AN INVESTIGATION OF CONCRETE TIES AND RAIL FASTENERS

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SYNOPSIS

During the past four years, the Department of Civil Engineering, San Jose State College, San Jose, California, has assisted in an extensive research and development program for the Ben C. Gerwick Company, San Francisco, California, in the development of mass transit and main line railroad prestress concrete ties.

The program included the following:

Rail seat and tie center static loading of the RT-7 tie.

Repeated load tests on the rail seat area.

Repeated load test on a rail fastener system.

Field investigation of the concrete ties, fasteners, and rail in service.

This paper presents an account of the methods, techniques, and equipment utilized in the laboratory and field testing program. Also included are the results of the tie and fastening system investigation.

1. INTRODUCTION

In the United States, user railroads require that proposed concrete ties and rail fastening systems be subjected to laboratory tests prior to acceptance. In the recent past, the AREA Proposed Specifications [1] have served as the criteria for acceptance of ties and fastening systems. At the present time, the American Railway Engineering Association has completed the AREA Preliminary Specifications for Concrete Ties and Fastenings [2]. These specifications have been proposed by a Special Committee on Concrete Ties of the AREA with assistance provided by the Concrete Tie Committee of the American Concrete Institute. These specifications are currently undergoing review and evaluation by the membership of AREA before final adoption.

Among the test requirements of the AREA Proposed Specifications [1] and the AREA Preliminary Specifications [2] are static and repeated load tests of the rail seat of the tie and repeated load tests of the rail fastening system.

2. DESCRIPTION OF CONCRETE TIES

The ties used in this investigation, designated RT-7 MK 37 and RT-7 MK 38, were 9'0" in length with an 8-3/8" height and 11" width. Concrete is designed for a minimum compressive cylinder strength of 6000 psi at 28 days. The MK 37 includes five 7/16" diameter prestressing strands with a total prestressing force of 108,000 lb. The Mark 38 tie is prestressed to

The rail anchorage at each rail seat consists of two 2-3/4" long cylinders with a flared end and 3/4" diameter internal threads. The material is stainless steel Type 304 meeting ASTM A-167-54 Specifications.

3. STATIC LOADING OF RT-7 TIE

In the process of developing the RT-7 tie, a large number of static tests were made to determine the cracking and ultimate moments and general structural behavior of the rail seat area and the tie center. The tests conducted in the structural laboratory of San Jose State College are herein reported.

The specified loading arrangement for static (and repeated) bending in the rail seat area which is in accordance with the original AREA Specifications [1] is shown in Figure 1(A). The modified loading arrangement as proposed in the AREA Preliminary Specifications [2] is shown in Figure 1 (B) During the course of the investigation, both arrangements were used.

The specified loading arrangement for static bending at the tie center is in accordance with the original AREA Specifications [1] as shown in Figure 1(C). The required loading arrangement of the AREA Preliminary Specifications [2] is similar except that the distance between load points is 6 inches instead of 8 inches.

In the latest series of tests, the static flexural capacity of the MK 38 rail seat area was investigated. The average moment at first cracking was 230,000 in-lb. which compares with and exceeds the requirement of 150,000 in-lb. in accordance with the 1968 AREA Specifications [1] and 220,000 in-lb. which is the required minimum cracking moment for a 9'0" tie at 30" spacing proposed in the 1971 preliminary AREA Specifications [2].

The corresponding average ultimate moment of the MK 38 ties was 380,000 in-lb. which is 165% of the average cracking moment. No ultimate moment capacity requirement is specified by the specifications because of the repeated load endurance requirement.

One of the significant factors that affects transfer length and subsequently bond strength is the diameter strand. During the development stage four 1/2 in. diameter strands were used instead of the six 3/8 in. diameter strands which are used in the current MK 38. Invariably, under static loading of the rail seat, cracking moment and ultimate moment capacity were a function of bond strength and not flexural capacity in ties with the four 1/2 in. diameter strands.

In observing the cracking behavior, the first crack was generally in a vertical direction at the beam soffit directly beneath the outer load point. With further increase in load, the crack widened and subsequently a second crack would develop. The second crack was usually oriented at about 30 to 45 degrees from the horizontal, leading from the outer load point to near the outer support.

Ultimate capacity also was attributed to bond failure of the strands between the end of the tie and the outer support. In most cases only two major cracks developed; the largest crack at ultimate was usually the first one that developed. The slippage of the strands, (predominantly the lower strands) was quite evident at this stage of loading.

In static tests on the MK 38 the cracking moment capacity and ultimate moment capacity has been increased owing to the increased surface area of the six 3/8 in. strands. Although slight strand slippage occurred at failure the rail seat area supported a moment approximately equal to the theoretical ultimate flexural capacity.

The design appears to be well balanced in that ultimate flexural capacity is close in magnitude to the ultimate capacity based on bond strength. A typical crack pattern at ultimate moment is shown in Figure 2.

The 1966 AREA Specifications [1] require a minimum negative moment of 125,000 in-lb. at the tie center to ascertain that the tie can support possible loading due to center binding which could produce negative moments. The AREA Preliminary Specifications [2] have increased this requirement to 150,000 in-lb. for 9'0" length ties. There is no requirement for ultimate moment capacity. Tests indicate that the cracking moment capacity is 155,000 in-lb. and the ultimate moment capacity 300,000 in-lb. for the RT-7. The type of failure is that of a typical under-reinforced concrete beam with elongation of the strands and crushing of the concrete is the zone of maximum moment.

4. REPEATED LOAD TESTS OF THE RAIL SEAT AREA

As a means of determining the fatigue characteristics of concrete ties under service conditions, laboratory repeated load performance tests are required prior to acceptance by the user railroad. The specifications that were first established by the AREA [1] required that the tie be subjected to a loading condition as shown in Figure 1(A) producing moments in the rail seat area varying from 20,000 in-lb. to 200,000 in-lb. For acceptance, the tie should withstand 2,000,000 cycles of repeated loading without failure.

Modifications were made to the original specifications by the Special Committee on Concrete Ties of the AREA [2]. The loading condition was changed to that shown in Figure 1(B) and the cyclic loading requirement made more severe. The initial phase of the test requires that a slowly applied vertical load be applied until a structural crack reaches the level of the lower layer of reinforcement. The tie shall then be subjected to 3,000,000 cycles of repeated loading with each cycle varying between 4000 lb. and a moment equal to 110% of the required minimum cracking moment. The tie must withstand this punishing cyclic loading without failure to be acceptable.

In the research and development reported herein, RT-7 ties were subjected to repeated load tests in a pulsator type hydraulic fatigue machine (Figure 3). The machine applies a constant upper and lower sinusoidal load through the entire loading history. All ties tested survived a minimum of 2,000,000 cycles of loading.

In one test, an RT-7 MK 37 was first loaded statically to a level which produced cracking at a moment of 200,000 in-lb. Subsequently, it was subjected to the loading condition as required by AREA [1], with the moment at the rail seat area ranging from a minimum of 20,000 in-lb. to a maximum of 200,000 in-lb. at a cycling frequency of 300 cpm. The tie survived 2,025,600 cycles of loading without showing any signs of structural distress Having satisfied the requirements of AREA [1], the fatigue test was terminated and the tie loaded statically to failure. The static ultimate moment was recorded at 288,000 in-lb., a reduction in strength of less than 8%, when compared to the opposite rail seat which was loaded to failure without cycling.

As part of the program, the fatigue characteristics of the RT-7 MK 38 tie with six 3/8" indented strands were studied by loading the tie in accordance with the AREA Specifications [1]. In the case of one tie, it was first loaded statically to a maximum moment of 200,000 in-lb. Under this loading condition at maximum moment there were no concrete structural cracks observed with a 5X illuminated magnifier. Overall, there appeared to be no effect on the structural integrity of the tie.

