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THE KANSAS TEST TRACK
NON-CONVENTIONAL TRACK STRUCTURES
DESIGN REPORT

SEPTEMBER 1972

DEPARTMENT OF TRANSPORTATION · UNITED STATES OF AMERICA

DEPARTMENT OF TRANSPORTATION
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16. Abstract

This report discusses the design of three non-conventional Railroad track support structures. These non-conventional structures, which include a continuously reinforced concrete slab, twin cast-in-place concrete beams and twin precast concrete beams, are part of a research program to develop practical, low maintenence, high quality track structures for conventional and advanced rail vehicles. Included in this report are discussions of:

Basic structural concepts, design methods and models, and recommended construction, inspection and maintenance techniques.

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1. INTRODUCTION

This report discusses the rationale for the use of certain design procedures and concepts, and the development of specific values and constants used in that design, for three non-conventional track structure systems to be included as parts of the approximate 2 mile length of test track to be constructed jointly by the U.S. Department of Transportation and the Atchison, Topeka and Santa Fe Railway Company between Aikman and Chelsea, Kansas, located as shown on Sheet 1 of the Design Plans, contained in Section 10.

The three non-conventional track support structures, each 800 feet in length, are: twin cast-in-place continuous reinforced concrete beams joined by gaging members; a continuously reinforced concrete slab; and a system of twin precast beams made continuous at abutting ends and joined by gaging members. These systems will be integral parts of the Santa Fe's single track main line between Aikman and Chelsea, Kansas, and will carry both eastward and westward trains approaching 50,000,000 gross tons annually.

Thus, the systems will be subjected to rigorous service, and it is anticipated that significant results for future use will be produced through observation, inspection, and instrumentation of the subgrade, concrete, reinforcement, and rail fasteners. Obviously, movement of such volume of conventional high and low speed trains subject to all the problems of an operating railroad requires that all work be planned and performed in a climate of positive safety consciousness.

As part of the research program related to developing practical, low maintenance, high quality track structures to accommodate high speed rail transportation, the Department of Transportation engaged the Battelle Memorial Institute to conduct a study designed to produce data for use in determining performance criteria and specifications for practical high speed track structures. Battelle performed theoretical analyses and made a number of field measurements of conventional track supporting moving conventional rail traffic. The analyses and tests showed that distinct subgrade pressure impulses are imparted by every passing axle load and that subgrade deterioration under certain conditions is directly related to the number of transmitted impulses. As a result of their study, it was concluded that track support structures with a stiffness of 4 x 10^{10} lb-in^2 would impart only one impulse per truck (2 axles) and would therefore materially reduce an important cause of subgrade deterioration.

Subsequently, the Department of Transportation, through the Santa Fe Railway, constructed a new embankment, designed and supervised by Shannon and Wilson, Inc., under closely controlled conditions, to produce a uniform subgrade for testing purposes. The Department of Transportation, Federal Railway Administration, authorized the Santa Fe to construct a number of different types of conventional track for test purposes, and authorized the Santa Fe to engage Westenhoff and Novick, Inc., to design certain non-conventional track structures within a set of constraints contained in the Battelle Specifications, and additional parameters stated by the Department of Transportation.

In general, once the goal of a system is established in terms of space, size, function, or esthetics, rational design of the system follows a definite pattern. First, and usually as an input to the goal establishing procedure, background and performance data are determined and analyzed. Then a method of analysis, which is consistent with state-of-the-art or established practice, is chosen for use in the design of the system. Input values are then collected, determined, or assumed, and their applicability established. The system is then analyzed and designed. The designed system is subsequently reanalyzed to assure the suitability of its function and its ability to satisfactorily respond to fulfill the original system goal.

For some well defined and documented systems, the whole design sequence may take a relatively short time; while for other systems, where function, analysis methods, or input are not well understood, or are not clearly definable, the procedure becomes one of trial and error, where each step requires definition and analysis in its own right.

The evolution of this project has shown it to be one of the latter systems. Its goal of building a functioning railway track structure is well defined. However, restrictions imposed by hardware, geometry, and performance criteria, and its essentially original nature, have rendered it to be unique.

This design report presents considerations of concepts, method, and hardware which have been used in reaching the fulfillment of the project goal.

2. AUTHORITY

By agreement dated June 7, 1971, Westenhoff and Novick, Inc., undertook the design and construction engineering for three non-conventional track support systems for the Santa Fe's test track project between Aikman and Chelsea, Kansas, under the sponsorship of the Department of Transportation, Federal Railroad Administration. The agreement was amended December 28, 1971, to provide that additional data be developed by Westenhoff and Novick, Inc.

3. OBJECTIVE AND SCOPE

3.1 Objectives

The purpose of this installation is to provide a facility which will permit the study of a series of railroad track support systems, each of which is a departure from conventional track construction. The test track structures will carry a heavy volume of conventional high and low speed trains subject to all the problems of an operating railroad, so that special care needs to be exercised at all times to provide safe rail traffic movement. All work should be planned and performed in a climate of positive safety conciousness. Recognition, evaluation and accommodation of practical railroad operating problems and peculiarities is required.

3.2 Scope

The initial agreement covered engineering services for the design and construction engineering for three non-conventional track structure systems, each 800 feet long, to be included as part of the test track to be constructed on Santa Fe property between Aikman and Chelsea, Kansas under the sponsorship of the U.S. Department of Transportation, Federal Railroad Administration.

The track structure systems, including the conventional systems and non-conventional systems, will be built on a embankment previously constructed under close control to provide uniform subgrade conditions over the length of the test section. The scope of Westenhoff and Novick, Inc. work included: plate bearing tests of the as-built embankment, complete with compacted ballast; analysis, design and submission of a brief design report for the three non-conventional track systems, including transition section designs where appropriate; preparation of required contract plans and documents; construction engineering services and control; and preparation of a post construction report. The track support structures which were designed include: a continously reinforced concrete slab; twin-cast-in-place reinforced concrete beams; and twin precast, conventionally reinforced beams.

For the foregoing designs, Santa Fe was required to provide complete details of properties of the as-built test embankment; recommendations for structural configurations and reinforcement based on recommendations of the Battelle Institute and the Portland Cement Association; complete geometric details and load transfer characteristics of the specified rail fastener; and complete details of the instrumentation to be used for the project.

The amendment dated December 28, 1971, made certain changes in the initial agreement, including the authorization for Westenhoff and Novick, Inc., to develop certain data and information orginally to have been furnished by the Santa Fe. The development of this data and information included:

(1) Rail torsional constant
(2) Fastener restraint characteristics
(3) Embankment properties

In addition, the amendment authorized the determination of ballast shear values and the analysis of structural behavior in response to adverse support conditions (i.e., subgrade softening).

Instrumentation for the embankment and structures including location, hardware, and installation, are provided by others and do not fall within the Westenhoff and Novick, Inc., scope. In addition, DOT excluded the fastener, and development thereof, from the initial agreement and the structural stiffness recommendation made by Battelle was specified as being rigid criteria for design.

4. HISTORICAL AND RECENT DEVELOPMENTS

4.1 General

The conventional track structure of today is the culmination of almost a century and a half of evolution imposed by the unrelenting growth in the weights, gross tonnages and speeds of rail traffic. During this development a multitude of innovations, many of them rational and well-conceived, have been exposed to service, but natural selection has preserved only those few elements which have survived the rigorous functional requirements.

Today's economic and transportation demands reveal certain practical limits and deficiencies in the conventional track structure. Relative maintenance cost, while lower than only a few years ago, is still too high, and will become even higher for tracks required to accommodate high speed traffic. Thus, evolution inexorably develops a generation of track support structures to fulfill the need.

From time to time, over many years, effort has been made toward developing economical systems of track support structures that would eliminate or materially reduce the attention and maintenance required by conventional track. These efforts have had little more than nominal success, except for special applications, because of high construction costs and, until recently, the lack of an adequate degree of elasticity in the track structure and inadequate all-weather electrical resistance between rails.

4.2 Historical Developments

As early as 1909 the New York Central used reinforced concrete slabs on unstable subgrade to support conventional ballasted track; between 1914 and 1916 the Long Island Railroad used this same technique at three locations. These installations are believed to have been successful for the special purposes intended.

In 1914 the Northern Pacific Railway experimentally constructed three sections of non-conventional track, comprised of concrete slabs on a ballast bed, with the rails secured to wood blocks set in or on the concrete. Considerable detail regarding the sectional configurations and wood block installation is available. These systems were in service for at least 15 years, but it was reported that timber renewals and line and surface maintenance costs were disproportionately high. A number of other railroads experimented with the concept of a rigid track support, but success was marginal at best.

In 1926 the Pere Marquette Railway built an initial non-conventional track support system 1326 feet long, comprised of cast-in-place slabs about 10 feet wide by 20 inches deep, with 39 feet between expansion joints. The reinforcement arrangement involved the use of structural sections instead of conventional reinforcing bars. Ninety lb/yd rail was used, and was fastened by means of rail clips bolted directly into concrete inserts. Part of the length of rail rested directly on the concrete, the remainder rested on a thin fiber pad (about 1/4 inch thick) between the rail base and the concrete.

The Pere Marquette built a second system, 390 feet long, in 1929, consisting of cast-in-place beams about 24 inches wide by 14 inches deep resting on a 6 inch thick slab which was 9 feet wide. Concrete diaphragms on about 6 feet spacing were cast between the beams. The entire assembly was apparently made an integral unit through the use of conventional reinforcing bars and was cast monolithically in lengths of 19 feet 6 inches between joints. The ends of the units were keyed, and they were provided with lifting stirrups for use if unequal settlement required adjustments. Rails were 90 lb/yd, and were fastened by means of bolts threaded through protruding steel plates anchored in the concrete. The rails rested on one thickness of 7/8 inch treated lumber set in grooves each about 4 3/4 inches wide and 3/4 inch deep in the concrete, running longitudinally directly under the rails. The space between the beams was provided with lateral drainage outlets and was filled with ballast stone. The shoulders on both field sides of the beams were filled with ballast to approximate the normal ballast shoulder.

In August 1971 reliable employees of the former Pere Marquette (now C&O - B&O), selected for their knowledge of the experiment, were interviewed in an effort to develop information that normally would not appear in technical journals, but which would be fundamental in considering new non-conventional track systems.

The foreman on the section containing the old non-conventional track systems at the time a passenger train derailed on December 17, 1937 (terminating the project), and for several preceding years, was interviewed. He had been present during the derailment clearing operations and had directed part of the work. From this interview it was found that:

1) No perceptible slab or beam settlement was experienced.

2) Lack of sufficient lateral adjustability in the fasteners made the proper maintenance of line and gage very difficult, sometimes impossible.

3) Bolt failures were common, with several occurring each day. Daily walking patrol of systems was required. It was difficult to change bolts in the initial system, as the insert had to be removed from the concrete, the old bolt extracted and the insert re-set. Breaks and replacements were not so frequent nor so difficult with the second system. Normally, with either system, bolt failure locations were scattered, with no alarming number of consecutive broken bolts.

4) Signal failures were common, as a result of false shunting due to breakdown of the resistance between the rails, particularly in wet weather.

5) The track systems afforded very hard riding characteristics and rail replacements were required almost annually. Bolted joint rail was

used and the joints were severely battered.

6) The old steam engines, because of the severe hunting (yawing) characteristics and drivewheel pounding (dynamic augment) were notoriously hard on all track elements.

An engineering officer in responsible charge of maintenance and engineering advised that although he had no personal familiarity with the experiment, he had discussed it with others who had been in the railroad's engineering department during the time when the non-conventional systems were in service, and he had learned that:

1) Riding quality over the rigid systems was hard and unsatisfactory.

2) Difficulty was experienced in transitioning from conventional track to rigid track and vice versa, with very distinct bumps apparent when aboard fast-moving trains.

3) The derailed train was the third westbound train passing the location within a short period of time. It was surmised that the first train (freight) had broken several bolts, the second train (passenger) had broken several more, and the last train had broken enough additional bolts to allow the rail to be forced over.

4.3 Recent Developments

A number of different so-called "ballastless" track systems are now in limited use, and are being observed to ascertain behavior under service conditions, particularly on new rapid transit lines. Some types, where short ties are supported on concrete, as through railroad tunnels or for other special purposes, are not new and have performed satisfactorily, though perhaps not without their unique problems. Considerable effort has been directed toward developing satisfactory "ballastless" track systems for carrying main line rail freight and passenger traffic in certain other countries, notably, Japan, Germany, Switzerland, England and the Soviet Union. Many concepts and innovations have been tried; for details of these experiences the reader is directed to available publications.

The results produced by these projects have been largely due to observation and practical maintenance attention. Such results have varied widely, as might be expected when it is considered that in addition to the use of differing structural systems, dissimilar subgrade conditions and rail fasteners have added to the number of inherent variables.

Commercially available non-conventional track support systems, which are adaptable to special conditons, were investigated to determine their suitability for adaption for this project. They were eliminated from consideration due to the lack of a convenient method for conversion to continuous structures, which DOT had specified as design criteria.

It is believed that the Kansas Test Track project is the first installation of non-conventional track support systems to be constructed on a subgrade of predetermined uniformity, and fully instrumented, to produce results which, with exercise of experienced engineering judgment, can be rationally applied to future development efforts.

5. BASIC STRUCTURAL CONCEPTS AND GOALS

5.1 General

The purpose of including the continuous reinforced concrete track support structures in the test project is to allow an assessment of the installation, and performance, and maintenance costs for "rigid" track systems. From an operational viewpoint the advantage of rigid systems is essentially due to the ability of the structure to spread loads over a larger area, thereby reducing both the magnitude and frequency of loading imparted to any one point in the supporting subgrade. A conceptual development of this response is contained in the Battelle reports.

