily production.

AAR Type 3 (MR3)

Similar in appearance to version three of the MR2 tie, the MR3 tie (Fig. 2) is deeper at the rail seats and has an initial prestress force of 100,000 lb. It is designed for cracking moments of 150,000 in.-lb. and 75,000 in.-lb. at the rail seats and center of tie respectively.

This tie, which incorporates the latest recommendations of the AAR, is in limited production. It is being produced by machine methods and also by a modified long-bed method. To date, a few have been installed in track.

The Gerwick Tie

The Gerwick tie, Fig. 3., so named because it was designed by Ben C. Gerwick, Inc., of San Francisco, Calif., has been installed in a number of locations for test and evaluation purposes. The design of the Gerwick tie is based on a center-to-center spacing of 30 in. for Coopers E-72 loading and 29-in. for E-75 loading. It is designed for a 132,000 in.-lb. bending moment with zero stress in the bottom of the tie under the rail, for a cracking moment of 191,000 in.-lb. under the rail, and for 112,000 in.-lb. bending moment in the center of the tie with the center top stress equal to zero.

The tie is pretensioned with eight 3/8-in. strands. Initial prestress force is 112,000 lb. Final prestress force is assumed to be 89,600 lb. Tie weight is about 540 lb. for an 8-ft. long tie. Gerwick has also developed an 8-ft. 6-in. tie and recently was awarded a contract to produce 146,000 9-ft. long prestressed concrete crossties (RT-2) for the Bay Area Rapid Transit system being constructed in the San Francisco, Calif., metropolitan area. Gauge for this system is 5 ft. 6 in.

To date all Gerwick ties have been produced by the long-bed method.

The Swedish "101" Tie

The Swedish 101 crosstie has been installed for test and evaluation purposes in one American and one Canadian railroad and in three rapid transit systems. The ties installed in the rapid transit lines are the standard ties used in Sweden (Fig. 4.) and were manufactured in Sweden.

Those being tested by the railroads also were manufactured in Sweden. They are similar to the standard tie but were designed for 79,000-lb. axle loads and have larger end blocks to increase track stability and reduce ballast pressure.

The 101 tie utilizes the two-block configuration, the blocks being connected by a steel tube. It is posttensioned and is manufactured by machine methods.

Other Prestressed Ties

A number of other prestressed concrete crossties have been installed on American or Canadian railroads for test and evaluation purposes.

The British Type F tie, a monoblock tie produced in England, is pretensioned and manufactured by the long-bed process.

The Miron tie, a monoblock tie produced in Canada is posttensioned and manufactured by machine methods.

The modified German B58 tie, a monoblock tie produced in the United States, is posttensioned and manufactured by machine methods.

The Dywidag B66 tie, a monoblock tie recently introduced and in production in the United States,is postensioned and manufactured by machine methods.

The Interpace-Abex tie, a monoblock tie recently introduced and produced in the United States, is pretensioned and manufactured by the long-bed method.

The French RS Tie

The RS tie is a non-prestressed two-block tie, the end blocks being connected by an angle bar. This tie has been installed for test and evaluation purposes on a Canadian railroad. The tie is produced by machine methods.

Methods of Manufacture

A review of the foregoing shows that, in general, posttensioned ties are manufactured by machine methods while pretensioned ties are manufactured by the long-bed method. An exception is the AAR Type E (MR2) tie which is a pretensioned tie manufactured by machine methods. The long-bed method lends itself exclusively to pretensioned ties while machine methods can be used for pretensioned ties as well as posttensioned ties. Each method has inherent advantages and disadvantages related to the manufacturing process as well as the appearance and performance of the finished ties.

The long-bed method consists of setting up a number of forms end to end on a prestressing bed; stressing pretensioning tendons, common to all forms, between abutments located at the ends of the bed; placing stirrups, inserts, or other embedded metal in proper position within each form; placing and vibrating concrete into each form; and curing. The prestress force is released to the ties when the concrete has attained a prescribed strength.

This method which uses a separate form for each tie being made on the prestressing bed requires a considerable investment in forms. This can be offset by using the bed for producing other structural members when it is not needed for tie production. The prestress force can be applied easily and reliably to the tendons and if "gentle" methods are used to release the prestress force to the ties an excellent uniformity of prestress force in the ties results. The vibratory compaction of concrete in the forms on the bed has not been as efficient as that obtained on the vibrating tables usually used in machine methods. This, coupled with the fact that an efficient steam curing chamber cannot be used, generally results in a lower-strength concrete than that obtained by machine methods. Finally, the rate of production per unit of labor and also per unit of required space has been lower than that attained with machine methods.

Although machine methods vary, they generally consist of a vibrating table which has either a built-in form or the capability of accepting removable forms. Posttensioning tendons with bond breakers are generally positioned in the forms prior to placing concrete. Vibratory compaction of the concrete is very efficient, resulting in extremely dense, high-strength concrete. Because of this density and the relatively dry mix used, the ties are removed from the forms immediately after forming. After a prescribed preset time the ties are placed in a steam chamber for curing. Other methods of curing may be used but when a high rate of production is required, steam curing is most efficient. The tendons are tensioned as soon as the concrete has reached a prescribed strength.