The tie was then subjected to 2,101,000 cycles of loading under this condition without failure. Moreover, there were no signs of serious structural distress observed during this loading history.

Upon static loading, two cracks of 3/4" in height, directly beneath the load points, were observed at 154,000 in-lb. of moment. Upon loading, the cracks closed and became invisible at 140,000 in-lb. of moment. Upon loading to 200,000 in-lb. of moment the cracks increased to approximately 1 inch in height. The load was increased to a magnitude of 242,000 in-lb. which produced a maximum height crack of 2 1/2 inches which is equal to the concrete cover of the lower layer of strands. In Figure 4, structural crac are observed with a 5X magnifier.

The tie was then subjected to an additional 3,045,930 cycles of loadin at the magnitudes specified by AREA [2], without failure. The maximum moment of 242,000 in-lb. to which the tie was subjected is 110% of the required flexural cracking moment. Subsequent to this loading condition, the tie appeared to be in sound structural condition.

The rail seat was then statically loaded to an ultimate moment of 407,000 in-lb. which was only 4% less than the ultimate moment capacity of

5. REPEATED LOAD TEST ON A RAIL FASTENER SYSTEM

To simulate vertical forces due to wheel loads and simultaneous horizontal forces due to lateral hunting and centrifugal action, a repeated load test was conducted on a fastening system consisting of fasteners (clips) Type 3 (Figure 15), an elastomeric pad (6" x 9" x 3/16"), and two 3/4" x 3 1/2" ASTM A307 Class A (SAE Grade II) steel bolts.

Loading Arrangement

The fastener system was loaded in accordance with the American Railway Engineering Association Specifications [1]. The specifications require that the fastener system be loaded by means of a section of rail to which loads are applied. For acceptance, the entire system should be able to withstand 2,500,000 cycles of loading without failure of any part. The loading consists of alternating loads from the field and gage side. The loading from the gage side has a resultant magnitude of 21,370 lb. at an angle of 20.55° with the vertical. The loading from the field side has a resultant magnitud of 20,350 lb. at an angle of 10.60° with the vertical. See Figure 5.

To accomplish this, a half section of a Gerwick RT-7 tie was used as the supporting block. Attached to the tie were fasteners and an 18 in. length of 119 lb. rail to which was welded a loading fixture. The entire assembly was placed in a large rectangular steel loading frame and attached to an 18W64 steel section positioned in an upright position. Four threaded inserts cast in the bottom of the tie were bolted to an inclined plate which in turn was bolted to the upright. The plate was positioned at an angle of 4.98° with the horizontal by means of a roller spacer. With the load actuators each positioned at 15.58° with the vertical, the desired component of loading were obtained. See Figure 6.

Compression loads varying from 0 to 21,370 lb. on the gage side and from 0 to 20,350 lb. on the field side at the specified angles, were applied alternately with the two actuators. The frequency of loading was 120 cycles per minute. The application of a single load from both rams (two load applications) constitutes one cycle of loading.

Prior to loading, the unlubricated anchor bolts were each torqued to 150 ft-lb. (Figure 7). The loading was applied continuously for 24 hours per day, including weekends, except for stoppages required for checking of the loading, inspection of the equipment, and torque readings.

Load Control

The alternating repeated loading on the rail fastener system was obtained with two 100,000 lb. capacity hydraulic actuators equipped with servo valves and manifolds. Spherically seated loading plates were attached to each of the actuator ends. An electrical strain gage load cell was placed between each actuator and the loading head, as shown in Figure 8. Each actuator was positioned at 15.58° from the vertical to obtain the correct angles of loading. A 10 gpm hydraulic unit served as the power supply.

Loading of the fastener was controlled with a servo controller connected to each actuator. The servo controller is the control unit of the closed-loop servo system.

Each load cell contained two Wheatstone bridges. One bridge was utilized for load control and the other to record the load. For greater accuracy, a strain indicator was used to establish the required loading.

To obtain the desired force-time relationship of loading, the output of a sine wave function generator was modified to obtain a half sine wave of loading for each actuator. This was accomplished by designing and fabricating an auxiliary circuit which included components of diodes and resistors. The resultant signal input into the servo controller was precisely that required to obtain the desired alternating load response. A 2-channel chart recorder was employed to check the time response of each force during the cyclic loading.(Figure 9).

Test Results

During the test, the following items were monitored or observed:

(a) torque on anchor bolts
(b) temperature change of rail base
(c) deflections of fasteners (rail clips), anchor bolts, rail base, and rail head
(d) possible wear of the fastener system components

No torque loss in the anchor bolts was observed until 2,097,000 cycles of loading. At this point, the gage clip torque has been reduced to 65 ft-lb. but the field clip torque was at the original level of 150 ft-lb. The reduction in torque is attributed partly to slight permanent deformation of the pad and in part due to slight tensile deformation of the gage bolt.

At 2,500,000 cycles, having successfully satisfied the AREA [1] requirements, deflection readings were taken and the system inspected. The torque readings were 52 ft-lb. gage; 144 ft-lb. field.

The reduction in anchorage torque indicates that it is necessary for the user railroad to include anchorage retorquing in its maintenance program if satisfactory service is to be expected.

Deflections

Deflections of various parts of the fastening system were taken at 1 cycle and 2,500,000 cycles. Designations shown on Figure 5 are:

G - Gage Load
F - Field Load
1 Vertical deflections at top of rail base on gage side
2 Vertical deflections at top of gage clip
3 Vertical deflections at top of rail base on field side

5 Horizontal deflection of rail base
6. Horizontal deflection of gage clip
7 Horizontal deflection of gage bolt
8 Horizontal deflection of field clip
9 Horizontal deflection of field bolt
10 Vertical deflection - top of rail head
11 Horizontal deflection - top of rail head

In the following tabulation, the positive sign (+) indicated upward movements or movements to the right. The negative sign (-) indicates downward movements or movements to the left.

DEFLECTIONS OF RAIL FASTENER SYSTEM (0.001 IN.)

Cycles	Deflection Point	Due to G	Due to F
1	1	+2.5	0
	2	+2	0
	2	+2	0
	3	-0.5	+2
	4	-0.5	+1.5
	5	-	-
	6	+5	-2
	7	+6	-2
	8	+6	-3
	9	+7.5	-3
	10	-3	-1.5
	11	+3	-0.5
2,500,000	1	+5	-2
	2	+5	-2.5
	3	-1	+0.5
	4	-1	+1.5
	5	+9	-8
	6	+2.5	0
	7	+3	-0.5
	8	+3	-1.5
	9	+7	-4
	10	-1	0
	11	+7	-1

The magnitudes of lateral and vertical movements of the rail under simulated loading conditions of the type described herein are an index of possible train instability. The small magnitude of movement of the various parts in both the vertical and horizontal directions of this fastening system indicate that the fastener system is sufficiently stiff to maintain train stability after 2,500,000 cycles of loading.

Clips

There was no structural failure of either clip. During the test, corners of each clip rubbed against the top of the rail base due to lateral rail movement. This motion produced a fine rust colored residue due to abrasive wear of both clips and the top surface of the rail base.

Elastomeric Pad

The pad displayed some wear on both top and bottom surfaces due to abrasive action. The degree of wear on each surface seemed to be approximately equal. Longitudinal lines forming the pattern at each surface were still quite visible after 2,500,000 cycles. A careful assessment of the endurance of the pad would indicate that excellent service could be expected.

Bolts

At 2,500,000 cycles of loading the bolts displayed no sign of structural distress.

Temperature

During the entire loading history there was no significant difference in temperature between the rail base in the vicinity of the clips, and ambient. The maximum difference recorded was 2° F.