Structurally, a rigid track support system may be developed using a multitude of configurations and materials. However, the use of reinforced concrete appears to be economically attractive in initial cost and may result in reduced maintenance expenses. The future development of continuous, automated, construction methods (similar to slip forming pavements or the use of precast units made continuous by post-tensioning in the field) may realize significant economies in construction costs. First costs as formulated by this project will serve as an upper bound estimate for the three systems used. It should be realized that removing the influence of the variables associated with the embankment and track fastener from this project has resulted in additional costs which would not be experienced in ordinary construction. Unusual costs are also being incurred as a result of inclusion of instrumentation in the construction.

In order to provide a uniform load spread capability in the structures it is necessary to attain continuous shear and moment transfer. This is required for two reason: (1) articulated support structures would give rise to localized stress and deflection concentrations in the rail, particularly with relatively stiff direct fixation fasteners; and (2) localized areas of relatively higher subgrade stresses would be created at structural discontinuities, leading to maintenance problems related to subgrade movements.

While the analysis of infinitely continuous structures poses no unusual difficulties, the relatively short lengths of structure used for this project impose significant boundary conditions on the analysis, requiring the consideration of more complex conditions. In a practical sense, the ends of structures will receive impact loadings higher than an infinitely long structure, and large deflections will be realized near the ends. Also, higher negative moments (concave downward curvature) will be induced longitudinally by these loadings. Transitions, as discussed in Section 5.7, have been designed to attenuate these influences.

5.2 Stiffness

As stated in the Battelle Report, it is desirable to obtain a "single peak" response of soil pressure per truck, rather than a load pulse under each wheel of the truck. This effectively reduces the cycles of loading on the embankment by one-half. A practical way of achieving this reduction is to provide a track structure with sufficient stiffness to spread the wheel loads over the underlying embankment.

Battelle determined that structural stiffness, EI, "values as low as 2 to 4×10^{10} lb-in^2 per track structure might be acceptable."

This is an extremely important decision since the moment of inertia for any given section is much greater if that section does not crack (the difference being between multiplicative factors of two to four for the reinforced sections under consideration). Thus if an EI of 4×10^{10} lb-in^2 is calculated on the basis of a cracked section, the effective EI may be more in the order of 16×10^{10} lb-in^2 if the stress range does not exceed the tensile strength of the concrete. The value of embankment soil modulus (i.e., available support for the structure) is very important, as it has a significant effect on the structure's deflection and will influence the stresses in the concrete. Due to the critical nature of this item, it was discussed repeatedly during the design effort, at which times DOT reaffirmed that they required an EI of 4×10^{10} lb-in^2 to be used for design, and that the value was to be based on a cracked section.

5.3 Continuous Slab Structure

The continuous slab structure will distribute the loading over a large area of subgrade, and will be the most effective structural type for that purpose. Load distribution will take place longitudinally and transversely through the slab as a function of directional relative stiffness and subgrade modulus. The longitudinal stiffness required by DOT can be supplied only by a substantial concrete structure with a relatively large percentage of steel.

The influence of transverse stiffness was not developed during the conceptual study for these systems. However, the magnitude of structure required for longitudinal stiffness, even with minor amounts of temperature steel, will possess significant transverse bending stiffness. Also, the geometry of the fastener and slab aids the structure in transverse distribution of loads by shear as shown conceptually on Figure 5.3.1, which also indicates that transverse curvature of the structure under vertical loading will be concave downward with bending stresses occurring near the longitudinal center line of the slab. An example of this phenomena is the "center binding" tendency of ties, with resultant failure of the tie.

5.4 Continuous Beam Structures

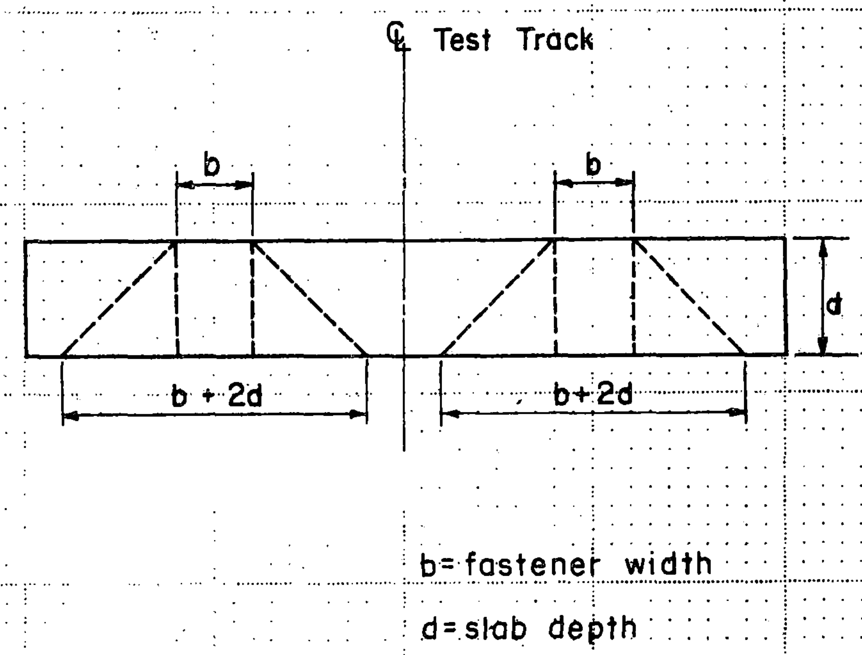
The use of a twin beam structure for track support will result in higher subgrade pressures than would be experienced with a slab. However, material economics may be achieved with this configuration and it may be more applicable to the use of precast units.

The DOT required that each beam be designed having one half of the longitudinal stiffness specified for the slab. This essential sameness of structural stiffness will allow an assessment of the influence of bearing area, and resulting subgrade pressures, on load carrying response and maintenance requirements. No requirements were stated by DOT, nor investigated in the conceptual study, concerning the longitudinal stiffness necessary for the beams to support the bending induced by the lateral wheel-rail loads.

The beams will be subjected to a condition of bi-axial bending, due to the combined load. In addition, torsion, as discussed in a subsequent paragraph, will be superimposed on the above loadings to create a complex state of stress within the beams.

In light of Figure 5.3.1 it may be recognized that the requirement for transverse bending stiffness is bypassed, and that the beams will fundamentally be shear blocks in relation to transverse distribution of loads.

Since the restraint of gage spread under lateral loads is critical, the use of gaging members is essential. The members will aid the structure in resisting



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GENERAL TRANSVERSE SLAB GEOMETRY

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FIGURE 5.3.1

lateral loads by bringing the full beam-ballast shear interface of the structure into play, thereby helping to alleviate a structural inefficiency, relative to the slab. The use of gage members is considered to be a necessary, conservative feature of this project.

Under lateral loads,the single support beam will tend to roll to the field side. This rolling tendency will impart torsional stresses to the member, a consideration whose range of influence was not investigated in the conceptual studies, and will result in a triangular or trapezoidal distribution of soil pressure under the beams. It will also contribute to gage spread. The gage members will aid in resisting this tendency.

The response of reinforced concrete under torsional loadings is relatively poor and the usual practice is to design in a conservative manner. This is due in part to the lack of rigorous solutions for non-circular section response under torsional loadings, and to the composite nature of the structure itself. Until the advent of ACI Standard 318-71, no widely recognized, rational design method existed for combined bending, shear, and torsional loads. Since this criterion became available early in 1972, it was selected for design for these combined loadings.

Cast-in-place beams will have the advantage of a "natural" interface bond between the concrete and the ballast subgrade, while precast beams will require field alignment, will need "bedding" on the ballast (an artificial interface bond), and will require field joints to be made by welding reinforcing bars and casting a concrete segment.

5.5 Gage Members

Gage members are required for the beam structures as discussed in the previous section. Their design requires the optimization of several variables in order to: 1) minimize gage spread; 2) extract maximum resistance of the complete structure to laterial loads; 3) reduce rolling tendency of the laterally loaded beam; 4) accomplish the foregoing without imparting any undesirable secondary loading to the beams.

Since the primary structural function of the gage members will be to reduce subgrade pressures and structural stresses associated with torsion, they will be required to supply a moment to the beam to resist the overturning tendency. This may be accomplished in two ways: 1) design the members to have a high order of bending stiffness, or 2) position a relatively flexible member such that induced axial loads will have a moment arm above the center of rotation of the beam. The first of these alternates appears unattractive for several reasons, and was dismissed for the reasons described below.

If a gage member were designed with significant bending properties it would become a major structural element which is subjected to repeated loadings and stress reversals. This would require design to withstand a severe fatigue conditon, and the member would require extensive design, detail, and construction work. Even with flexible members the effect of stress reversals will be significant.

In addition, it would be desirable to leave the member uncovered for inspection purposes, and to shield it from contact with the ballast in order to circumvent the phenomena which leads to center-binding of ties, which might introduce undesirable secondary loadings to the beams. This type of member was not considered practical nor consistent with the goals of the project.

The second alternative appeared most in line with the project goals in the sense that no additional variables nor significant secondary conditions would be introduced. An idealized member of this type would be a rod (or chain) fastened to the base of rail by a ball (or universal) joint. This would give the longest possible moment arm about the center of rotation and directly inhibit gage spread of the rails. Since it was not practical to attach the member to the rail or the fastener, a logical second position, it was established that it would be attached to the beam at the highest feasible location. A rod passing through the beams above the midplane was selected as the type of member to use.

In light of the brief treatment afforded the gage members during the conceptual study and the simple nature of the designed element, the gage members would yield significant information about their actual function if instrumented, a measure not considered prior to the design analysis.

5.6 Transitions

The use of non-conventional track structures gives rise to a problem commonly encountered in conventional rail systems at crossings and bridge abutments, that of abrupt changes in track embankment stiffness (impedance). These areas are sometimes quite noticeable to passengers and create severe loading conditions on rolling stock. In addition, these locations require frequent maintenance and are a continuing source of trouble.

The introduction of transients into the test sections is highly undesirable in that it creates unusual loading conditions, in terms of infinitely long track structures, and perturbates the rolling stock such that loadings and structural responses in the location of the instrument arrays may be affected.

To minimize this influence, transition sections which will give a continuous rather than discontinuous stiffness (impedance) change were developed as described in Section 8.5.

In-service observations of the transitions included in this project should indicate their adaptability to other practical situations.

6. DESIGN ANALYSIS

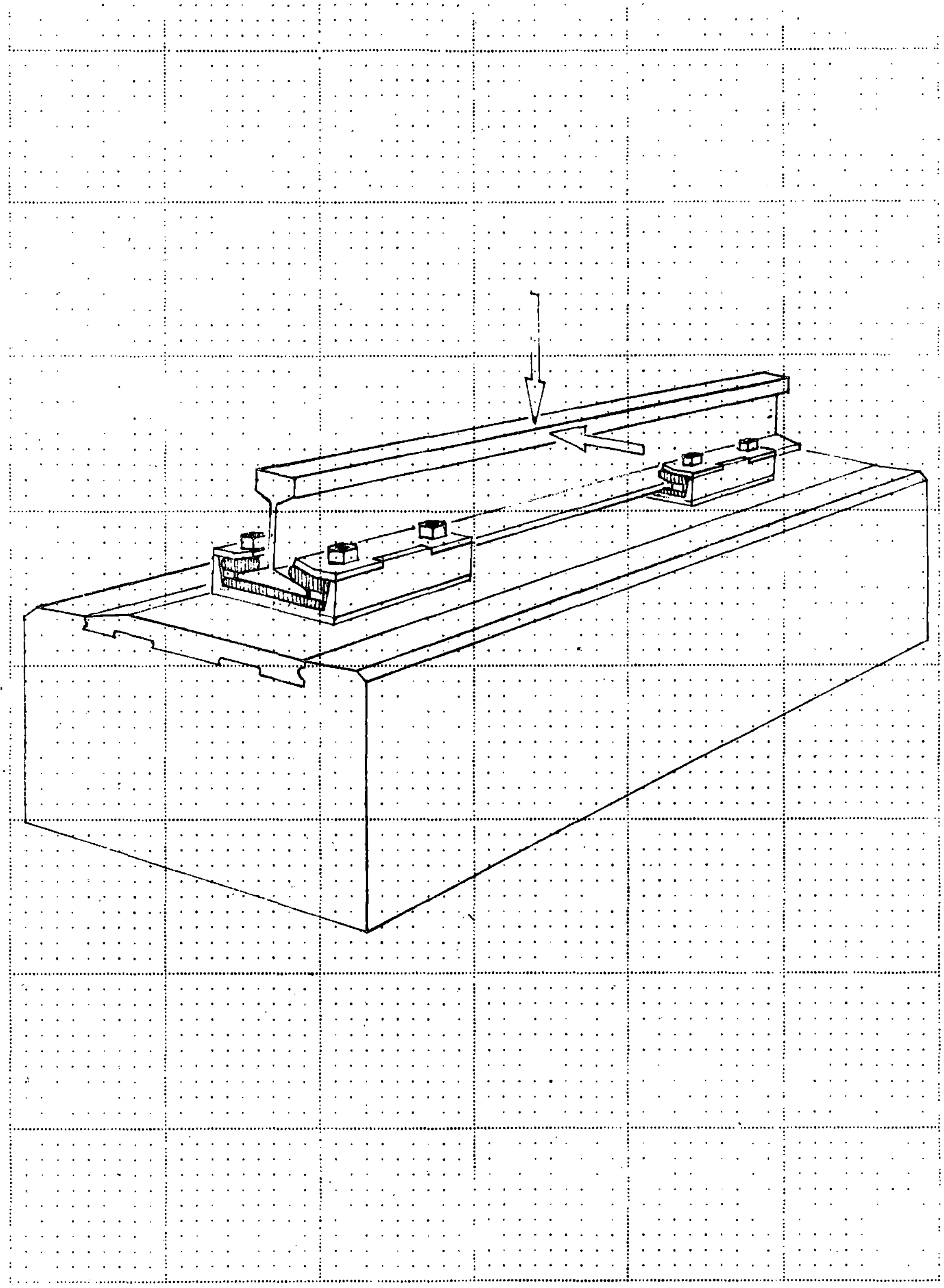
6.1 Methods

Although a specified longitudinal stiffness was desired by the DOT (4×10^{10} lb-in^2) the influence of several additional variables needed to be taken into consideration in analysis. The torsional stiffness and shear requirements due to lateral loads on the beam structure, as well as the gaging member requirements under the same loading, had not been previously defined. In addition the three dimensional nature of the structures, as evidenced by transverse bending stiffness in the slab and non-uniform distributions of contact pressure across the transverse sections, had not been analyzed. These considerations were further complicated by the finite nature of each test section.