Machine methods lend themselves to automation. For example, concrete can be fed by conveyor from a central mixing plant and ties can be fed into the curing chamber on a conveyor system set to operate at a speed that will move the ties through the curing chamber in the prescribed curing time. Tie machines located in northern climates can be housed in a building so that production can proceed throughout the year.

Under conditions of partial utilization of equipment the long-bed process has the advantage. High initial cost for plant machinery is a major disadvantage of machine methods of producing concrete ties. This is especially true during the period when the ties are being introduced and the machinery is in operation on a part-time basis only. However, when the machinery is in full-time operation, the disadvantage of high initial cost disappears due to lower labor and certain other costs.

When pretensioned ties are produced by machine methods the tendons are stressed between the ends of the forms. Since the prestress force cannot be released to the tie until the concrete has attained adequate strength, separate forms are required for all ties manufactured during the time it requires to complete that portion of the manufacturing cycle. The machine used to produce pretensioned ties is unusually efficient in vibrating and compacting the concrete. Cubes cut from MR2 ties have yielded compressive strengths of 12,000 to 14,000 psi.

IV. In-Service Performance

During the eight years that concrete crossties have been in service on American railroads, many inspections have been made to determine their in-service performance. A number of the early installations were instrumented by the research staff of the AAR so that field loading conditions could be determined more accurately. Strain gages were placed at critical points on the ties, fastening systems, and rails so that stresses in those components could be determined. Devices were used to determine the rail-wave characteristics and the accelerations imparted to the ties by rail-wave action. Impact values were measured as was the distribution of load to the ties at 30-in. spacing.

The results of AAR's field investigations and recommendations for improving the AAR tie designs are contained in reports ER-20 and ER-58 published by the Engineering Research Division, Association of American Railroads, Chicago, Ill.

In connection with several concrete crosstie test installations, test sections of new timber crossties were installed at the same time at one or both ends of the concrete installations. This was to compare the in-service performance of the two types of ties under similar conditions of traffic, rail, ballast, drainage, and subgrade. All such comparisons revealed that the track with concrete crossties maintained better alignment and grade, especially with continuous welded rail, while providing a quieter, smoother ride.

Visual inspections of many concrete crosstie installations have been carried out by the AAR, the Portland Cement Association, and the individual railroads. Most of these inspections have dealt with ties of AAR design. The following comments therefore are limited to those ties. No attempt will be made to identify any particular test installation, since all inspections have indicated, in varying degree, similar in-service performance.

The most common condition observed is a pattern of hairline flexural cracks distributed over the top surface of the center 3 ft. of the tie, caused by center binding. More than 50 percent of all the ties observed show these hairline cracks. Such cracks usually are not considered signs of structural distress in prestressed members since they open only under load and close when not under load. However, under the severe weather conditions to which crossties are exposed, coupled with brine drip from refrigerator cars, corrosion of the prestressing tendons might occur or continual working at the crack interface might ultimately shorten the service life of the ties.

Occasionally, a wider single flexural crack near the center of the tie has been observed. This indicates a possible compressive failure of the concrete on the surface of the tie in contact with the ballast, caused by center binding, or a loss of prestress due to undetermined causes. Ties containing this type of crack can be considered to have failed.

Also of concern, but only occasionally observed, is a wide crack in the center portion of the tie at an angle of 45 deg. to its longitudinal axis. This appears to be a torsional failure. The author feels that monoblock ties with a greatly reduced center cross-section, when subjected to the loadings and conditions found on American railroads, may be subject to torsional problems.

Yet another type of serious crack has been observed. This crack extends upward through the tie and usually terminates at the top surface of the rail seat at a bolt hole. It is generally associated with bond failure and resultant tendon slippage. Prestress force in the tie beneath the rail seat is reduced and failure ultimately occurs. This crack occurs at the point of maximum moment and may be within the transfer length of the prestress force.

The author feels that the cracking in the rail seat area would be eliminated by posttensioning, by pretensioning with a greater number of smaller, preferably deformed tendons, by using end restraint on the pretensioning tendons, or by using a gentle method of releasing the prestress force to the tie. This would insure full prestress at the point of maximum moment in the rail seat areas.

The vast majority of AAR-type ties installed have utilized clips, bolts, bearing pads, and embedded inserts to secure the rail to the ties. Insufficient insert embedment depth in some crossties has resulted in rail seat pullout and failure of the ties. Clip breakage has not been excessive. However, there is a question about how flexible or rigid the fastening system should be. Very rigid and very flexible fastening systems have been used. Field performance indicates that something between rigid and very flexible might be best suited. The AAR has developed such a fastening system which has been fully tested in the laboratory and found acceptable but which has not yet been tested in the field.