Test Results - After 3,000,000 Cycles of Load

At 2,718,000 cycles the field and gage bolts failed at the top of the threaded insert. At the field side two or three lengths of insert thread were broken out. To date, there is strong indication that the ASTM A307 bolts are inadequate in withstanding a large number of repeated rail loads. An improvement would be made by installing ASTM A449 (SAE Grade V) bolts, which have a higher yield and tensile strength than the A 307 bolt.

For this test, the bolts were replaced, torqued to the torque which existed prior to initiation of failure, and the test resumed.

At 2,905,000 cycles the field bolt failed at the top of the broken insert. The broken stub within the insert was removed and the bolt replaced and retorqued to the value of torque which existed prior to failure.

The test was then carried successfully to 3,000,000 cycles.

There was no more discernable wear in the elastomeric pad than existed at 2,500,000 cycles.

There was no structural failure of either clip at 3,000,000 cycles. The rate of abrasive wear seemed to decrease near the end of the test. The gage clip had a small section of a corner chipped out (5mm) due to the bolt failure at 2,905,000 cycles.

The fastener system components after 3,000,000 cycles of loading are shown in Figures10 and 11.

6. FIELD INVESTIGATION OF CONCRETE TIES, FASTENERS AND RAIL IN SERVICE

The field investigation was conducted in a main line section of the Western Pacific Railroad test site near Sacramento, California (Figure 12). The laboratory support program took place at the Structural Laboratories of the Department of Civil Engineering of San Jose State College.

The study utilized 12 ties of similar design except for variations in the shear reinforcement and slight variation in the seating arrangement. Of the 12 ties analyzed in the study, seven were strain gaged, calibrated in the laboratory, and installed in the main line, one was strain-gaged and kept in the laboratory for control purposes, two were used to obtain cracking and ultimate moments and two were used to obtain prestress losses with electrical strain gages and Whittemore strain gages.

The following were investigated: field static moments at the rail seat and centerline of the ties, moments in the ties with 28 in., 30 in. and 32 in. spacings, moments in a concrete tie placed in a timber tie section, moments in ties due to moving passenger and freight trains, rail stresses for 28 in., 30 in., and 32in. spacings under static and dynamic loads, strand stresses, prestress losses, stresses in three different types of fasteners, and track modulus for the concrete tie section and for a timber tie section.

Prestress Losses and Transfer Length

Electrical strain gages were used to determine prestress losses and transfer length.

Prior to prestressing, strain gages were installed on strands of two ties, located 6 in. to the outside of the center of rail seat, at the centerline of the rail seat, and 6" from the rail seat, toward the center of the tie. The strain readings, which were used to calculate strand tensile stress, were taken at various prestress levels during the prestressing operation. In addition, strains were obtained prior to release after the ties had cooled for 2 1/2 hours, after release prior to removal from forms, and after removal from forms.

The stresses at full prestress of 193 ksi (calculated from strain readings) compared very favorably with the theoretical stress (load divided by cross-sectional area) of 191 ksi.

The prestress loss due to release averaged about 10 ksi or approximately 5%. It was also determined that the transfer length was within the end 18 inches of the tie.

Seven Whittemore gage points were installed on the concrete surface at the ends of the two ties. One point was located 1" from the end and the other six at 5" centers along the tie longitudinal centerline, at the tie bottom (top surface during casting). Readings were taken prior and subsequent to release, and compared with those obtained with the electrical

Calibration of field ties

The seven ties that were subsequently installed in the track section at the test site were first calibrated in the laboratory.

Each of the seven ties was instrumented with twelve linear electrical resistance strain gages. Gage locations for a typical installation are shown in Figure 13. Gages 1 through 9 were attached to one side of the tie only. Gages 10S, 11S, and 12S were attached to the surface of a single wire of the middle strand of the lower layer. The strand gages were attached and waterproofed after tensioning of the strands at the prestressing plant. The concrete gages were attached and waterproofed in the laboratory.

Each tie was calibrated by loading it in a universal testing machine as shown in Figure 13. Supports were placed close to the ends in order to produce significantly higher moments in the end zones and higher strain readings from the strain gages in these zones. The magnitude of load P used for the calibration was limited to that corresponding to 90% of cracking moment. Strain readings taken at increments of 0.25 were used to obtain a relationship between strain and moment at each gage location. Calibration curves were drawn for this data.

To obtain a similar relationship for negative moment, the tie was loaded in an upside down position with the same support and loading conditions.

Installation of ties at test site

The seven calibrated ties were installed with 120 similar ties at the Western Pacific Railroad test site which is a tangent section of track with continuously welded rail and 18 in. of ballast. The ties, including those calibrated, were spaced at 28 inches, 30 inches, and 32 inches. One calibrated tie was installed in a timber tie section at 19 inch spacing. Various fastening systems were used but all rail anchorage bolts were initially torqued to 150 ft-lb.

Static readings of the strain gages on the ties were taken three weeks after installation of the ties and dynamic readings two months after installation.

Static Scale Car

In order to obtain static readings fo. the tie, rail, and fastener loadings and readings for track modulus, a 40 foot long scale car weighing 100,000 lb. was utilized. The car had two trucks each with four wheels longitudinally spaced at 7'8". At the middle of the car were four hydraulically actuated jacks which lowered steel blocks onto the rails and caused the car to lift from the rails. In this loading position each block exerts a 25,000 lb. force on the rail. The blocks are fixed at a longitudinal spacing of 7'0". It is possible to position the car (and blocks) at any desired location along the track.

The 25,000 lb. block loading (50,000 lb. per rail) is compared to the AREA [2] design rail seat load of 37,000 lb. for ties of 30 inch spacing and 28,000 lb. for ties of 21 inch spacing. The maximum American railroad wheel loading is 41,000 lb.

Moments Due to Static Loading of Scale Car

The block load of 25,000 lb. from the scale car was placed directly over each tie being considered, thus imposing a static loading of 25,000 lb. at the center of each of the two rail seats of a tie. Due to the fixed spacing of the jacks, the other two jack loads of 25,000 lb. were located 7' 0" downtrack from the tie being investigated. The rail seat and center line moments due to this loading were read directly from the calibration curves which were obtained from the laboratory strain gage readings.

Due to the scale car loading, the tie rail seat moments ranged between +80,000 in-lb and +136,000 in-lb with the larger moments acting in the ties spaced at 32 in. In most cases, the rail seat moments in the 8'6" ties were less than those in the 9'0" ties.

The centerline moments ranged between +27,000 in-lb. and +46,000in-lb. Here also the larger moments were produced in the ties spaced at 32" and the 8'6" ties registered smaller moments than their 9'0" counterparts.

The distribution factor of loading (ratio of total tie load to axle load) was determined to be 35% for the ties spaced at 30". Uniform distribution of ballast pressure was assumed. This factor compares with 50% reported by the Association of American Railroads [3] and a value of 60% which is recommended by AREA [2].

An attempt was made to transform the moment distribution (obtained from the strain gage readings) along the length of a tie to ballast pressure distribution by determining the second derivative of moment. Due to experimental error, the results were inconsistent, thus are not reported herein.

The single prestressed tie in the timber section developed a positive moment of 92,000 in-lb. at the rail seat and a negative moment of 17,000 in-lb. at the centerline.

Moments Due to Moving Loads

Strains were recorded with multi-channel strip chart recorders for loadings due to moving trains. Readings were taken for trains moving in both directions. Passenger trains including the California Zephyr traveled in each direction at 70 mph with maximum wheel loads of 32,660 lb. Freight trains with maximum wheel loads of 32,280 lb. traveled in both directions with speeds ranging from 15 mph to 55 mph.