The finite element method was selected as being appropriate to the demands of the analysis. The general capability of the method to describe complex structural interaction and boundary conditions, including the three-dimensional nature of the structure, was considered essential to fulfill the goal of the project. Contemporary texts are available which describe the method. The finite element method results in a lower bound solution for this analysis.

Two programs which are commercially available were used in the course of the analysis and design. EASE and NASTRAN, as implemented on the Control Data Corporation Cybernet System, were used for certain selected simplified influence analyses and more explicit influence and design analyses, respectively. Both software packages were executed on a CDC 6600 series computer. EASE documentation is available from CDC, while NASTRAN documentation may be purchased from COSMIC at Athens, Georgia.

The rail, fastener, slab or beams with gage members, and subgrade were each considered by use of an appropriate structural element in the finite element models subsequently described herein. Each will be treated in a separate sub-section,



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RAIL-FASTENER INTERACTION

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FIGURE 6.2.1

6.2 Rail

This project will utilize 136 lb/yd, continuous welded rail throughout its entire length. The condition of continuity poses the widely recognized problem of longitudinal restraint of the rail, which imposes certain load carrying requirements on the rail fastener where conventional and non-conventional track systems abut. This condition is certainly not new, but for the hardware and structures of this project, the interaction relation is unknown.

In the reinforced concrete track systems the rail serves to transfer vertical and horizontal loads to the fasteners by virtue of its structural stiffness. This stiffness is represented by a resistance to bending about two axes, and a torsional resistance to lateral loads which are applied at the rail head as shown in Figure 6.2.1. The finite element method is able to take into account the effects of these properties, which were required input data.

The moment of inertia about the vertical and horizontal planes were obtained from the literature and DOT, however, the torsional constant for the rail section was not available.

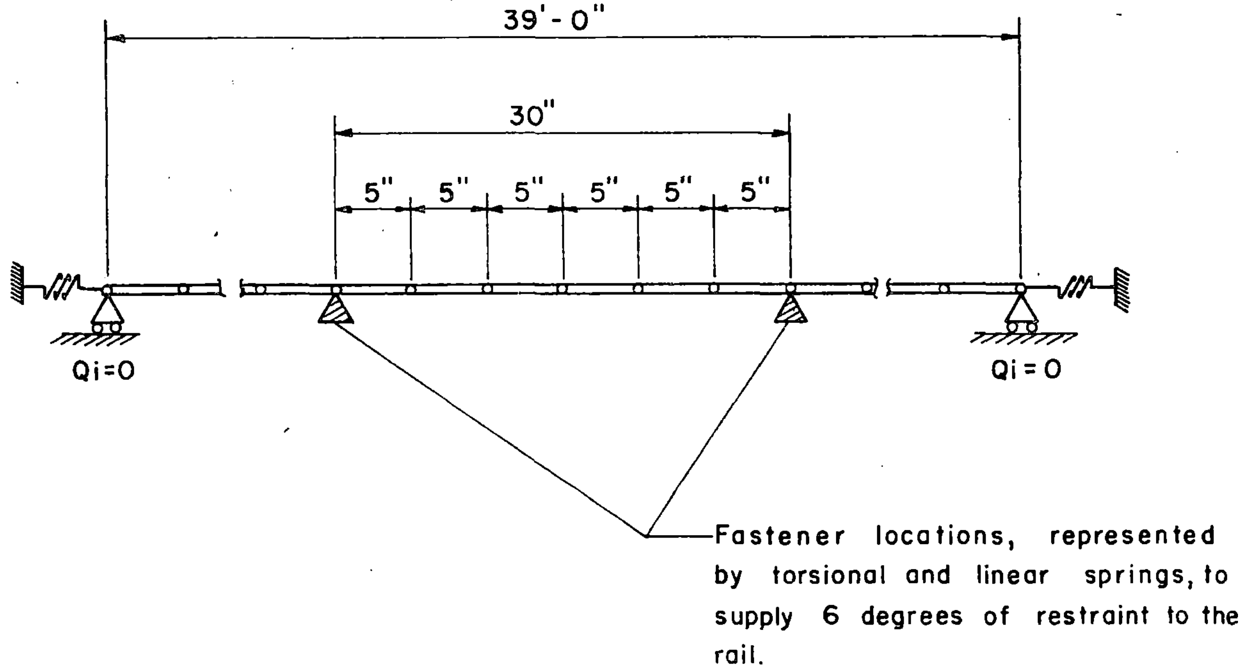
A simple geometrical approximation was used to check the moment of inertia about the vertical plane, while a somewhat different approximation was used to establish a lower bound for the torsional constant. In addition, an upper bound value of torsional constant was calculated and a more exact analogy was then used to establish a design value. These analyses are covered in Appendix A.

A simple finite element model was used to model a rail-fastener system on an infintely rigid structure as shown in Figure 6.2.2. This model was then utilized to assess the influence of load condensation and fastener property variations, as covered in succeeding sections, and to check the variation in load distribution due to rail torsional constants. The results of this analysis are contained in Table 6.2.1. As may be seen, the maximum variation in fastener loading occurs at the fasteners or fastener nearest the load, and is approximately 7% to 10% of the value obtained using the value chosen for design.

The rail was included in the finite element model shown in Figures 6.4.1 and 6.5.1 as a beam element. The single modeling assumption necessary, aside from element length, was that the elastic center, center of rotation, and center of gravity of the rail section are coincident. This is a conventional simplification.

6.3 Fastener

The fastener selected by DOT, and used in the analysis and design of these structures, is the Fastex Fastall shown on Figure 6.3.1. For the analysis,



EASE Rail - Fastener Model

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EASE RAIL - FASTENER MODEL

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FIGURE 6.2.2

TABLE 6.2.1

DISTRIBUTION OF LATERAL LOADS ON 136 lb/yd
RAIL TO FASTENERS - INFINITELY
RIGID SUPPORTING STRUCTURE AND LONGITUDINAL
FASTENER CONSTANT OF 1.4 x 10^6 in-lb/radian

Item	Fastener Overturning Moment, % Applied Moment							Rail Torsional Constant, in^4
	1	2	3	4	5	6	7	
Horizontal Load @ Mid-span	29.2	12.2	5.1	2.1	0.9	0.4	0.2	4.44
	27.0	12.4	5.7	2.6	1.2	0.6	0.3	5.68 (Design)
	25.0	12.5	6.3	3.1	1.6	0.8	0.4	7.24
Horizontal Load @ Fastener	41.2	17.2	7.2	3.0	1.2	0.5	0.2	4.44
	37.1	17.0	7.8	3.6	1.6	0.7	0.3	5.68 (Design)
	33.3	16.7	8.3	4.2	2.1	1.0	0.5	7.24

Kansas Test Track
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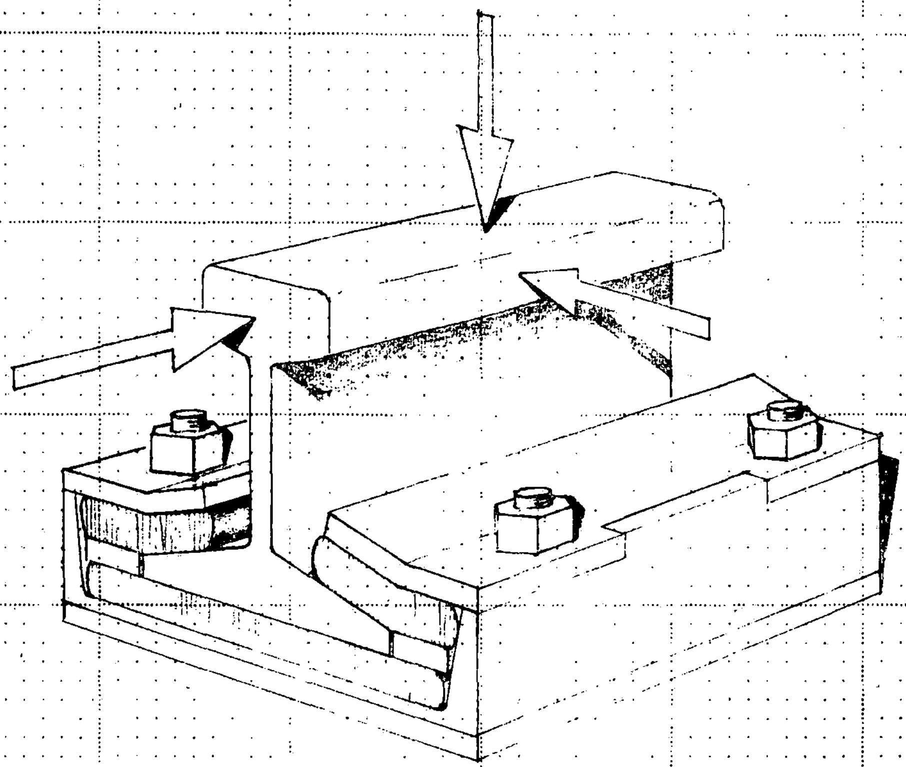
it was considered to have vertical adjustment capabilities of ± 1/2 inch by means of steel shims between the fastener and the structure, and lateral adjustment capabilities of ± 1/2 inch by means of nylon inserts within the fastener proper. Physical dimensions are 13 1/2 inches long and 12 inches wide, with a square pattern of studs on 8.171 inch centers.

The fasteners will be suppled to the job site as a four fastener unit on a single sheet of extruded asbestos-cement. The asbestos-cement is to be shaped such that a 1:40 rail cant may be established by installation, rather than by the fastener. See Sheets 6 and 7 of the Design Plans, and Figure 6.3.2 for installation and drainage configurations. The selected fastener for the non-conventional systems has six mechanical properties which will function to restrain the rail and transfer load into the supporting structure (i.e. 6 degrees of restraint). These restraints are functional along and about all three axes as shown by Figure 6.3.1. The finite element method of analysis is able to account for the effects of these properties, and they were required input data. Since these properties were not known for the specified fastener, Westenhoff and Novick, Inc. recommended that a series of tests be performed to experimentally determine their order of magnitude. These tests were performed for the Department of Transportation by Portland Cement Association during the period of November 2, through November 4, 1971. Descriptive sheets and tabulations compiled by Portland Cement Association are attached as Appendix B. Plots resulting from interpretation and analysis of the data by Westenhoff and Novick are shown on Figure B.1. Specific values of fastener properties which were incorporated into the design analyses are summarized in Table 6.3.1.

It should be noted that the response of the selected fastener is temperature and load rate dependent, due to the elastomers and the design prestress. The order of magnitude of this variation in response was not amenable to quantification for this project. Although beyond the scope of the design, it should be considered in the analysis of resulting test data and subsequent design efforts. Also, the values of properties obtained from the fastener tests are singular in values, which establish a set of values unique to one fastener. Application of these values to design introduces some question of statistical validity into the design. It is felt, however, that they represent lower bound of values of the fastener restraint properties.

Since the fasteners had not been performance, or acceptance, tested at the time of design, DOT relieved Westenhoff and Novick from making an assessment of rail gage spread. However, it became necessary to evaluate the influence of a variation of fastener properties on the actual loads transmitted to the structure by the fasteners. The simple finite element model of Figure 6.2.2 was utilized for this purpose by varying the values of the springs which represented the fastener.