It has been found that the rail fastening systems used with the AAR-type ties have not provided sufficient electrical insulation between the rails and the ties under certain conditions of moisture and with some of the more sophisticated signal and CTC systems. Considerable research has resulted in the development of several satisfactory methods of insulating the AAR-type of rail fastening system. These methods have been fully tested in the laboratory but have not yet been tested in the field.

Rail joints, switches, and poorly drained subgrades have presented problems for concrete crossties as well as for timber ties. Joint bars at rail joints are not sufficiently rigid to allow the rail to act continuously through the joint. This results in greater loads on the ties at the joints and settlement of the ties into the ballast, producing low joints. This, combined with rail end batter at the joint, causes a loading condition on the ties that produces failures in concrete crossties and deep plate cutting in timber ties. The solution to this problem might be to use wider ties adjacent to the joint to reduce ballast pressure, and to provide a more resilient pad beneath the rail to absorb the shock. These techniques, combined with adequate joint maintenance, should greatly reduce if not eliminate problems with ties at joints.

Because no economical method has been devised to accurately locate rail fasteners in switch ties, it is standard practice to use timber ties under switches. The wave action of rail supported on timber ties is greater than that of rail supported on concrete ties. The wave begins some distance ahead of the train and affects the first several concrete ties adjacent to the transition point between the timber and concrete ties. This results in excessive vertical movement of those ties, excessive stress in the rail fastening systems, and a tendency toward pumping. A solution to this problem might be to stiffen the track structure by spacing the timber ties closer together so as to reduce the magnitude of the wave action of the rail supported on timber ties to that of the rail supported on concrete crossties. Thus there could be a smooth transition between track having adjacent sections of timber and concrete crossties.

Impervious, poorly drained subgrades, soft or otherwise poor ballast, and lack of suitable maintenance have contributed, singly or in combination, to the problems encountered with concrete crossties in service on United States railroads. Excluding ties installed during early stages of development and others not conforming to the recommendations of the AAR, it is probable that more than 95 percent of the AAR-type crossties installed in mainline service are performing satisfactorily. The ties that have failed or are showing signs of serious distress have been subjected to batter at rail joints, excessive rail wave action, pumping, and consequent overloading caused by poor drainage, poor ballast and/or inadequate maintenance, or torsional loads exceeding the torsional capacity of the ties.

V. Research

As a result of in-service performance investigations of concrete crossties, several research projects were undertaken to find solutions to some of the more serious problems observed in the field. In addition to close cooperation between the research laboratories of the AAR and the Portland Cement Association during the early development of the AAR type crossties, the PCA has carried out independent research projects which have produced significant contributions.

Electrical Resistance of Crossties

Investigations have indicated that under certain conditions of moisture, concrete crossties will allow sufficient current to pass between the rails to interfere with operation of the more sophisticated railroad signal systems. Since these signal systems operate by passage of current through the rails, the problem could be solved only by insulating each rail from the tie or by increasing the electrical resistance of the tie itself. To save time and to guard against the possibility that the first approach might not produce a solution, both approaches were studied simultaneously, though independently.

Preliminary investigation of the electrical properties of concrete, including studies of both direct and alternating current, showed that moist concrete is essentially an electrolyte with resistivity of about 10,000 ohm-cm, a value in the range of semiconductors, while oven-dried concrete has a resistivity of about 100,000,000,000 ohm-cm, which makes it a reasonably good insulator.

After determining the nature of the electrical properties of concrete, many tests were carried out on 1-in. cement paste cubes and 4-in. concrete cubes in which all the known variables inherent in portland cement and portland cement concrete had been included so that the effect of these variables on the electrical resistance of the cement and concrete could be determined. The variables which were studied included type of cement, water-cement ratio, aggregates, curing, admixtures, temperature, moisture content, electrical frequency and potential, and coatings. The modifications where applicable were both quantitative and qualitative.

The conclusion was that there is no practical way to increase the electrical resistance of the concrete in crossties sufficiently to prevent interference with modern railroad signal systems; that insulation of the rails from the concrete would be the most effective way of obtaining adequate rail-to-rail resistance.

Electrical Insulation of Rail Fastening Systems.

Preliminary investigations indicated that there was no particular problem in insulating the rail from the concrete. Numerous materials with excellent insulating properties are available. The problem would be to select materials that would retain both their electrical properties and dimensional stability during millions of cycles of railroad loading.

Using clips, bolts, pads and embedded inserts as recommended for rail fastening systems by the AAR as a starting point, three different approaches to insulating the rail from the concrete were developed.

Method 1 (Fig. 5), recommended by the AAR, utilized the clips, bolts, pads, and inserts without change, but introduced a molded nylon insulating pad between the clips and the base of the rail. The base of the rail rested on a 3/16-in. thick polyethylene pad. The rail therefore was insulated from the concrete and the rail fastening system.

Method 2 (Fig. 5), utilized AAR-recommended bolts and inserts but modified clips and pads. The clip bolt holes were enlarged to allow a moulded nylon thimble to be inserted to electrically isolate the bolts from the clips. The 3/16-in. thick polyethylene pads were extended outward on both sides of the rails to separate the heels of the clips from the concrete. Thus the rails and clips were insulated from the hold-down bolts and the concrete.