The maximum recorded positive moment under dynamic loading was +180,000 in-lb. This moment occurred at the centerline of the rail seat of a 9'0" tie and was due to a freight train with a maximum wheel load of 32,200 lb., traveling at 55 mph. A typical moment vs. time output for a

The impact factor due to this train was 20%. This factor was based upon maximum readings taken during the passing of the freight train and compared with maximum readings of another freight train traveling at 0 to 15 mph. Both trains had similar maximum static wheel loads.

The maximum recorded negative moment at the centerline of the rail seat under dynamic loading occurred in the same tie and due to the same load as for maximum positive moment. The moment of -112,000 in-lb. developed the instant after the wheel causing maximum positive moment unloaded. This seems to indicate that maximum negative moments of an appreciable magnitude are likely to develop as a result of sudden positive moment unloading caused by inertial forces in the section of the tie which lies outside the rail.

The maximum positive moment at the center of the tie was +60,000 in-lb. The maximum static wheel load was 32,660 lb. The maximum negative moment at the same point was -50,000 in-lb. and was due to the same train. It was difficult to determine whether the maximum negative moment directly followed the maximum positive moment owing to external disturbances being picked up by the galvanometer of the dynamic recorder.

Comparative Rail Stresses

Electrical strain gages were placed on the bottom surface of the rail in the field to determine maximum tensile stresses under various loading conditions and tie spacings.

The average rail stresses due to static loading of the scale car are as follows:

32 in. spacing	-	8000 psi
30 in. spacing	-	6040 psi
28 in. spacing	-	4280 psi
23 in. spacing	-	6430 psi (wooden tie section)

These are maximum tensile stresses which occurred at the bottom of base of the 119 lb. rail. Maximum compressive stresses will be somewhat higher. The 25,000 lb. jack load from the scale car was placed directly over the strain gages which were located at the mid-point of the span between adjacent ties.

Strain gage readings of the rail under dynamic loading were taken at only one location, at a section of 32 in. tie spacing. The maximum tensile stress at the rail base varied between 13,400 psi and 20,400 psi for the various passenger and freight train loads at different speeds. The maximum stress was due to a passenger train with maximum wheel loads of 32,660 lb. traveling at a speed of 70 mph. Based on the static response of the rail gage readings, the dynamic impact factor is 95%.

Tension and Torque in Fasteners

Three different types of fasteners (Figure 15) were installed at the test site to determine their response under field loading conditions. All were initially torqued at 150 ft-lb with the standard 3/4 inch diameter anchorage bolt.

Prior to field installation electrical strain gages were attached to the top surface opposite the bolt hole of each of the three types of fasteners. In the laboratory, relationships were obtained between strain and tensile bolt load. In another series of tests on the same fasteners, strain was obtained for increments of torque. These values were obtained from tests on inserts of three different ties. Superimposing the total experimental strain data resulted in a relationship between torque and bolt tension for the system of anchorage insert, anchorage bolt, fastener and base pad.

At maximum torque of 150 ft-lb., the corresponding bolt tensile forces were:

Fastener Type I	8550 lb.
Fastener Type II	7500 lb.
Fastener Type III	8600 lb.

In the field, the same strain gaged fasteners were installed with the existing field bolts and strain readings taken at increments of torque. The strain vs. torque relationship was linear, as in the laboratory, but the strain readings were much larger than those recorded in the laboratory for similar torques. The calibration curves obtained from the laboratory data were used to convert strain to bolt tensile force.

At 150 ft-lb., the corresponding bolt tensile forces were:

Fastener Type I	11,800 lb.
Fastener Type II	12,900 lb.
Fastener Type III	11,100 lb.

It is known that the bolt tensile force for a given torque is a function of type of thread, bolt size, frictional coefficient at the head, and frictional coefficient in the shank. The reason for the higher bolt tension in the field than in the laboratory is that the frictional coefficient was lower in the field owing to the lubrication of the bolts at installation and deposits of oil from residue of passing trains.

At 150 ft-lb. of torque on the bolts, the scale car was alternately moved onto and away from the ties with the fasteners under investigation. The difference in strain, and hence bolt tension between the loaded and unloaded condition was negligible.

Track Modulus

Track modulus is an extremely significant factor in track system structural behavior. Track modulus is a function of the tie flexural stiffness, tie width and length, tie spacing, type of fastening system, type of ballast and subsoil, compaction of ballast and subsoil, and other factors. It's variance in magnitude during loading history produces a

The Association of American Railroads [4] reports that based upon field investigations track modulus for concrete tie sections was much higher than the track modulus for a timber tie section, primarily due to the higher flexural stiffness of the prestressed concrete ties.

In the field study reported herein, track modulus was determined for a section of 9'0" long concrete ties spaced at 28 in. and a section of 7" x 9" x 8'6" fir timber ties spaced at 23in.

To measure deformations for track modulus, one mechanical dial gage was placed at each of seven consecutive ties for one rail and the scale car moved intermittently in increments of the tie spacing in order to obtain vertical deflection readings. Readings were first recorded when the scale car was sufficiently uptrack to cause only slightly perceptable deflection movements. Conversely, readings of the dial gages were continuously taken until no deformations in the section under study were discernable as the scale car moved downtrack.

The calculated track modulus for this study is that which is analogous to the modulus of elasticity of a foundation. The expression used was:

$$k = \frac{P}{S \sum_{n=0}^{\infty} y_{j \pm n}}$$

where

k = track modulus, force per unit of deflection per unit of length of track

P = force of scale car on one rail (50,000 lb.)

s = tie spacing

$y_{j \pm n}$ = the deflection of the $(j \pm n)^{th}$ tie

Using the matrix of deflection values, the calculated values of track modulus were:

Concrete Tie Section:	k	=	9810 lb/in/in
Timber Tie Section:	k	=	3680 lb/in/in

7. CONCLUDING COMMENTS

The results of the laboratory and field studies reported herein and conclusions reached by others [5] corroborate the belief that prestressed concrete stranded ties and related fastening systems can provide satisfactory service in main line track systems.

However, further studies on concrete tie response under track loading conditions are essential toward a better understanding of some of the unknowns. Additional field experimental data is needed on dynamic loading, abrasive wear, tie pressure distribution, optimum tie spacing, anchor bolt fatigue, tie pads, and other considerations. A number of the unknowns will be investigated in the current study at the Santa Fe Railroad test-track facility in Kansas, where the installed ties are of the Gerwick RT-7 MK38 design. This project is sponsored by the Federal Railroad Administration of the Department of Transportation.