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FASTEX FASTALL TRACK FASTENER

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FIGURE 6.3.1

TABLE 6.3.1

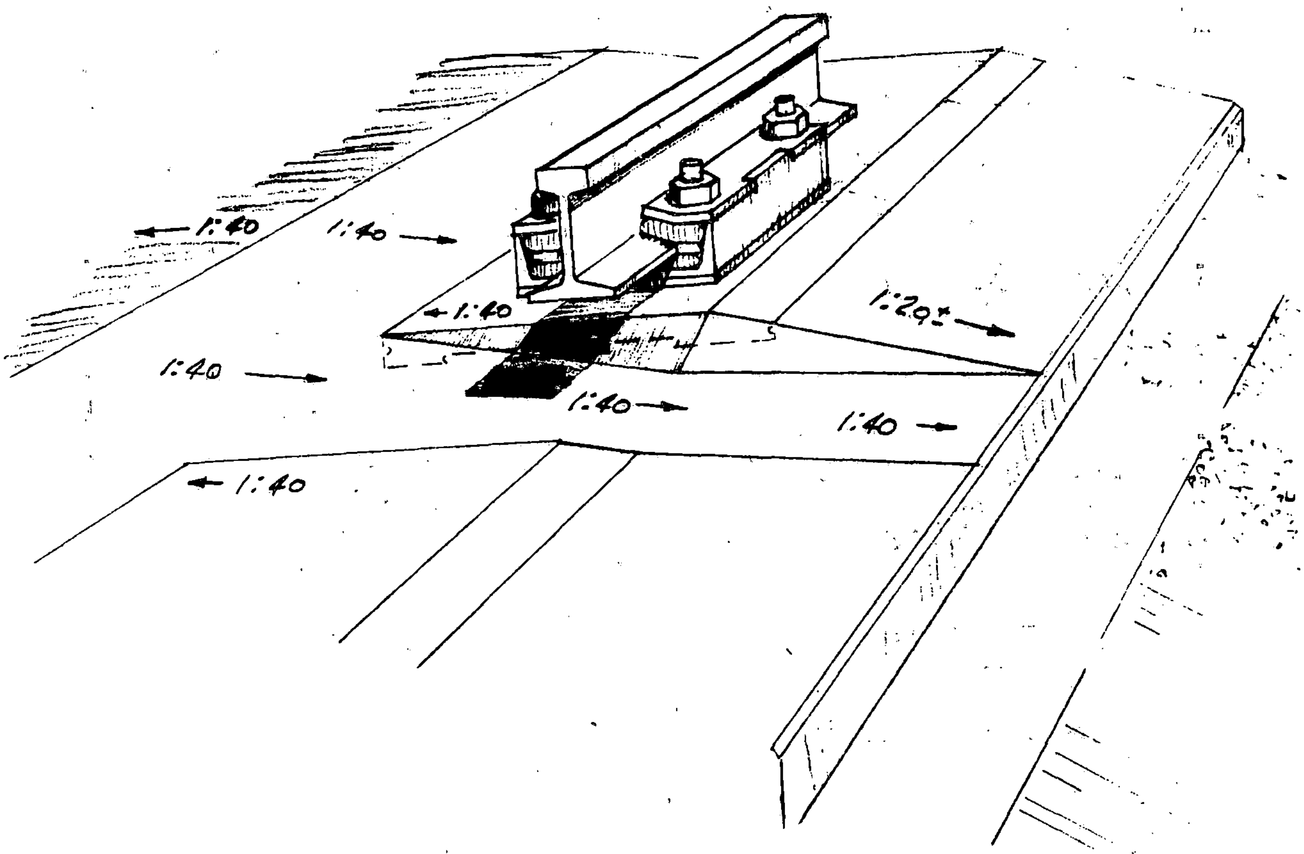
SPRING RATE VALUES FOR

FASTEX TRACK FASTENER

(Tested on November 2-4, 1971)

Property	Value
Vertical Rate	1.0×10^6 lb/in
Longitudinal Rate	1.3×10^4 lb/in
Transverse Rate	2.6×10^4 lb/in
About Vertical	7.4×10^6 in-lb/rad
About Longitudinal	1.4×10^6 in-lb/rad
About Transverse	3.4×10^6 in-lb/rad

Kansas Test Track
Santa Fe/DOT



KANSAS TEST TRACK

SANTA FE / DOT

SLAB DRAINAGE CONFIGURATION

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FIGURE 6.3.2

As a comparison base the fastener properties were arbitrarily doubled. Results of this study are shown in Table 6.3.2. As may be seen, the maximum influence is on the fasteners nearest the load. Percentage change in load applied to the structure by these fasteners is from about 9% to 32%, with the largest variation occurring in response to overturning moments on the rail.

The variation under vertical loads, as expected on a tangent track, is from about 9% to 15% for a 100% increase in properties. A reduction of these values, by load spreading, occurs when the structure deflects under load, further reducing the percentage variation. In this light, the use of the fastener test results appeared adequate for the project.

The fastener is represented in the finite element models by a short column having torsional, flexural and extensional properties derived from the results of the experimental tests of the fastener. The fastener beams are connected to rail and slab nodes, using the NASTRAN offset provision, as shown on Figures 6.4.1 and 6.5.1.

6.4 Slab

Reinforcing was apportioned to the slab in accordance with the DOT specification for a cracked section stiffness of 4 x 10^{10} lb-in^2. This resulted in two different section stiffnesses, one for positive bending and a second for negative bending, since symmetrical reinforcing for the section was considered generally unnecessary and uneconomical. Transverse steel was allocated on the basis of experience, to cope with transverse bending and anticipated construction conditions.

As NASTRAN is not specifically coded to deal with cracked section analysis of reinforced concrete members, the equivalent homogeneous section was calculated for the positive bending stiffness in the longitudinal and transverse directions. The stiffness values and equivalent sections were then used for the quadrilateral plate-bending elements (QUAD2) utilized to model the slab.

The orthotropic nature of the slab was described by defining the material description matrix (MAT2) in line with the solution by Timoshenko and Woinowski-Krieger.

The element size, and aspect ratio, were determined by geometry. Since transverse bending was in question, and the element force output is an average elemental force output for the centroid of the element, a line of elements was set to straddle the longitudinal centerline. To reduce modeling efforts for the beams,

TABLE 6.3.2

DISTRIBUTION OF LOADS ON 136 lb/yd

RAIL TO FASTENERS - INFINITELY

RIGID SUPPORTING STRUCTURE

Load Position	Fastener Load as % of Applied Load* 1	2	3	4	5	6	7	Fastener Stiffness
Vertical Load @ Mid-Span	47.2	6.4	-2.7	-1.0	0.03	0.09	0.01	1×10^6 lb/in
(Line of symm @ left of fastener #1)	51.1	1.4	-2.6	-0.2	0.10	0.01	—+	2×10^6 lb/in
Vertical Load @ Fastener	60.7	22.6	-0.9	-2.0	-0.30	0.11	0.05	1×10^6 lb/in
(Fastener #1 is line of symm)	70.1	18.8	-2.9	-1.1	0.17	0.06	-0.03	2×10^6 lb/in
Horizontal Load @ Mid-Span	32.3	15.3	4.0	-0.1	-0.8	-0.5	-0.2	2.6×10^4 lb/in
	35.5	13.2	2.1	-0.6	-0.5	-0.2	-0.03	5.2×10^4 lb/in
Horizontal Load @ Fastener	36.6	23.7	8.3	1.2	-0.7	-0.7	-0.3	2.6×10^4 lb/in
	42.3	23.5	6.1	0.1	-0.7	-0.3	-0.08	5.2×10^4 lb/in
Overturning Moment @ Mid-Span	27.0	12.4	5.7	2.6	1.2	0.6	0.3	1.4×10^6 in-lb/radian
(i.e. Moment from load on rail-head)	33.8	12.2	3.8	1.3	0.5	0.1	0.3	2.8×10^6 in-lb/radian
Overturning Moment @ Fastener	37.1	17.0	7.8	3.6	1.6	0.7	0.3	1.4×10^6 in-lb/radian
	49.3	16.8	5.7	1.9	0.7	0.2	0.08	2.8×10^6 in-lb/radian

*Negative sign indicates fastener reaction acting in direction of applied load.

+Dash indicates value less than 0.01%

Kansas Test Track
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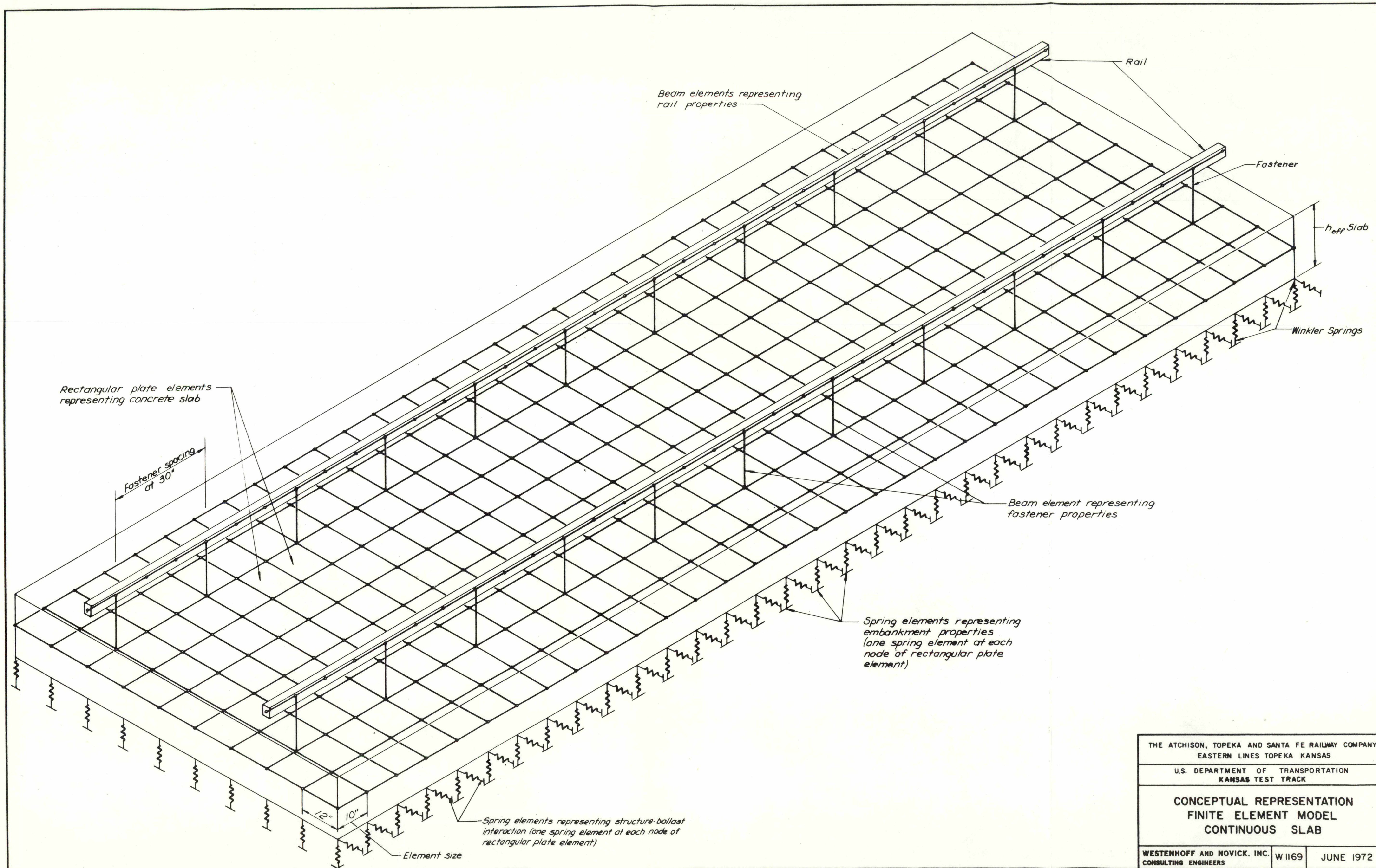

Rail
Beam elements representing rail properties
Fastener
h_{eff} Slab
Winkler Springs
Rectangular plate elements representing concrete slab
Fastener spacing at 30"
Beam element representing fastener properties
Spring elements representing embankment properties (one spring element at each node of rectangular plate element)
12"
10"
Spring elements representing structure-ballast interaction (one spring element at each node of rectangular plate element)
Element size
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KANSAS TEST TRACK
CONCEPTUAL REPRESENTATION
FINITE ELEMENT MODEL
CONTINUOUS SLAB
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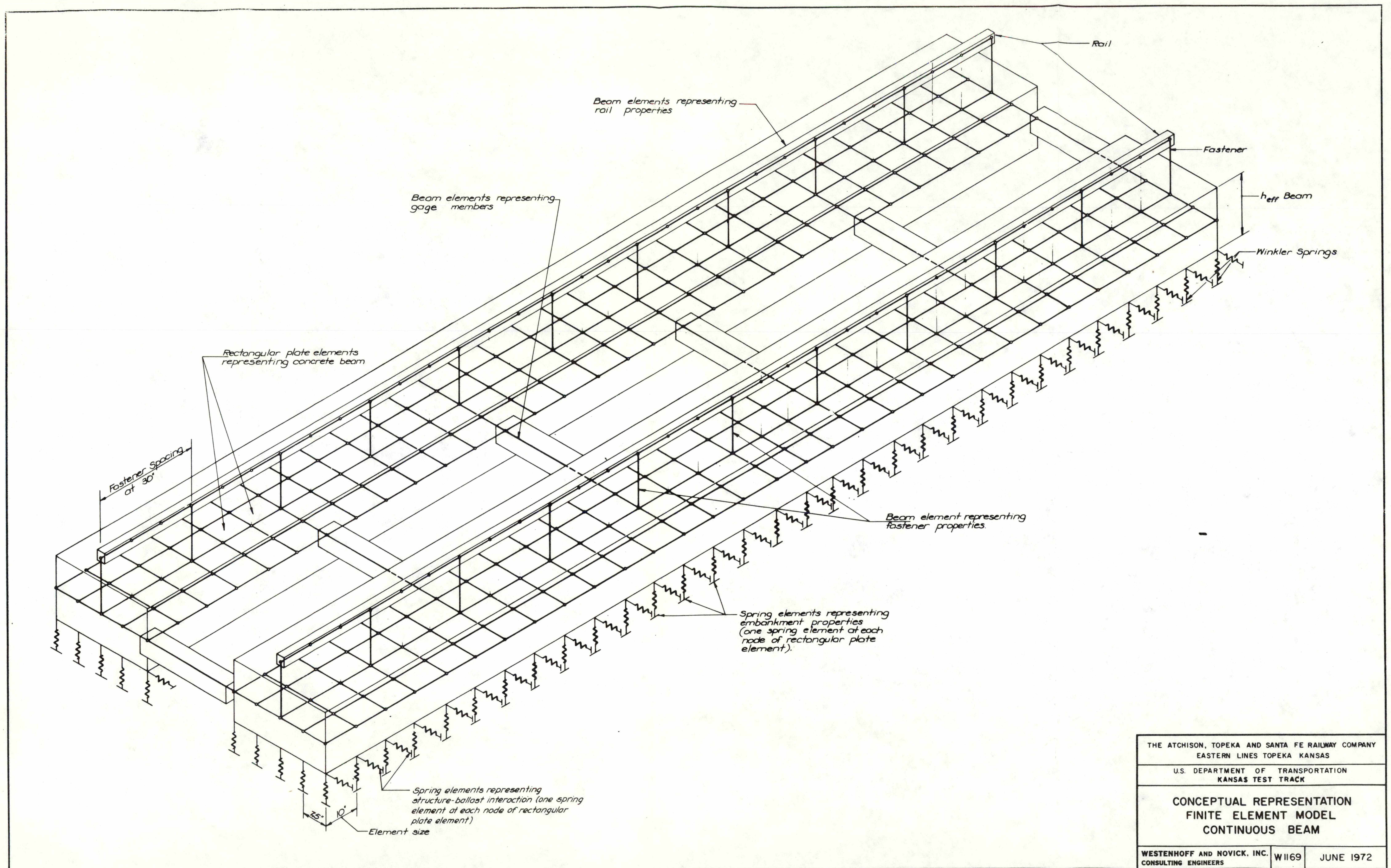
JUNE 1972

FIGURE 6.4.1


Rail
Beam elements representing rail properties
Fastener
h_{eff} Beam
Beam elements representing gage members
Winkler Springs
Rectangular plate elements representing concrete beam
Fastener Spacing at 30"
Beam element representing fastener properties.
Spring elements representing embankment properties (one spring element at each node of rectangular plate element).
Spring elements representing structure-ballast interaction (one spring element at each node of rectangular plate element)
25"
10"
Element size
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FINITE ELEMENT MODEL
CONTINUOUS BEAM
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FIGURE 6.5.1

the slab and beam models were made to be similar, thus allowing a single preprocessor to generate the necessary input data and minimize the effort required for bulk data input.