Method 3 (Fig. 5), utilized AAR-recommended clips and bolts but modified pads and inserts. The pads were identical to those used in Method 2. The inserts were insulated by various coatings applied by a vacuum process. In addition, the bottom of each insert was closed and a wire loop was welded to the body of the insert to provide mechanical anchorage. Tests were made to determine which of the various coatings were effective in insulating the inserts from the concrete. Some of the effective coatings were hard and tough while others were relatively soft. Kynar, polyethylene, and epoxy coatings were found to improve the electrical resistance from about 600 to roughly 100,000 ohms per insert. See Fig. 6.

Short sections of rail were fastened, using Methods 1, 2, and 3, to cast concrete blocks having rail seats formed in the top surfaces and embedded inserts. The hold-down bolts were torqued to 150-ft.-lb. in accordance with AAR recommendations. The blocks were securely fastened to the bed of a rail fastener wear-test machine and subjected to 3 million cycles of simulated railroad loading conforming to the recommendations of the AAR.

This machine, undoubtedly one of the most sophisticated of its type, utilized Amsler pulsaters arranged so that loads producing 20 kips vertical reaction and 7.5 kips horizontal reaction from the gage side would alternate 180 degrees out of phase with loads producing 20 kips vertical reaction and 3.75 kips horizontal reaction from the field side of the rail head. The machine operates at 250 cycles of loading per minute. Each cycle consisted of the loads producing reactions from the gage side and the field side.