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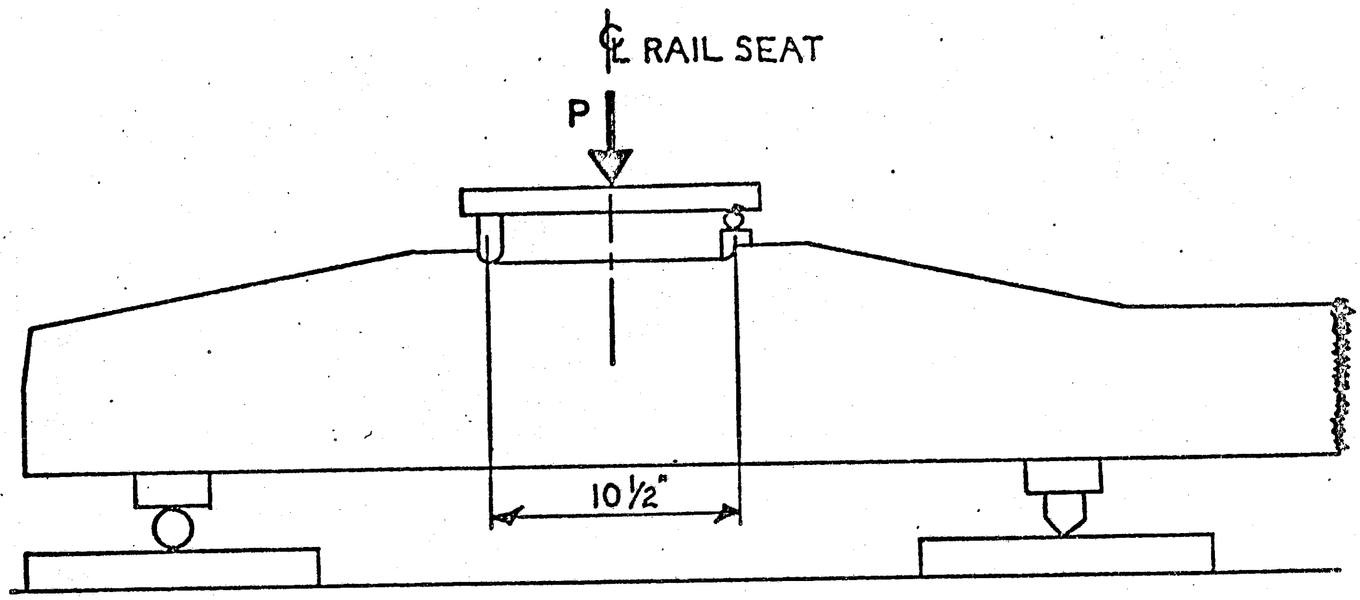
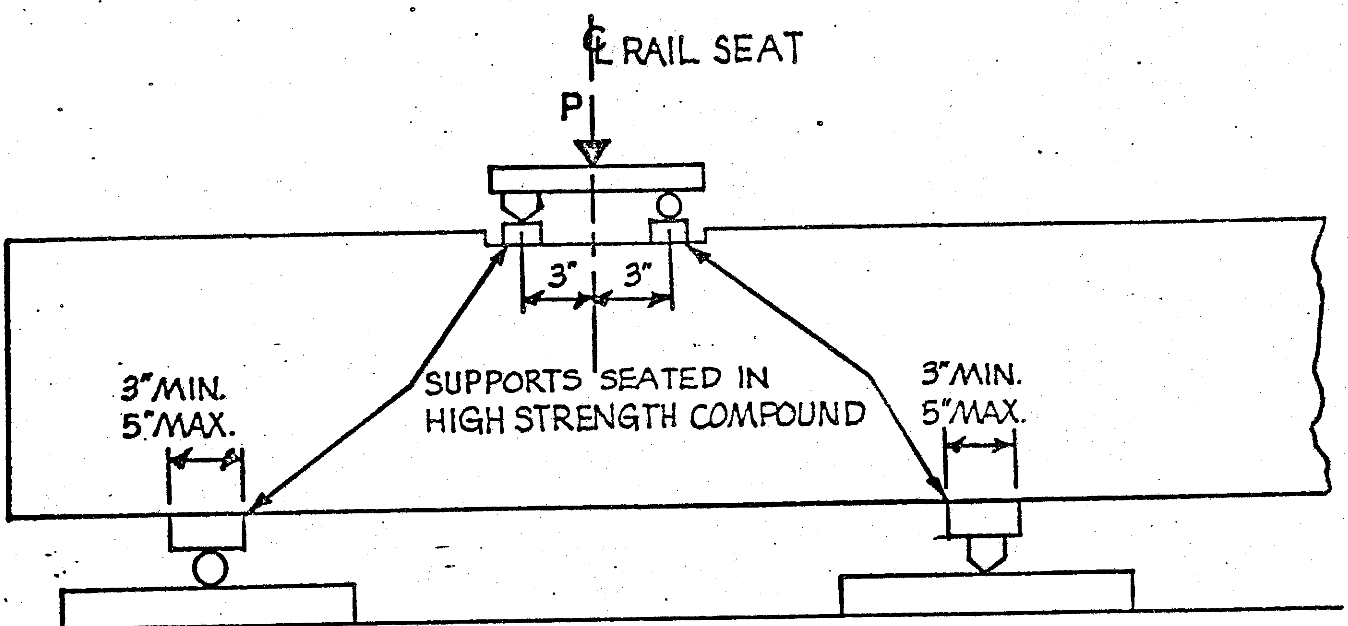
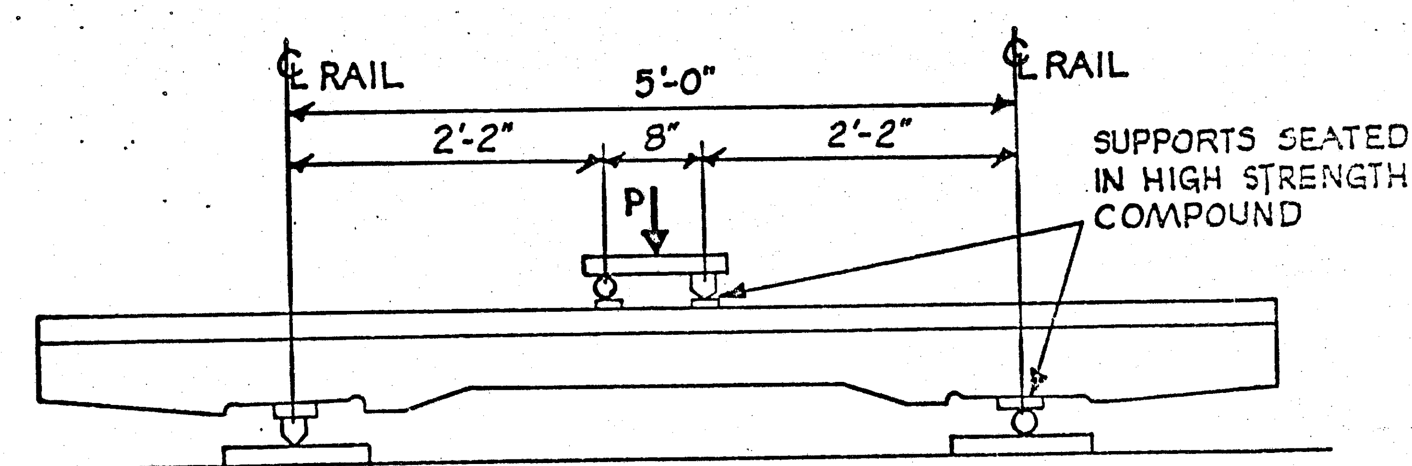

℄ RAIL SEAT
P
10 1/2"