To satisfactorily describe structure-subgrade interaction for the beams, two elements each were required on the field and gage sides of the rails. This resulted in a 9 element wide section for the slab. Nodes were required at fastener locations, and an effort was made to keep aspect ratios near one. In addition, the length of beam elements required to allow a satisfactory loading description for the rail was considered, and the beam and plate elements were made equal in longitudinal dimension. This resulted in the element sizes shown in Figure 6.4.1, which yielded a stiffness matrix with a small band-width.

Several assumptions are inherent in the use of the plate-bending elements to model the slab. First the plate element is assumed to lie in the center plane of the slab. Second, the stiffness in positive and negative bending is identical. This second assumption was considered acceptable due to the approximate nature of the values used for several other significant parameters in the analysis. Third, load is considered to be transferred to the slab by the fastener at a point rather than over an area of some 12 inches by 13 1/2 inches in length. This latter assumption becomes significant in the interpretation of computer output, as discussed in Section 7, and shown in Figure 5.3.1.

6.5 Beams

The same basic discussion presented in Section 6.4 applies to the modeling of the beams using plate-bending elements. These elements were considered desirable in order to describe the structure-subgrade interaction.

The total section is 4 elements per beam, with gage members connecting appropriate nodes, as shown on Figure 6.5.1.

6.6 Gage Members

For the reasons set forth in Section 5.5, gage members were included for the beam structures. These members were modeled using beam elements connecting nodal points on the gage side of both beams, as shown on Figure 6.5.1.

Spacing of these members was at 5 feet intervals, and they were connected to the beams at sections where fasteners were also connected. The analyses were performed using 1 inch root diameter rods, which were later redesigned.

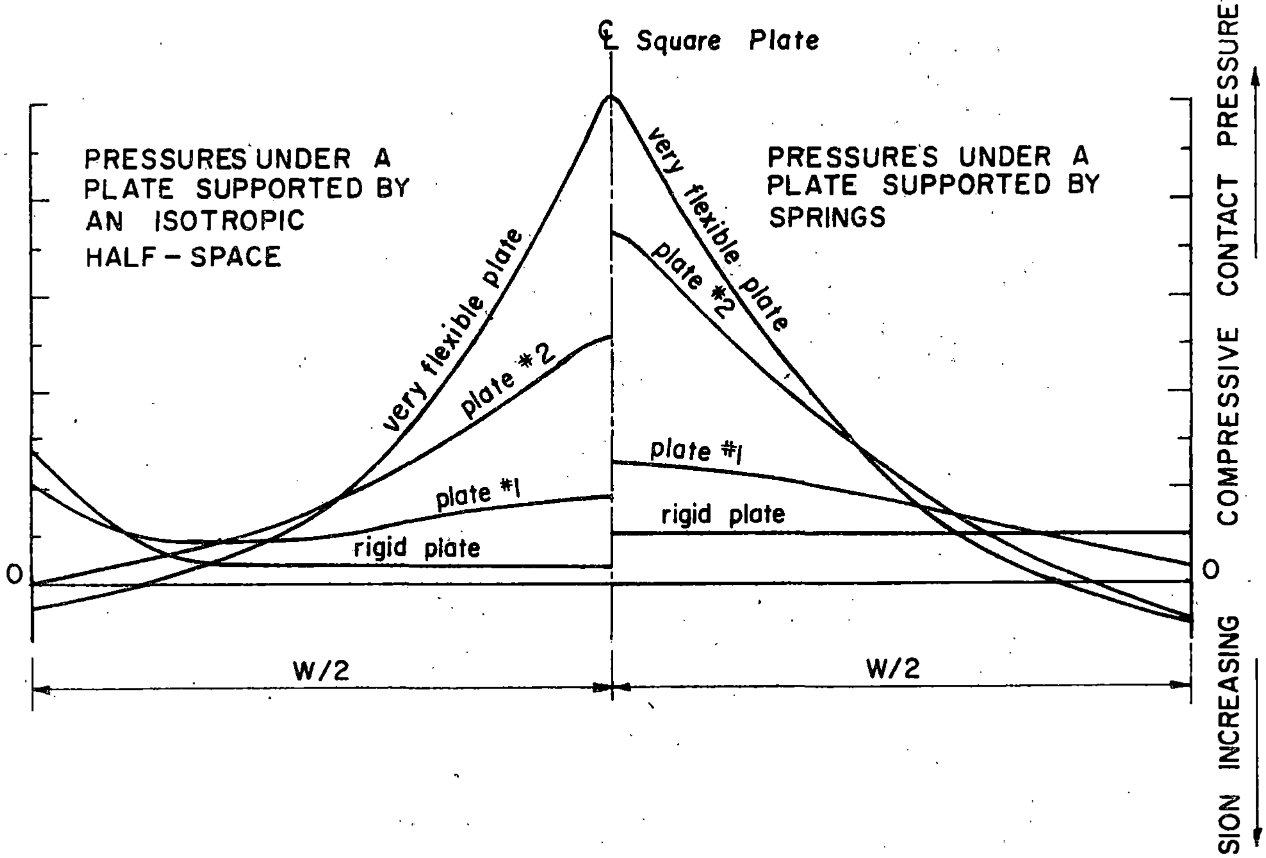
Two modeling approximations were involved for this member. Th e first was that connectivity was to nodes lying in the center plane of the beams, while the actual members would be placed above the centerline of the structure. The second was the location of the gage members at fasteners, rather than between fasteners. This results in higher loads on the gages, while the gages supply less resistance to the rolling tendency of the beams under lateral loads in the analysis. Both of these conditions are conservative for design.

6.7 Ballast - Subgrade System

A Winkler foundation has been selected for use in modeling the subgrade system. Although subject to certain errors, which are documented in the literature, this is a commonly accepted approach in design. A continuum approach is available in the finite element method. However, the use of static load criteria for a dynamically loaded structure, coupled with several unknown factors related to the effective structure stiffness and embankment properties, render the additional expense and effort necessary to achieve this sophistication unjustifiable at this time. Figure 6.7.1 shows a qualitive comparison of the continuum and discrete spring support systems.

Plate bearing tests, described in Appendix E, were performed on the ballast-embankment system by Westenhoff and Novick to determine values of static vertical subgrade modulus, k_V, for use in design. The locations of these tests were selected using the data developed, Appendix F, from construction test data transmitted by Shannon and Wilson. A spacial plot of test locations, similar to the one presented by Shannon and Wilson, and the data from the Corps of Engineers Vibro-Seismic Survey were used to aid in the selection of test locations.

Table E.1 shows the results of plate bearing tests made on the embankment. Values of subgrade modulus selected for design are $k_V = 200$ lb/in^2/in for the beam sections and $k_V = 150$ lb/in^2/in for the slab section. These values are the extreme limits of the Battelle recommendation for subgrade modulus and, on the basis of literature, the field tests, the apparent bilinear nature of the system, and the structure - subgrade interaction relations, are considered to be reasonable, and conservative for structural deflections and stresses. They compare favorably with values of dynamic k_V calculated using the Barakan formula, which is based on elastic theory. Values of shear modulus and Poisson's ratio for the embankment were taken from the Corps of Engineers report for use in the Barakan equation. Curves from this solution are shown on Figure 6.7.2 for different assumed lengths of rigid foundation for the structural cross-sections. From the computer analyses, the influence length for a single load is approximately 30 feet, establishing a range for dynamic k_V as a function of shear modulus.



(after Cheung and Zienkiewicz, 1965)

KANSAS TEST TRACK SANTA FE/DOT

COMPARISON OF FOUNDATION CONTACT PRESSURE FOR ACTUAL AND WINKLER FOUNDATIONS

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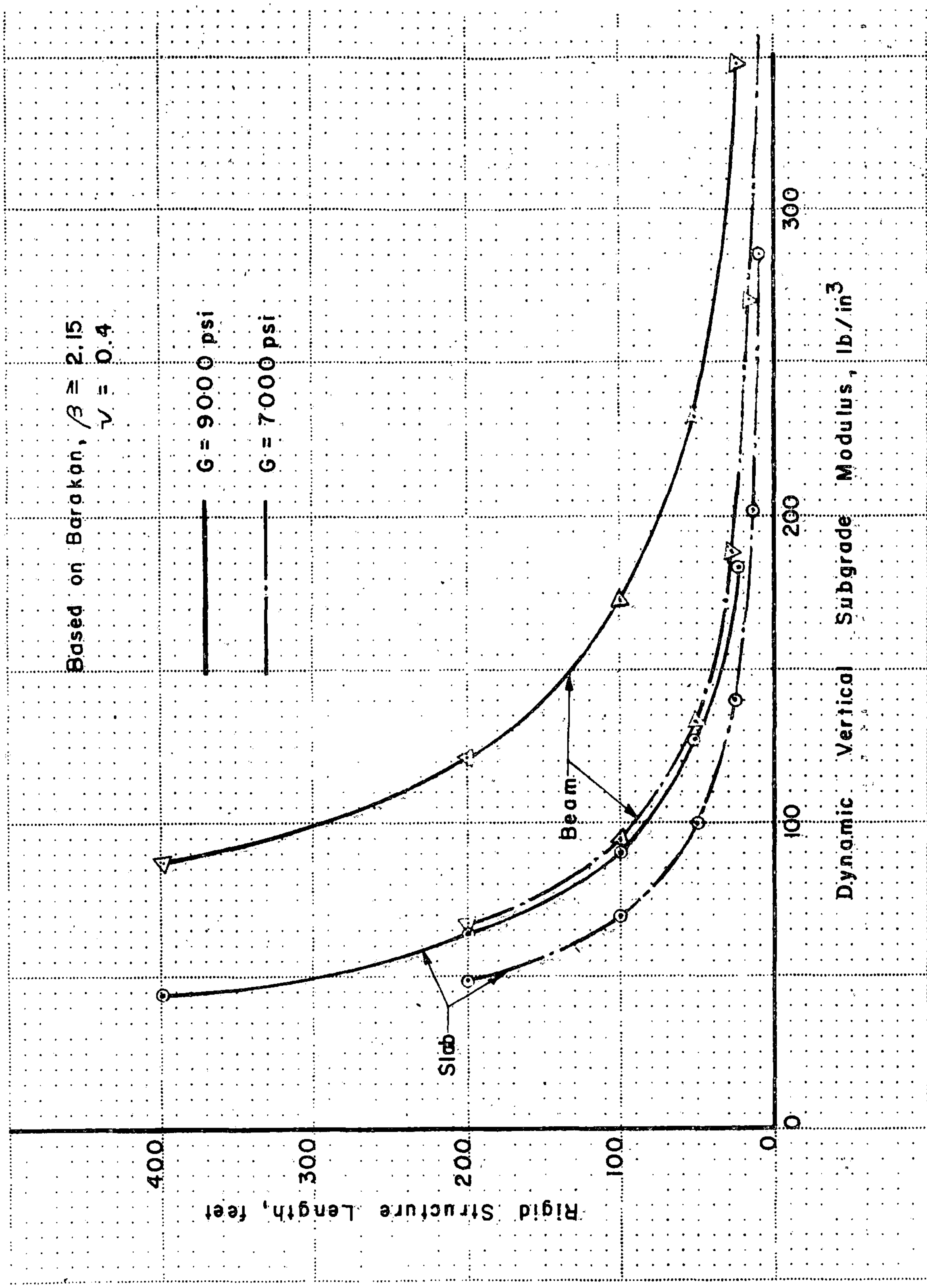
FIGURE 6.7.1



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THEORETICAL DYNAMIC SUBGRADE MODULUS