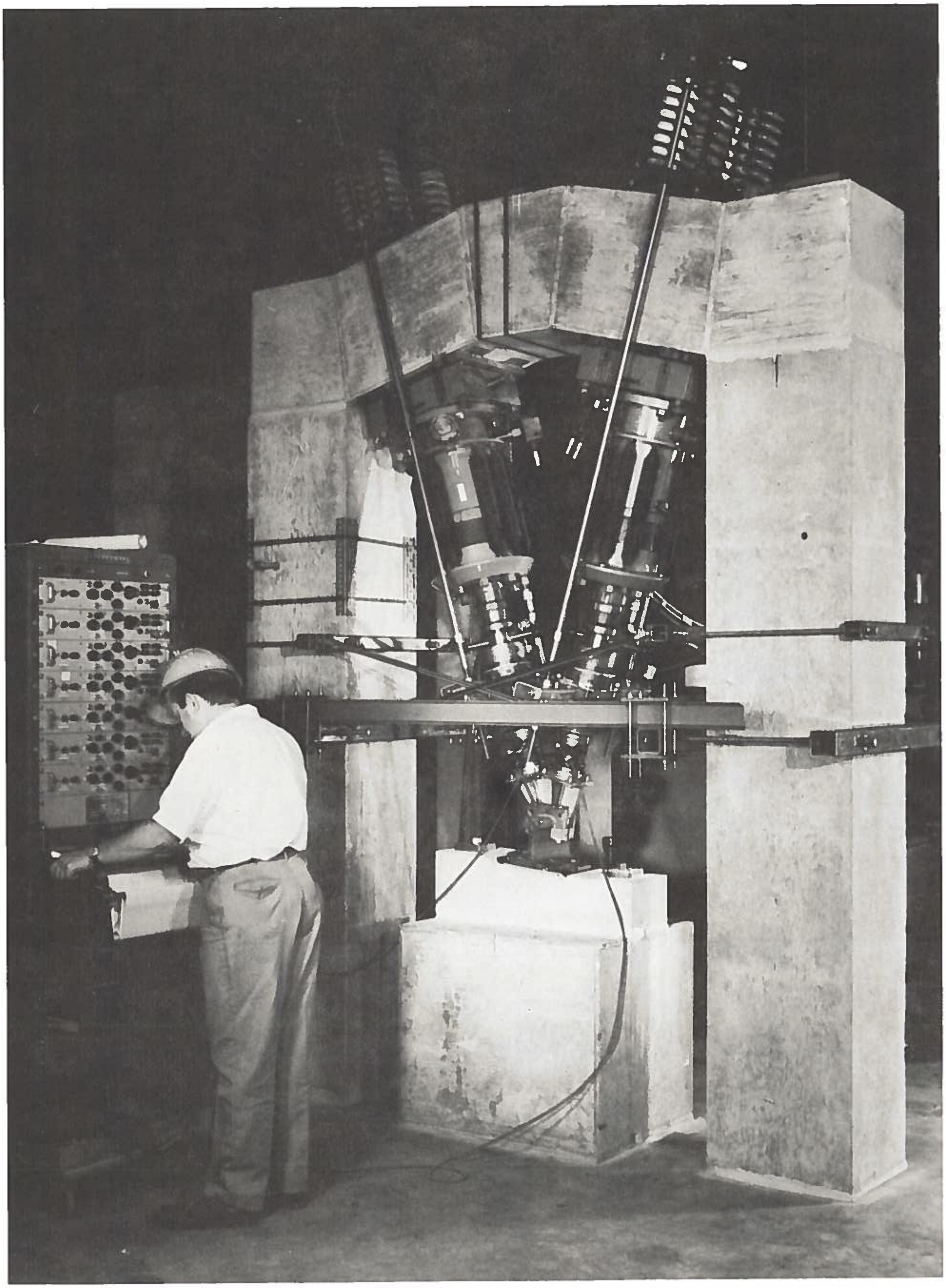

Fig. 6

During the tests it was necessary to re-torque the hold-down bolts four times with the Method 1 assembly and once with the Method 2 assembly to maintain the recommended torque. This apparently was caused by a slight cold flow of the nylon insulators. The bolts in the Method 3 assembly remained tight throughout the test.

Upon completion of the test, each assembly was disassembled and closely inspected for wear or other signs of distress. In all cases, wear was insignificant and the only sign of distress noted was some radial hairline cracking of the concrete around the polyethylene-coated inserts of the Method 3 assembly. Since the polyethylene coating was soft and relatively thick the inserts, due to flow of the coating, apparently were able to move in the concrete under the influence of the lateral loads and thus cause the cracking.

All methods tested successfully passed the wear test and subsequent electrical test. The performance of Method 3, using inserts with an epoxy coating, was superior to that of the other methods since it was not necessary to re-torque the bolts.

Insert Pullout Tests

Field observations indicate that ties having inserts embedded 1-3/4 in. into the concrete are subject to insert and rail seat pullout, while ties having inserts embedded 2-3/4 in., as recommended by AAR, are not. AAR recommends that inserts be able to meet a 9,000-lb.-per-insert rail seat pullout test. To determine the magnitude of the force required to pull the inserts from the concrete, pullout tests were conducted on inserts embedded in plain concrete blocks having approximately equal concrete strength. Standard inserts with 1-3/4 in. and 2-3/4 in. embedment were tested as well as plain and coated inserts as described under Method 3.

Standard inserts with 1-3/4-in. embedment pulled out at a force of about 6,700 lb.; standard inserts with 2-3/4 in. embedment pulled out at a force of about 12,500 lb., and the Method 3 inserts pulled out at a force of about 12,000 lb.

Strand Transfer Length

Field observations and laboratory flexural tests have indicated that crossties pretensioned with four 7/16-in. strands will sometimes fail by cracking through the rail seat area. The ties are designed for a cracking moment at the center line of rail of 150,000 in.-lb. The design assumes full prestress at that point. Such cracking in the field would indicate either that the design is not sufficient for field loading conditions, the ties were not properly made, or the full prestress was not available at the point of maximum moment.

Since the design is based on in-track strain gage measurements under known axle loads, it is unlikely that the basic design requirement of 150,000 in.-lb. cracking moment at the centerline of rail is inadequate for normal conditions. Also, although it is always possible for massproduced crossties to contain out-of-place strands or insufficient prestress, the machine method used to manufacture the crossties produces concrete having a compressive strength in the range of 14,000 to 16,000 psi and consequent greater cracking strength than the recommended 8,000-psi strength concrete. This greater cracking strength, for the most part, should offset manufacturing inadequacies.

Laboratory flexural tests have shown that some ties having concrete strengths in the recommended 8,000-psi, 28-day range have not been able to meet the 150,000 in.-lb. cracking moment requirement. If we assume the design is correct, it appears the assumption that full prestress is available at the point of maximum moment may not be entirely correct. It has long been suspected that the surface condition of the strand could cause variations in the transfer length of the prestress force; i.e., smooth bright strand would have a greater transfer length than rusted or deformed strand. However, the magnitude of the actual transfer length of 7/16-in. strands having bright, partially rusted, rusted or deformed surfaces using non-gentle methods of release of prestress to the concrete, was not known.

Accordingly, a series of tests was carried out on beams having tie-like size and design. All beams were pretensioned with 7/16-in. strands. Four beams had bright strand; two beams had partially rusted strand; two beams had rusted strand; and four beams had deformed strand. Prestress force was released to the beams by flame cutting the strands.

Transfer lengths were determined by measuring concrete strains on the exterior surfaces of the beams. After the transfer lengths were obtained, each beam was flexurally tested to determine cracking moment and ultimate moment at a distance from the end of the beam equivalent to that of the centerline of rail from the end of a crosstie. The results of the tests were as follows:

1. The average transfer length for the beams using bright strand was 28 in., average cracking moment was 148.9 in.-kips, and average ultimate moment was 202.2 in.-kips.
2. The average transfer length for the beams using partially rusted strand was 23 in., average cracking moment was 184.5 in.-kips, and average ultimate moment was 277.4 in.-kips.
3. The average transfer length for the beams using rusty strand was 19 in., average cracking moment was 188.75 in.-kips, and average ultimate moment was 283.6 in.-kips.
4. The average transfer length for the beams using deformed strand was 22 in., average cracking moment was 173.5 in.-kips, and average ultimate moment was 322.0 in.-kips.
5. The beams with bright strand did not consistently meet the 150,000 in.-lb. cracking moment requirement when tested in flexure. All beams, other than those having bright strand, did consistently meet the 150,000 in.-lb. cracking moment requirement.