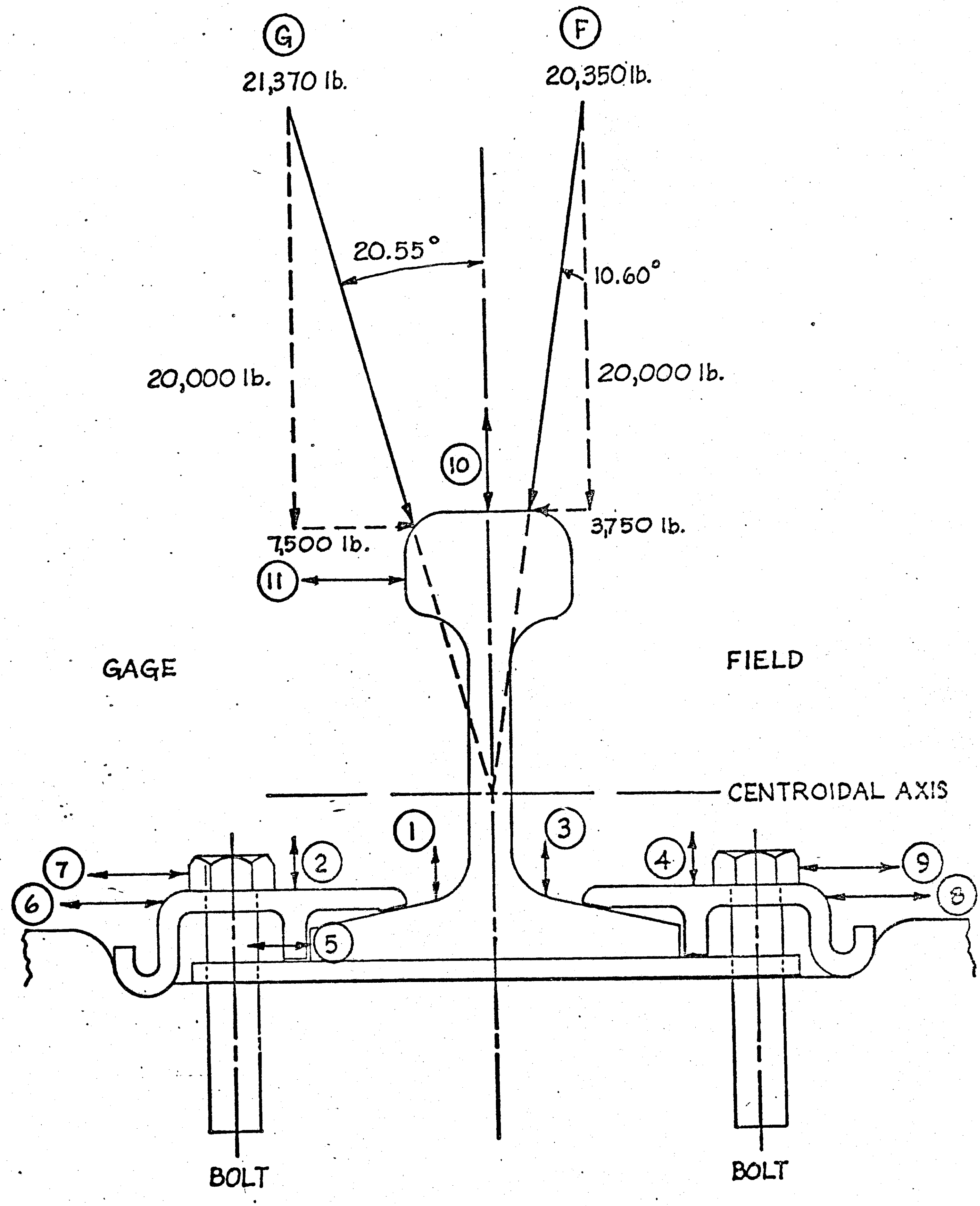
(A)


℄ RAIL SEAT
P
3"
3"
3" MIN.
5" MAX.
SUPPORTS SEATED IN
HIGH STRENGTH COMPOUND
3" MIN.
5" MAX.

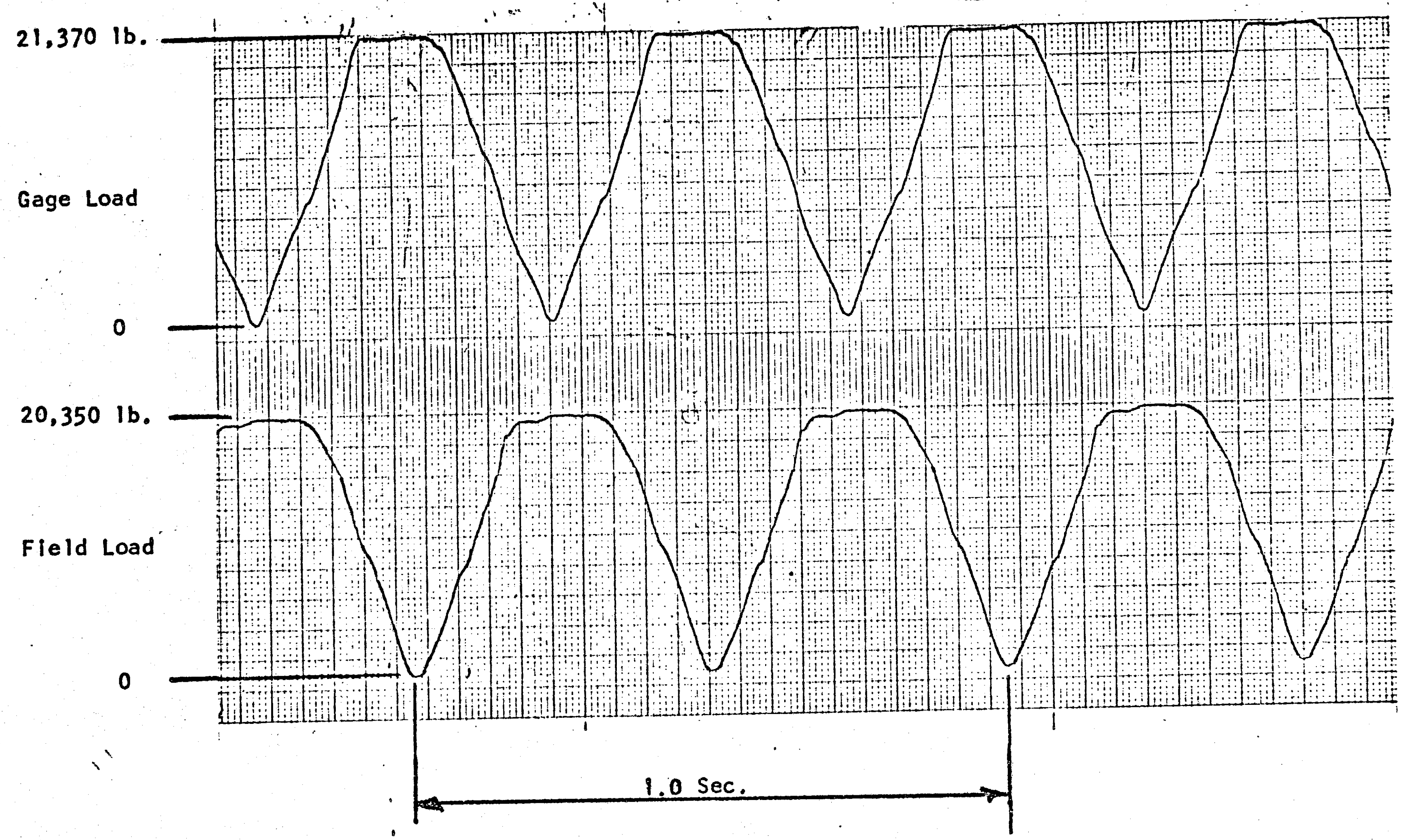
(B)