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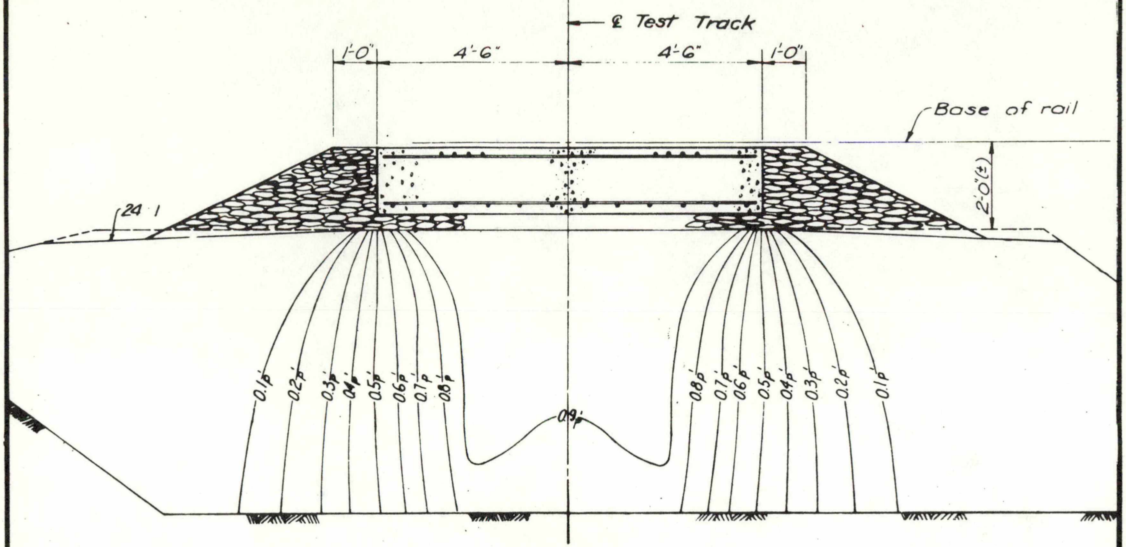
FIGURE 6.7.2

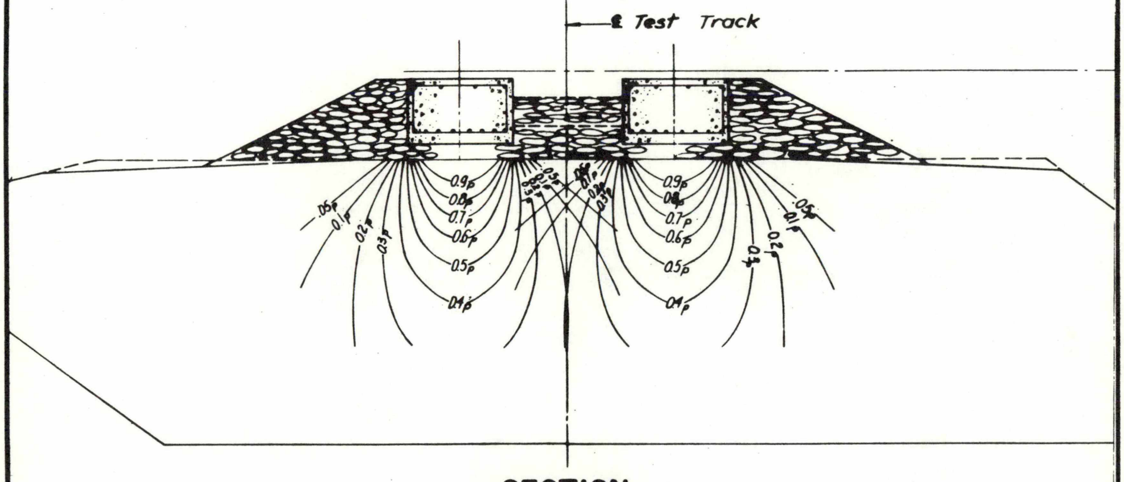


SECTION

Showing embankment and ballast at slab section.
Scale: 1/2"=1'-0"

1/2 1 2 3 4
GRAPHIC SCALE IN FEET

a

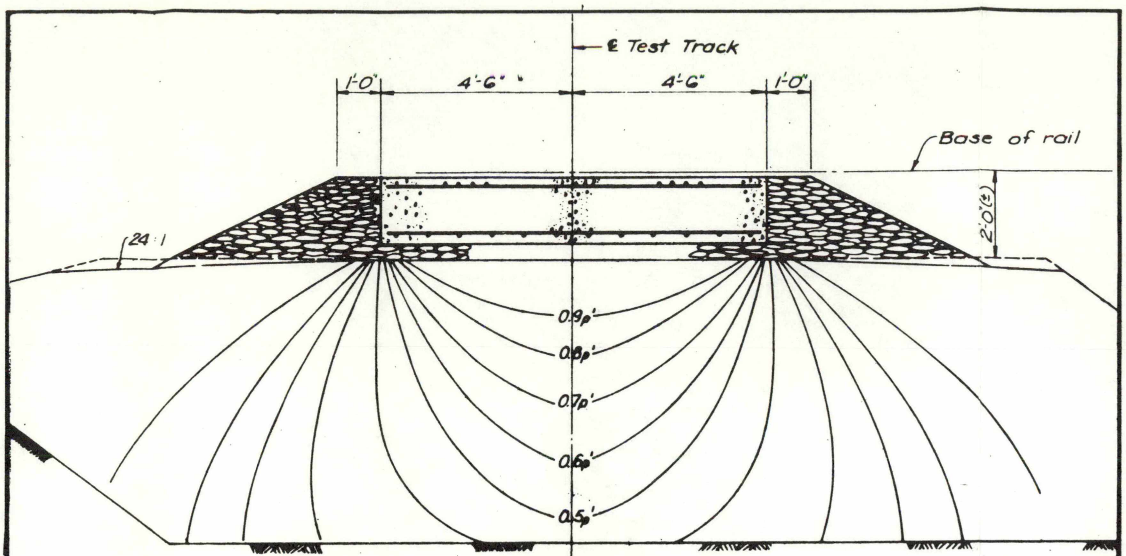


SECTION

Showing embankment and ballast at precast or cast in place beam section.
Scale: 1/2"=1'-0"

1/2 1 2 3 4
GRAPHIC SCALE IN FEET

b

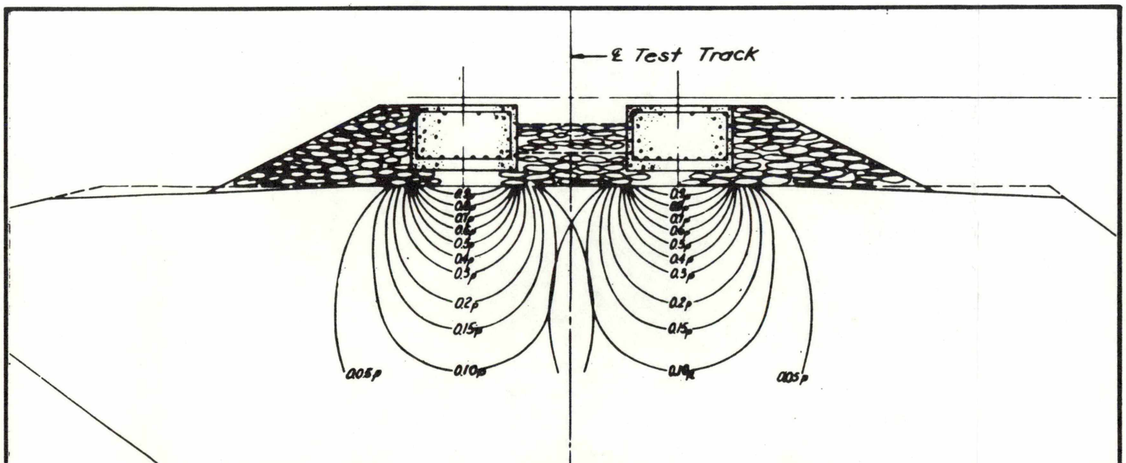


SECTION

Showing embankment and ballast at slab section.
Scale: ½"=1'-0"

½ 1 2 3 4
GRAPHIC SCALE IN FEET

c



SECTION

Showing embankment and ballast at precast or cast in place beam section.
Scale: ½"=1'-0"

½ 1 2 3 4
GRAPHIC SCALE IN FEET

d

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NON - CONVENTIONAL TRACK STRUCTURES

COMPARISON OF SUBGRADE STRESS DISTRIBUTIONS

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FIGURE 6.7.3

Selection of the values was based on an assessment of structure width and embankment depth effects, as shown in Figure 6.7.2. Figure 6.7.3 (a) and (b) show the theoretical stress distribution for a layered system, while (c) and (d) show the theoretical distribution for an isotropic halfspace. In addition, the influence of subgrade softening, the probability of the embankment increasing in strength under repeated dynamic loading, and the bilinear nature of the system (demonstrated by the plate bearing tests) were considered in the selection of these values.

Linear springs were used in the finite element model, as shown in Figures 6.4.1 and 6.5.1, with spring constants equivalent to the k_V values given above. The use of subgrade springs involves four assumptions. First, and most important, is the lack of shearing resistance within the embankment model. Second, the springs are linear. Third, is the concentration of resistance at discrete points. Fourth, is the ability of the model subgrade to act in tension; this effect will be essentially compensated for by the dead load of the structure.

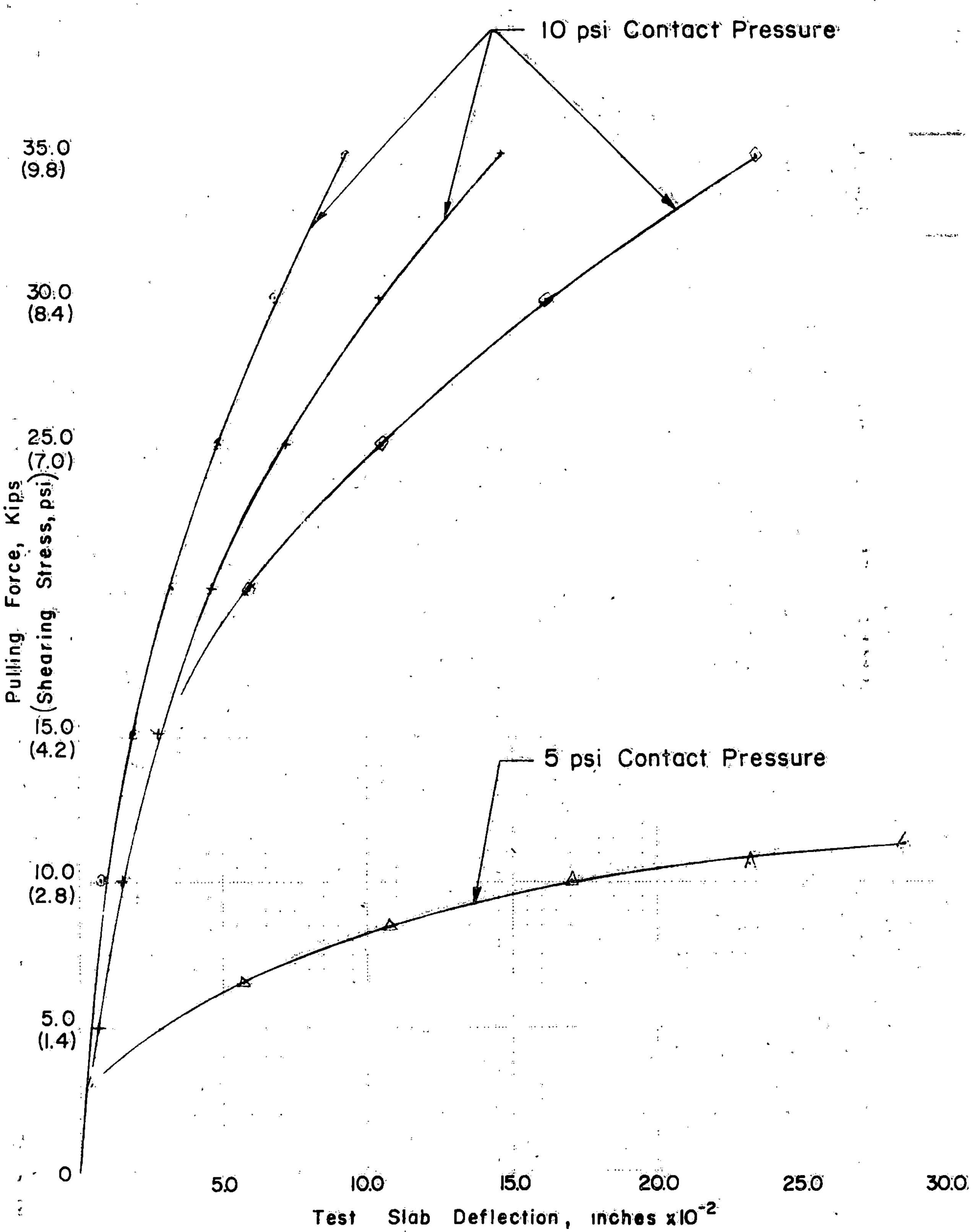
6.8 Ballast-Structure Interface

In addition to stiffening the subgrade and acting to spread the load on the embankment, the ballast supplies the only resistance to lateral load which the structures may mobilize. Little information exists in the literature related to strength properties (i.e., shearing strength), interlocking, and dynamic response of granular materials the size of ballast. No data were available from DOT on this subject.

Since resistance to sliding of the track structure, in a lateral and longitudinal direction will be derived from shear over the concrete-ballast interface, Westenhoff and Novick recommended that special field shear tests be performed.

These tests were performed on October 28, October 29, and November 9, 1971 by the Technical Research and Development Department of the Atchison, Topeka and Santa Fe Railway according to recommendations. A description is contained in Appendix D, while Tables D.1 and D.2 summarize the complete test series and contain values of peak and ultimate (sliding) friction derived from test data.