6. The transfer length for bright 7/16-in. strand is considerably greater than the distance from the end to the point of maximum moment in a tie-like beam. Thus, full prestress is not available at the point of maximum moment.
7. The transfer length for partially rusted, rusted, and deformed 7/16-in. strand is equal to or less than the distance from the end to the point of maximum moment in a tie-like beam. Thus, full prestress is available at the point of maximum moment.

Since previous research shows that transfer length is only slightly influenced by concrete strength, it can be concluded that concrete crossties, pretensioned with 7/16-in. bright strand and using non-gentle methods of release, may be subject to cracking in the rail seat area caused by insufficient prestress force at the point of maximum moment.

Previous research also shows that the transfer length for 7/16-in. bright strand is approximately 16 in. when a gentle method of release is used. Thus, under those conditions full prestress probably would be provided at the point of maximum moment in concrete crossties.

VI. Outlook for the Future

There is an excellent chance that concrete crossties eventually will be widely used by American railroads. The pioneering work carried out by the AAR has created a solid base upon which to build. Those who may feel that European experience might have created such a base should recognize that American railroad loadings are very much greater than European railroad loadings, resulting not only in greater stresses in the crossties and rail fasteners but greater pressure on the ballast and subgrade as well.

Over the past few years axle loads and train speeds have been increasing in the United States and they are expected to continue to increase. It is predicted that axle loads in excess of 80,000 lb. and speeds greater than 100 m.p.h. may be rather common in the next 10 to 20 years. This combination of load and speed may prove to be near the upper limit of or even beyond the capability of the steel wheel on rail-crosstie on ballast concept.

The cost of maintaining a track structure which is operating near the upper limit of its inherent capability would be considerable, though not nearly as large as the cost of replacing all existing main-line tracks with an entirely new rail support concept having greater inherent capability. The author feels, therefore, that future railroad loading in the United States may consist of two types of loading: heavy axle loads at medium speeds and lighter axle loads at higher speeds. The 350- and 500-car trains consisting entirely of 100-ton capacity hopper cars recently operated at 40 to 50 m.p.h. by the Norfolk and Western Railroad is a preliminary example of the first loading condition. The "Super C" high-priority freight trains recently operated at 80 m.p.h. by the Santa Fe Railroad is a preliminary example of the second loading condition. Each condition has its own particular problems.

The heavier loads will create high pressures in the ballast and subgrade and produce high bending stresses in crossties. These problems can be overcome by using high-quality ballast, specially stabilized and well drained subgrades, wider ties or reduced tie spacing to reduce ballast pressure, and maintenance intervals which are of such length as to insure the track structure will never be below a specified minimum acceptable state of maintenance. Trouble spots should be detected and remedied as soon as possible rather than waiting for the normal maintenance period.

Light axle loads and high speeds will create high stresses in the rail fastening systems caused by high vertical accelerations imparted to the ties by rail wave action and wheel and track irregularities. Field measurements taken at normal speeds on concrete crosstie test installations indicate accelerations in the range of 60 g. Conceivably, 100 m.p.h. speeds might produce accelerations of considerably higher magnitude. It is felt that rail fastening systems can be designed to handle these accelerations.

Thus, the concrete crossties of the future may be designed for each of the aforementioned load-speed conditions. The basis of design will be the maximum field loads imparted to the ties in a track structure which is in a specified minimum acceptable state of maintenance.

Research by at least one major railroad appears to be headed in that direction. An instrumented concrete crosstie test installation is being constructed in which ballast and subgrade pressures as well as tie, fastening and rail stresses are to be studied. The results of these studies could form the basis for future studies.

Much has been learned about the in-service performance of concrete crossties. Observed problem areas have become subjects of research and development, and solutions have been found. Although the knowledge required for designing and properly manufacturing prestressed concrete crossties for specific railroad loading conditions is now in hand, that knowledge has not yet been fully applied. Work in this area is continuing and it is hoped that concrete crossties incorporating all available knowledge will be under test in the not too distant future.

The author predicts that future tests will be completely successful and that concrete crossties will come into wide use on American railroads.