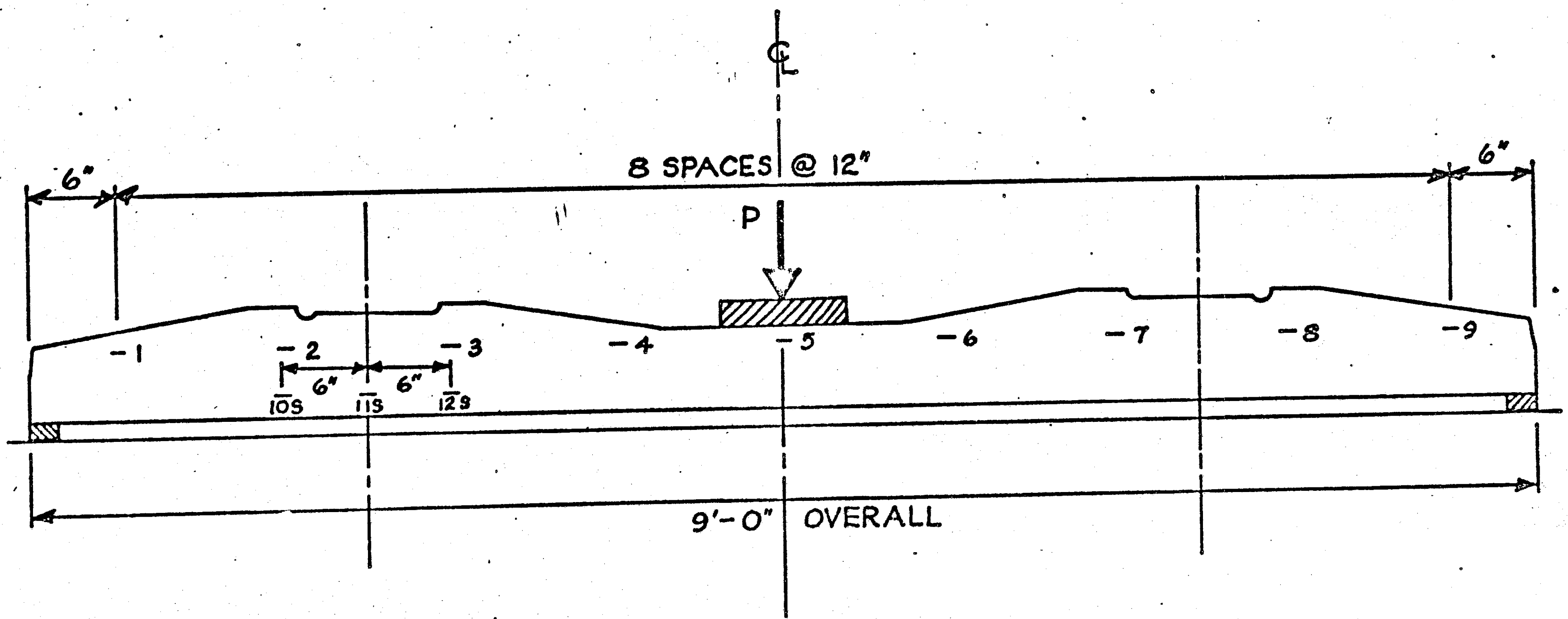

℄ RAIL
℄ RAIL
5'-0"
2'-2"
8"
2'-2"
P
SUPPORTS SEATED
IN HIGH STRENGTH
COMPOUND

(C)

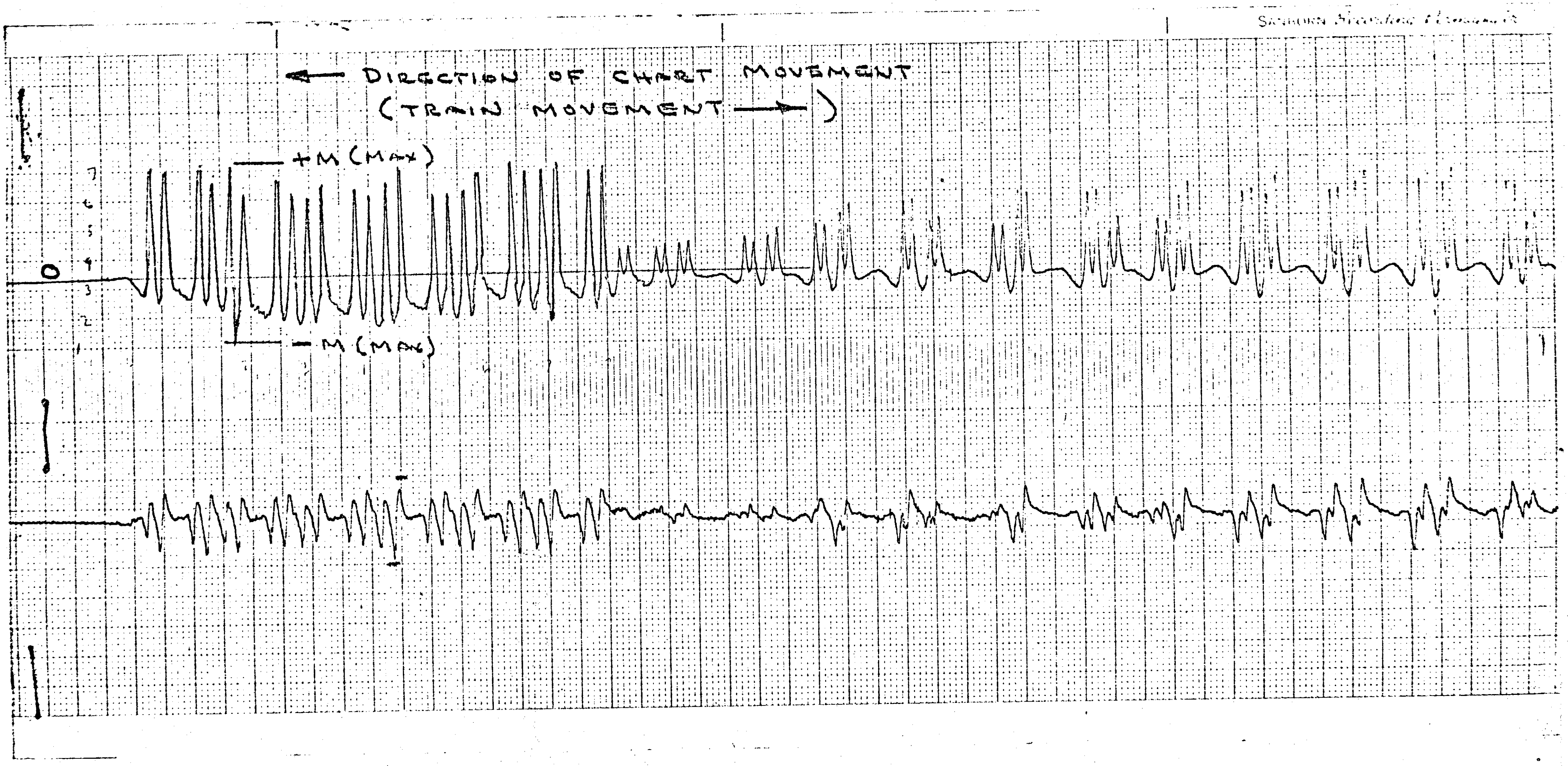


G
21,370 lb.
F
20,350 lb.
20.55°
10.60°
20,000 lb.
20,000 lb.
10
7,500 lb.
3,750 lb.
11
GAGE
FIELD
CENTROIDAL AXIS
1
2
3
4
5
6
7
8
9
BOLT
BOLT