After considering the possible methods of modeling the interface shear, and an evaluation of the software capabilities, it was decided to utilize linear springs to represent the interface condition. The test series data were then plotted, Figure



KANSAS TEST TRACK		SANTA FE/DOT
BALLAST – SLAB SHEAR TEST RESULTS CAST IN PLACE OR ROUGHENED FACE IN CONTACT		
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FIGURE 6.8.1

6.8.1, and the equivalent spring constant derived for the linear portion of the curve, k_h = 175 lb/in^2/in, where it appears the classical linear friction relationship holds. This relation was checked against the computer solutions to assure compatible stress ranges and verify the validity of the model. Assumptions for this model are similar to those stated in Section 6.7.

It is called to the attention of the reader that the properties of ballast are worthy of a singular, extensive research project, due to their important influence on the total track system.

6.9 Embankment

The original project concept of constructing the embankment such that it has a constant influence on track structure response is indeed valid. A measure of the success of this effort was obtained from the results of field tests made during construction, and were used by Westenhoff and Novick to determine the locations at which the plate bearing test would be performed.

In order to aid in quantifying the properties of the "constant" embankment, Westenhoff and Novick recommended the performance of a Vibro-Seismic Survey of the test embankment. These tests were performed in October, 1971 and analyzed for the Department of Transportation by the Corps of Engineers, Waterways Experiment Station. The results of these tests provided the following:

1) A "rough" profile of embankment depth
2) A measure of embankment uniformity
3) A modulus of elasticity for the embankment
4) A shear modulus for the embankment
5) A measure of embankment damping characteristics

Items 3, 4, and 5 were correlated by laboratory testing performed at the Waterways Experiment Station, at Vicksburg, Mississippi. The above data provided substantial qualitative and quantitative input to this design effort, and should prove of significant value in test data reduction and future dynamic analysis of the track systems.

6.10 Ballast and Embankment Properties

Although the Santa Fe initially intended to furnish certain necessary properties to Westenhoff and Novick for design purposes according to the terms of the original agreement, this information was not, at the time of analysis, available and Santa Fe, therefore, authorized Westenhoff and Novick to develop the required information by letter dated January 3, 1972, effective December 28, 1971.

6.11 Model Loading Condensation

NASTRAN utilizes nodal points as points of application for concentrated forces. Therefore, when the geometry of load spacing in the actual case resulted in off-nodal loading of the rail model, concentrated loads were assumed to be distributed to adjacent nodal points by static methods. This is in accordance with general practice.

To assess the influence of the load condensation, the EASE model shown on Figure 6.2.2 was loaded as if the beam elements were 10 inches long, using static condensation. Fastener loads were compared for the case of a load at a node (5 inch elements) and equivalent condensed uniform or concentrated loads. Differences were small, generally less than 2%, for the rail-fastener load distribution to the structure.

6.12 Loading Magnitudes and Distributions

6.12.1 Static Loads

Static loads for design of the track support structures were taken as specified in the Battelle report, "Studies for Rail Vehicle Track Structures", which specified a wheel load of 38 kips for locomotives and 35 kips for freight cars. Sixty percent of the vertical load was applied as a horizontal load on the rail head. An impact factor of 100% was used.

While these wheel loads appear to be adequate, or even severe, for tangent track, such as the project test sections, the spacing or pattern of loads as specified by the Battelle report appears to be overly simplistic for design purposes. Wheel spacing and loading criterion, recommended by the Santa Fe Railway for design consists of ten class 8000-9000 locomotives, as a maximum load condition, with wheel loads of 38 kips as specified by the Battelle Report.

A comparison was made of the axle spacing of common rolling stock and motive power to determine if the use of class 8000-9000 locomotive axle spacing was typical and adequate. After determining that this truck geometry was satisfactory for use in analysis, due to the total magnitude of load and a spacing of two axles which closely approximates a freight car truck, the truck and axle

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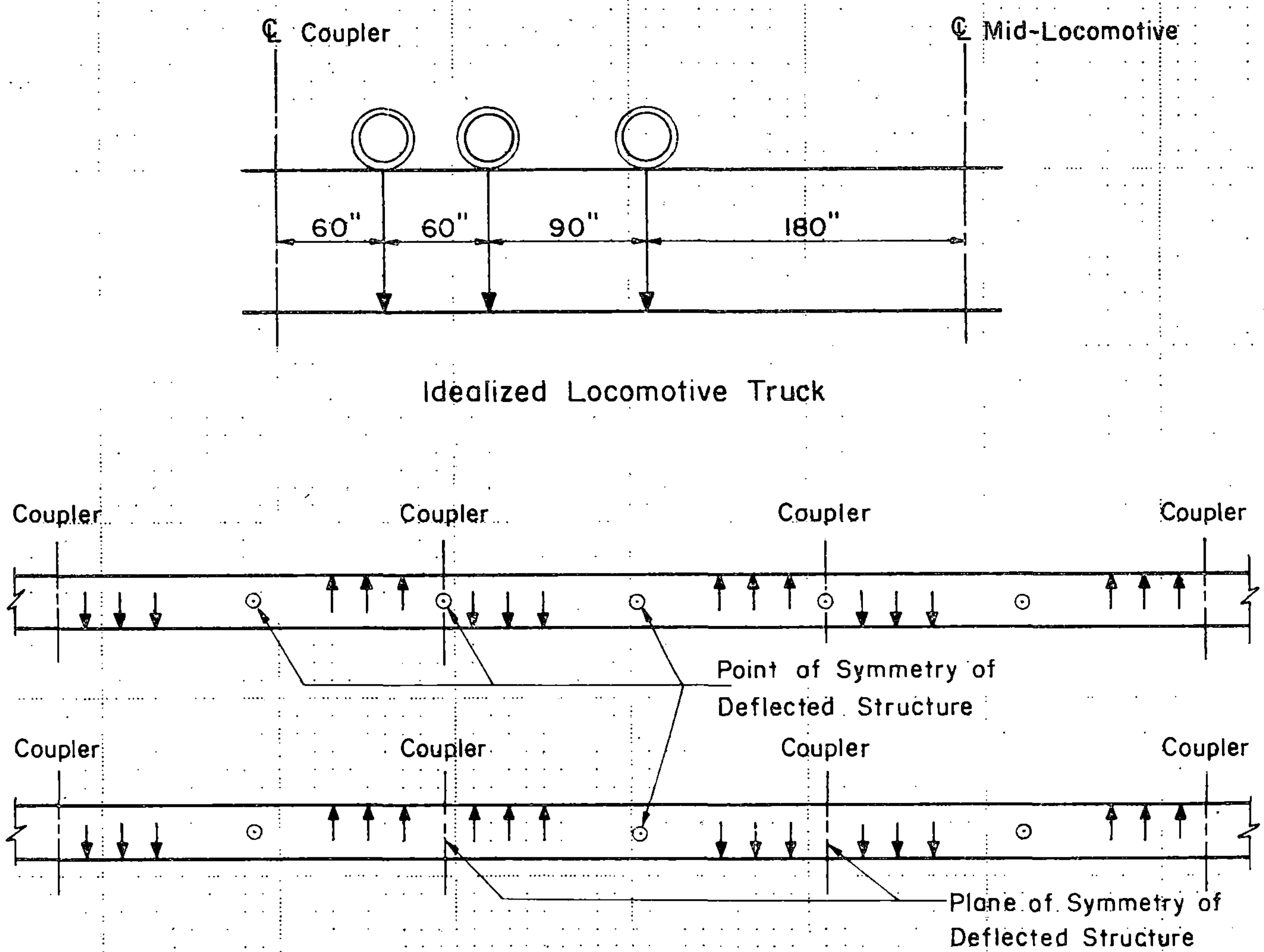
IDEALIZED LOAD SPACING AND DISTRIBUTION



Idealized Locomotive Truck

Idealized Lateral Load Patterns

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FIGURE 6.12.1

spacing was idealized, in conservative manner for the structure, as shown in Figure 6.12.1.

Design loadings on the track support structures will not account for the routing of special car loads over the test track. Since these cars normally proceed at reduced speeds, this will serve to reduce the impact effects and should lower individual wheel loads below the specified 35 kips with 100% impact. In addition, the relative infrequency of these special loads over this section does not justify designing for them.

6.12.2 Lateral Load Distribution

Since a tangent track will be used for the test sections, only some statistical distribution of lateral loads will be applied to the test structures. It is obvious that combined vertical and lateral loads at all wheels simultaneously is unrealistic. Without additional information on this point it is relatively obscure what lateral load sequencing may be expected. Two lateral load distributions were selected for evaluation for use in analysis. These distributions are shown in Figure 6.12.1.

One condition simulates the type of distribution which might occur under a backing action or a collapsing action caused by engine brake applications without train air application. Since this is a symmetrical distribution, it causes symmetrical deflection (plane) of the structures and a locomotive length (coupler to coupler) was analyzed for the infinite structure case. This condition would cause maximum rail head defection on the slab and maximum roll of the beams for a single rail or beam. This condition does not, however, cause maximum gage spread due to the track length over which loads are applied. A single-point constraint was used in the model to analyze the infinite structure for this condition. This was utilized to verify the occurrence of the point of symmetry, and to allow a comparison of stress levels with the second assumed load distribution.

The second condition is less probable than the first, but it is possible and it represents the severest load case for the structures. Also, gage spread is largest for this case due to the additive nature of the deflections of the rail or beams. A multi-point constraint was used in the model, based on the validity of occurrence of a point of symmetry verified in the single point constraint analysis, for this conditon. This load condition was used for design. Generally, the differences between the corresponding deflections and forces created by each load case were of a small order of magnitude.

KANSAS TEST TRACK

EASE MODELS OF FINITE BEAM STRUCTURE

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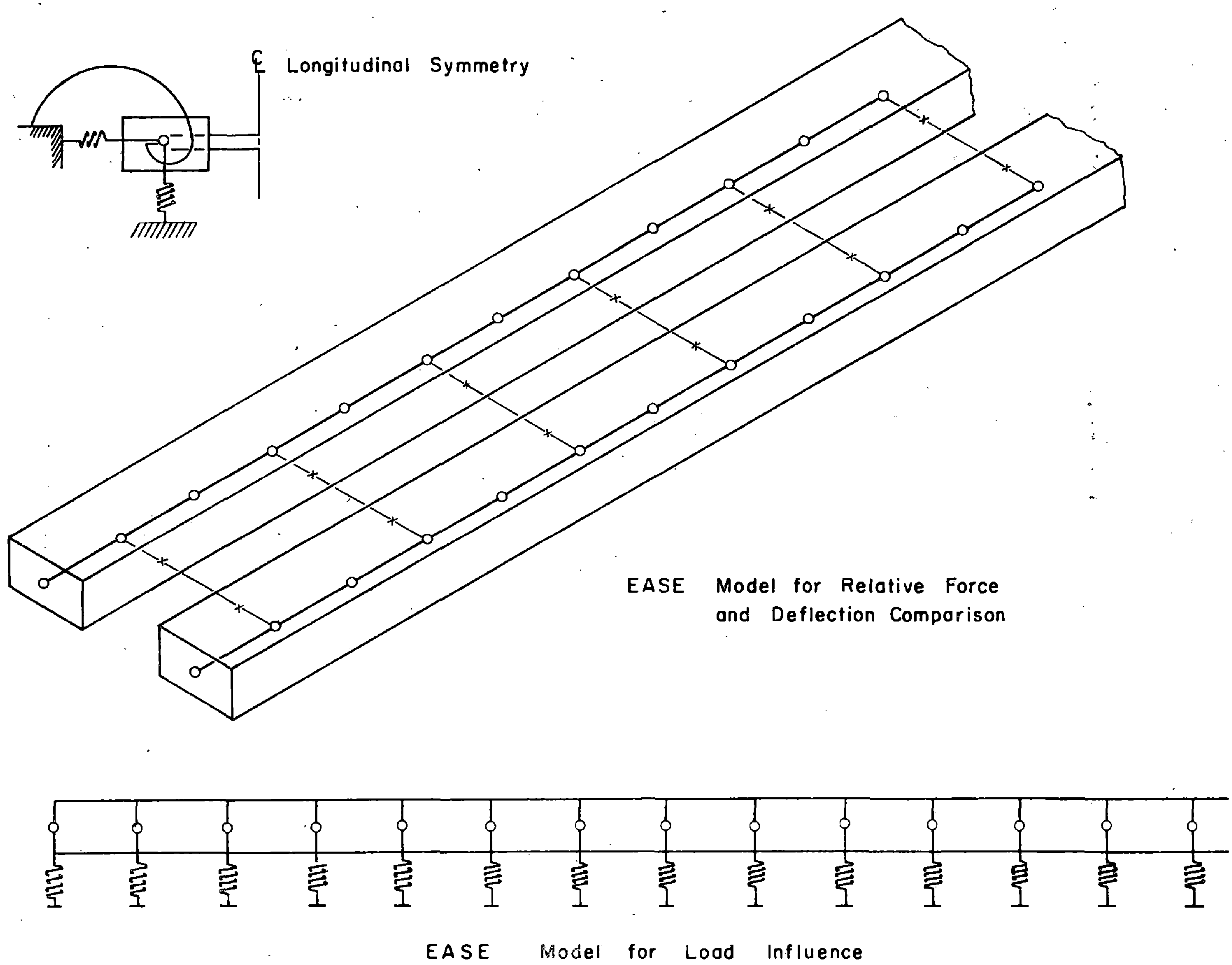


FIGURE 6.12.2

6.12.3 Finite Structure Loads

Two EASE analyses were performed using the model shown in Figure 6.12.2. Several single axle vertical loads were used and combined to give an indication of the critical load case for design. Subsequently, single axle vertical and lateral loads were used and combined for the same purpose. The load patterns investigated are shown in Figure 6.12.3. These analyses were also used to verify relative magnitudes and positions of results obtained using NASTRAN, and to show that a two axle truck (car) was not as critical to the structure as a three axle truck (locomotive) with equal axle loads. The load cases used to evaluate the finite structure for design were a single axle just on the structure, and a truck just entering (totally on) the structure; Load Cases 1 and 14 on Figure 6.12.3.