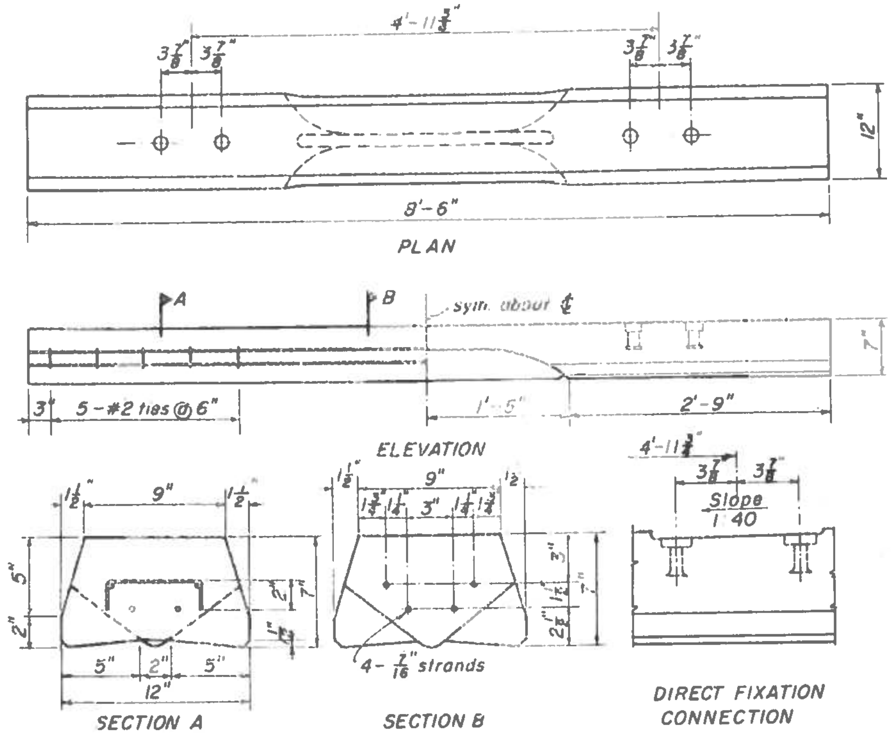

Fig. 1 - AAR Type E Tie

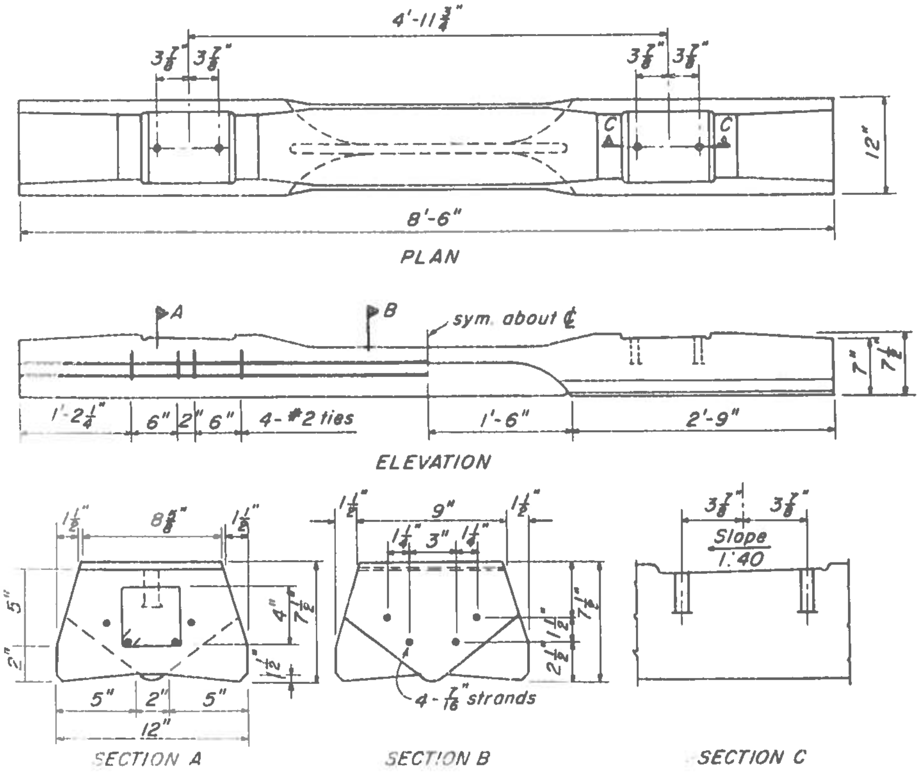

Fig. 2 - AAR Type 3 Tie

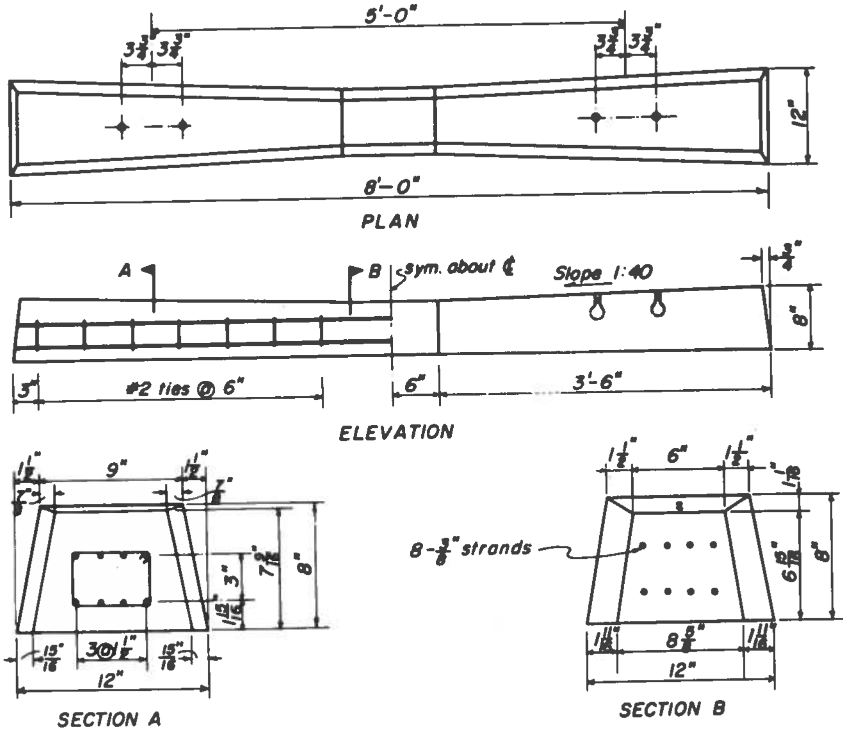