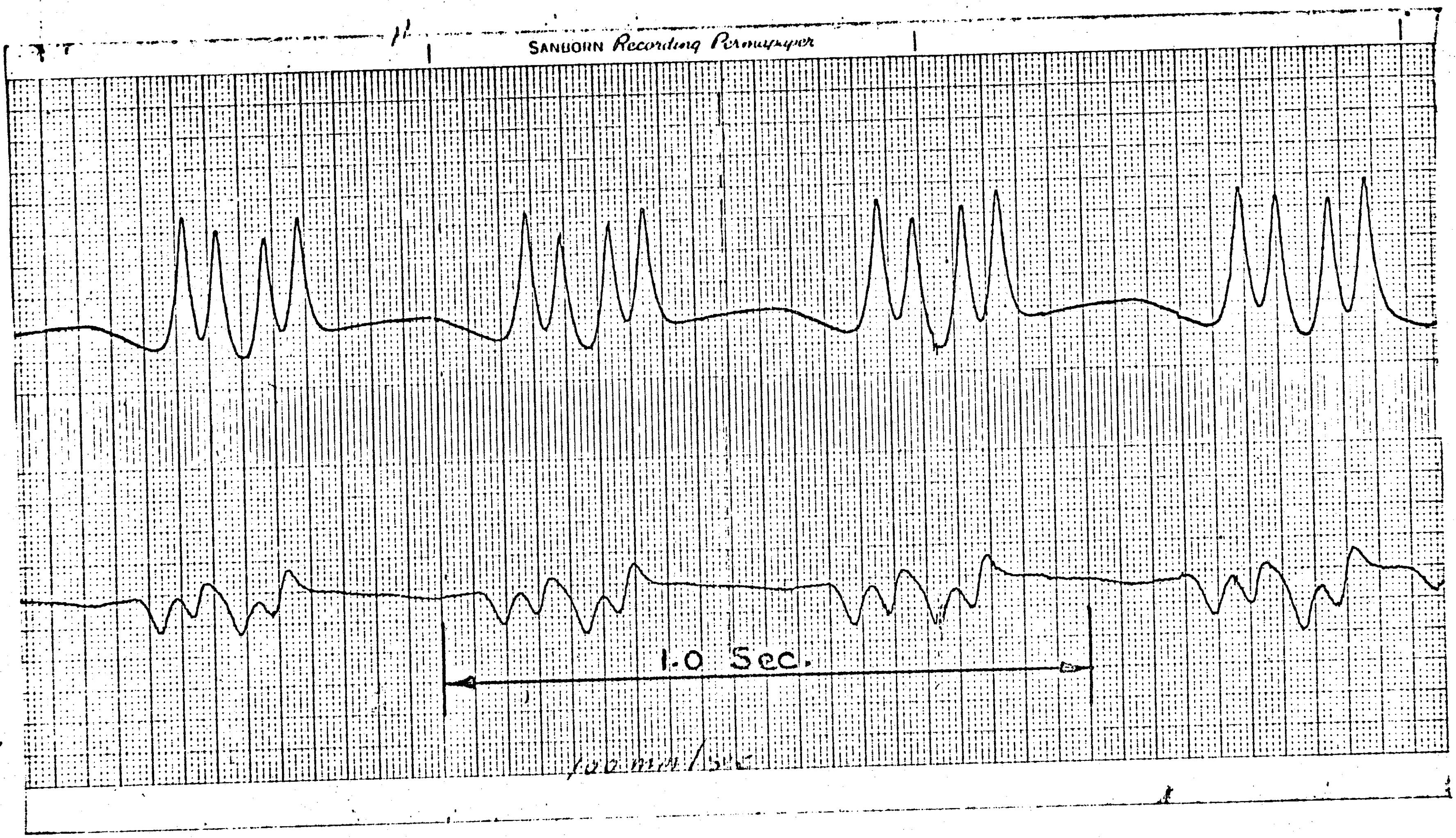

21,370 lb.
Gage Load
0
20,350 lb.
Field Load
0
1.0 Sec.

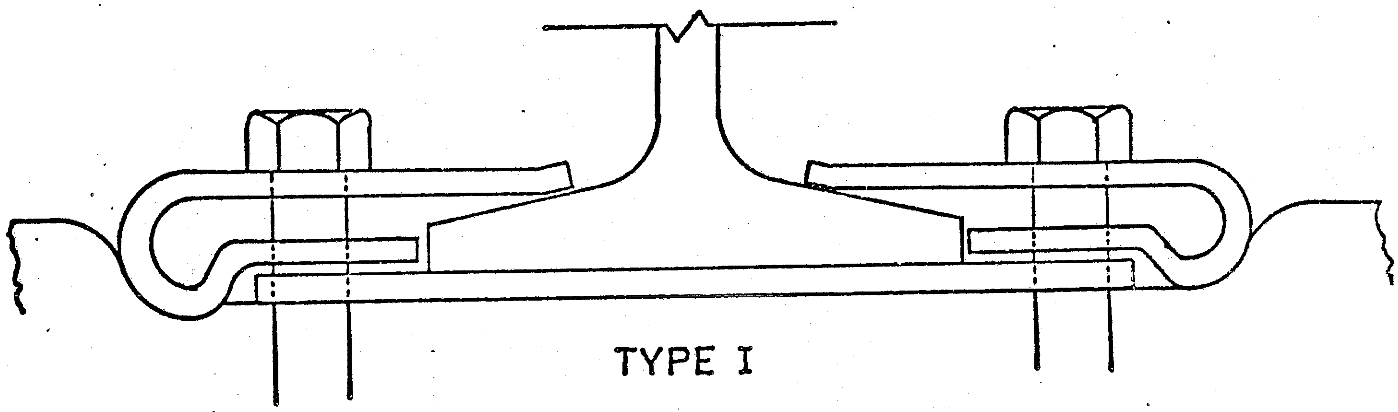


℄
8 SPACES @ 12"
6"
6"
P
-1
-2
-3
-4
-5
-6
-7
-8
-9
6"
6"
10S
11S
12S
9'-0" OVERALL

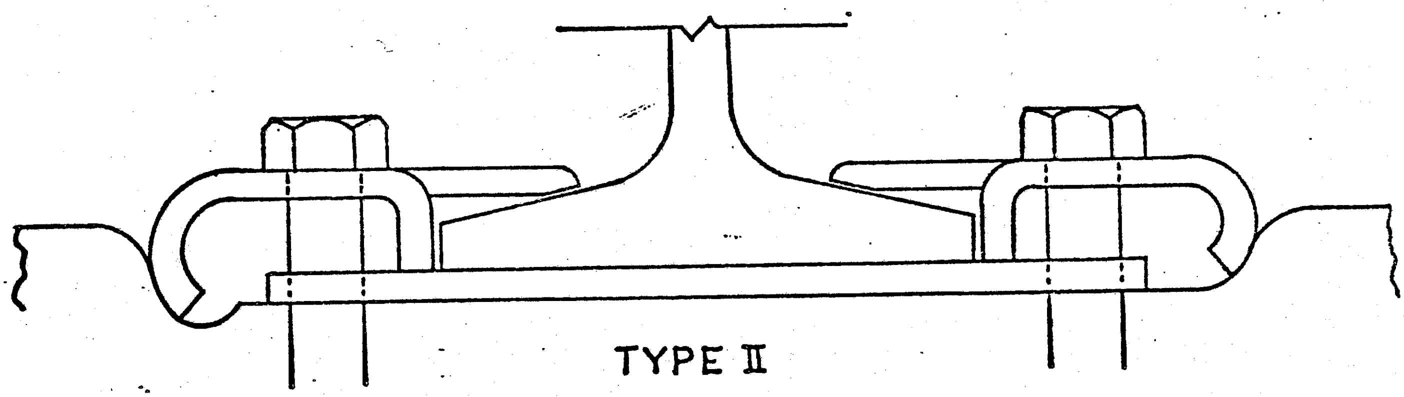


Freight 2^{25} PM
B2- M32 30" SP.
(Tie No. 2)

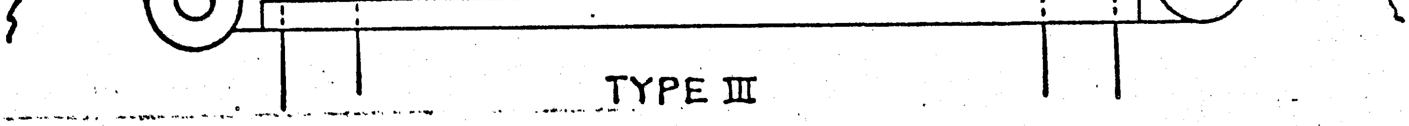

SANBORN Recording Permapaper
1.0 Sec.
100 mm/sec



TYPE I



TYPE II



TYPE III