Fig. 3 - Gerwick Tie

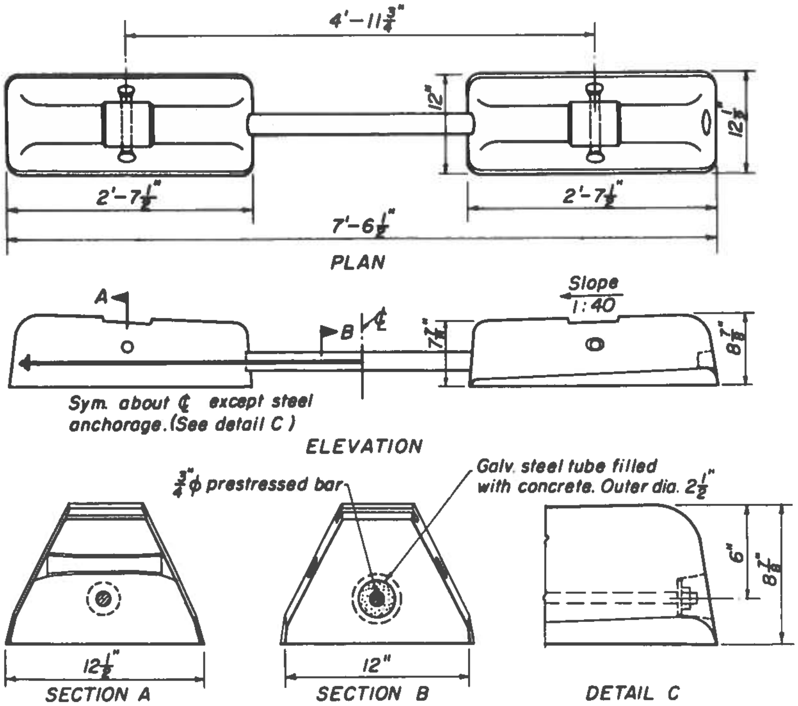

Fig. 4 - Swedish "101" Tie

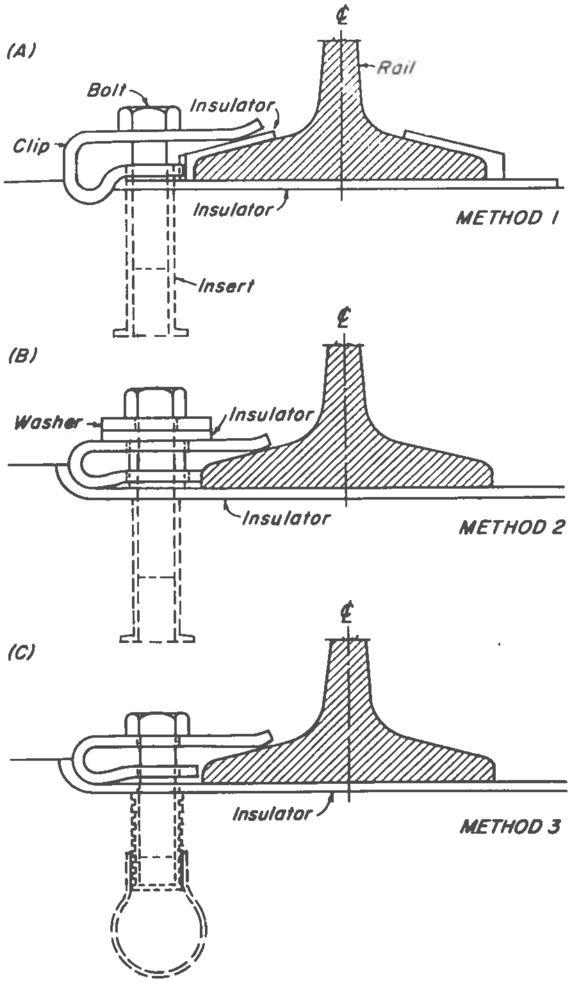

Fig. 5 - Insulated Rail Fastening Assemblies