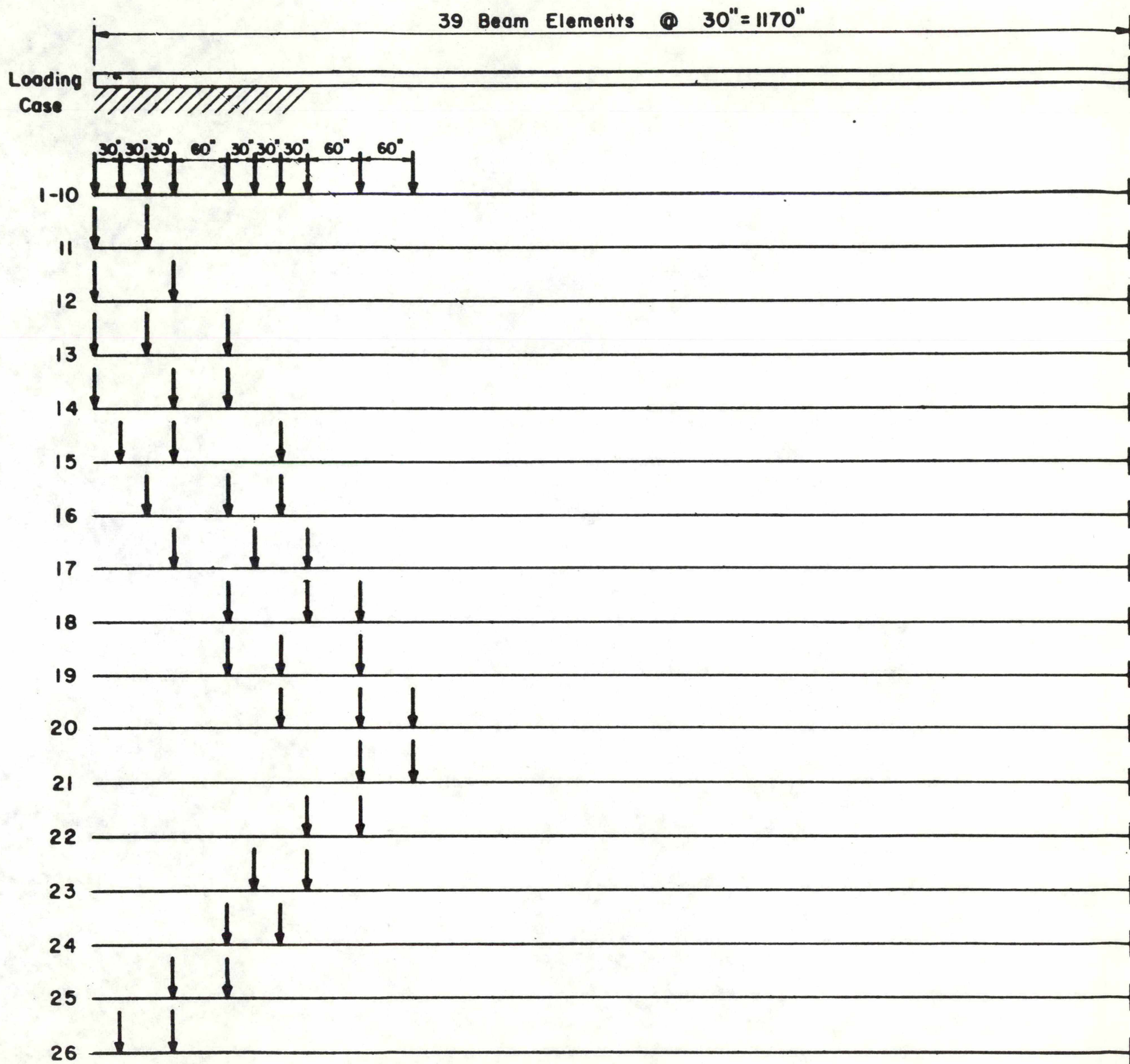
6.12.4 Dynamic Loads

As directed by DOT, the specific effects of dynamic loading will not be considered under the present scope of investigation, except as impact factor and lateral load. It is probable that some combination of speed, load pattern, and load frequency will result in structural stresses greater than those present under static load. However, using an impact factor of 100% of static live load may more than compensate for the increased stresses due to dynamic effects. The current AREA specifications call for a maximum impact factor of 60% for diesel locomotives on concrete structures, which may be further reduced if, in the engineer's judgement, the effects of impact may be dissipated. Future dynamic studies would be very useful in determining a more realistic impact factor for the types of track structure under consideration.

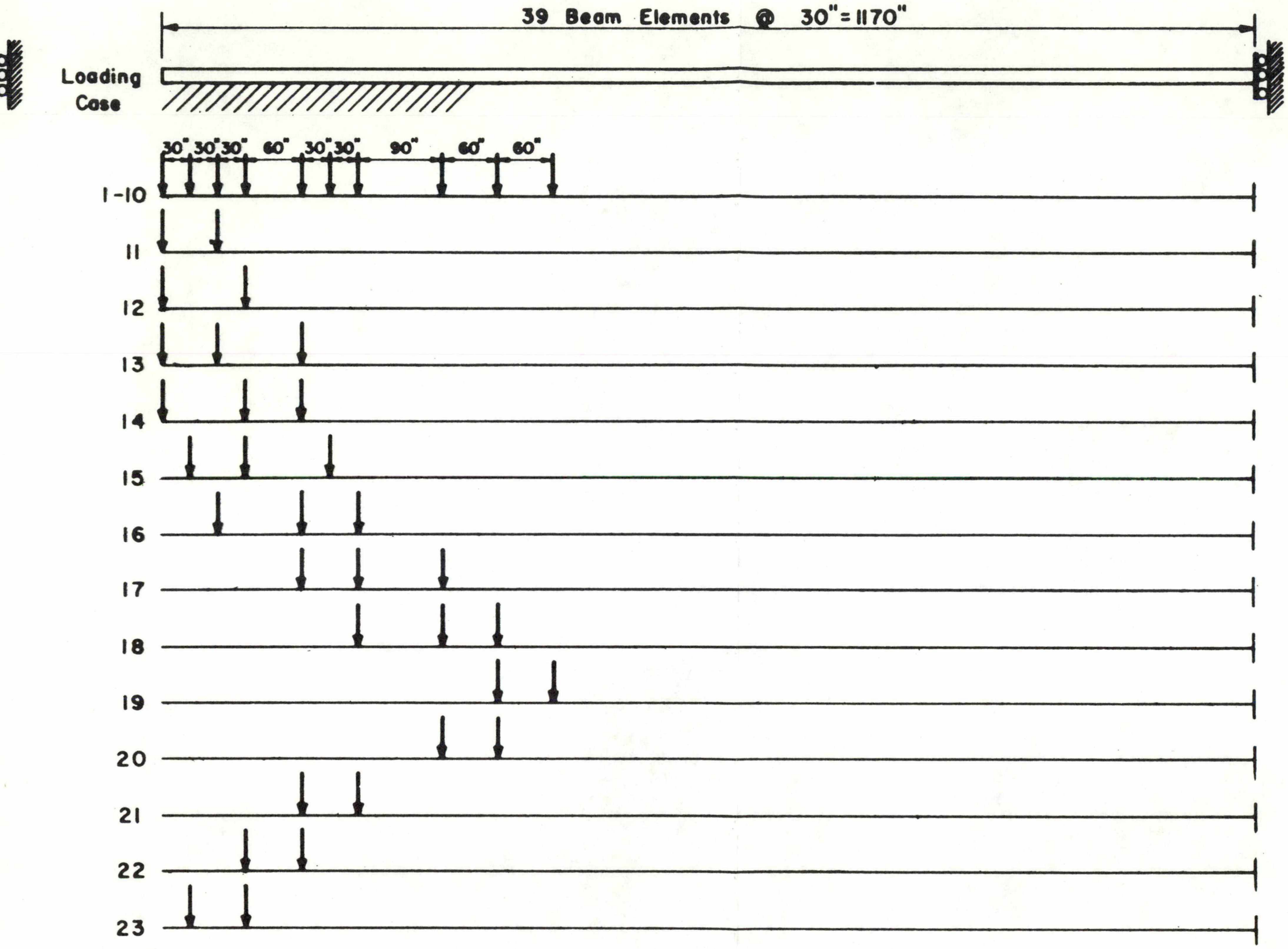
6.13 Transitions

Early in the design consideration was given to combining the finite structure models with a simple beam-spring model of the rail-tie and transition structures. In developing this model it became apparent that little could be determined analytically about the response, and hence also the model properties, of the transition base slab, ballast, and precast tie system.

Subsequently the structural relations of the two systems was studied and a model of the finite structures, felt to have fewer quantitive unknowns, was used. This model was one locomotive length long, and utilized single point constraints for the restrained (infinite) end of the model. The free (transition) end boundary conditions allowed, in plane deformation of the rail, while allowing the support



Vertical and Lateral Loading Positions



Vertical Loading Positions

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EASE FINITE STRUCTURE MODELS AND LOAD CASES		
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FIGURE 6.12.3

structure to be free. This condition was considered conservative for the analysis.

6.14 Reduced Subgrade Model

To assess the response of the structures to a subgrade softening, the most probable subgrade abnormality, the model of each infinite structure was changed such that an area existed immediately under one of the coupled trucks with a subgrade modulus of 25% of the balance of subgrade. This represents a very severe condition for the structures for the following reasons.

First, an abrupt change in foundation modulus is assumed. This will not generally be the case, as a gradual transition is most common. Second, the length of area which was assumed to be softened is arbitrary. Third, the model does not yield an increasing degree of subgrade support with increasing deflection of the structure. Fourth, when the models are considered in the symmetry condition both trucks of every other car are over a reduced subgrade. Due to influence lengths the result of this condition yields an unrealistically high negative moment.

In response to this type of condition the structures would distribute the load along their length, with associated cracking. However, the values obtained from the analysis are considered to be for a relatively severe case, the realistic value of which is only partially assessable at this time.

7. RESULTS OF ANALYSES

7.1 General

The analyses progressed according to the following sequence:

1) An EASE model of the rail-fastener system on infinitely stiff structure was made up as previously discussed. This model was used to assess the effect of static load condensation; of fastener properties on the total load and load pattern transferred to the structure; and to determine the effect which the rail torsional constant would have on the loading of the structure. These results are contained in other sections.

2) A long NASTRAN model (150±feet) was generated for each structure, using a small preprocessor to format the several thousand input cards for each run, and it was analyzed with a single axle

load and single truck loads to determine influence lengths for the structure, and to establish approximation orders for the rest of the models used in analysis. The influence length for a single load was less than 30 feet resulting in less than 2% error in forces for approximated models of that length, thus allowing an evaluation of subsequent models during their formulation. Figure 7.1.1 shows computer check plots of an abbreviated length of the models.

3) The preprocessor was altered to generate structural models for a single car length, the infinite case, to allow a consideration of the lateral load cases. The two cases were compared and the most severe case was selected to analyze for design. At this time the symmetry of the models was verified and it was determined that two models of the beam would need to be analyzed to account for gage member geometry.

4) As a prelude to analysis of the finite structure, an EASE model of a single finite beam was formulated and analyzed to determine the most critical load position for the end of the structure and to give a comparison of relative magnitude for the forces involved.

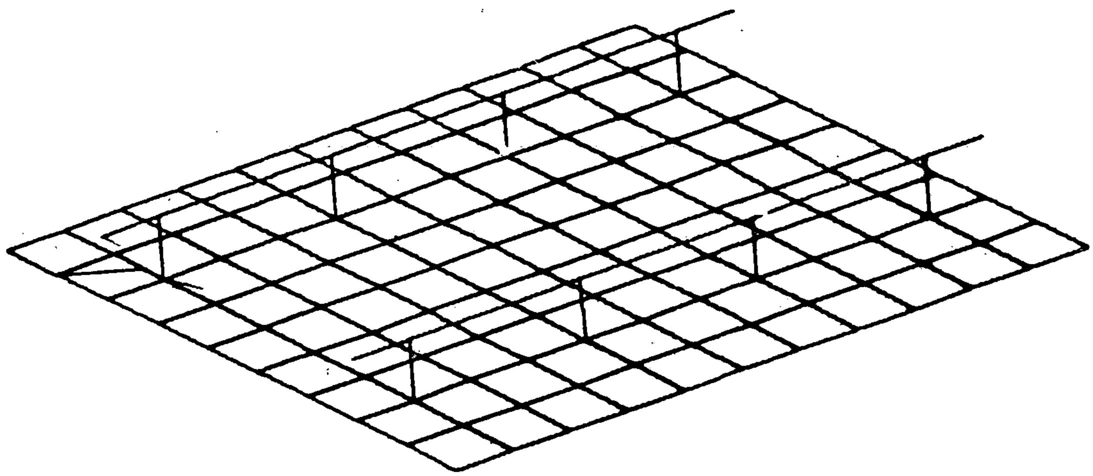
5) The NASTRAN preprocessor was altered to generate models of a single car length, with end conditions to give a finite structure for analysis.

6) An EASE model of a twin beam, finite structure, was used to give a comparison of relative order of magnitude and position of resulting forces for one of the end models.

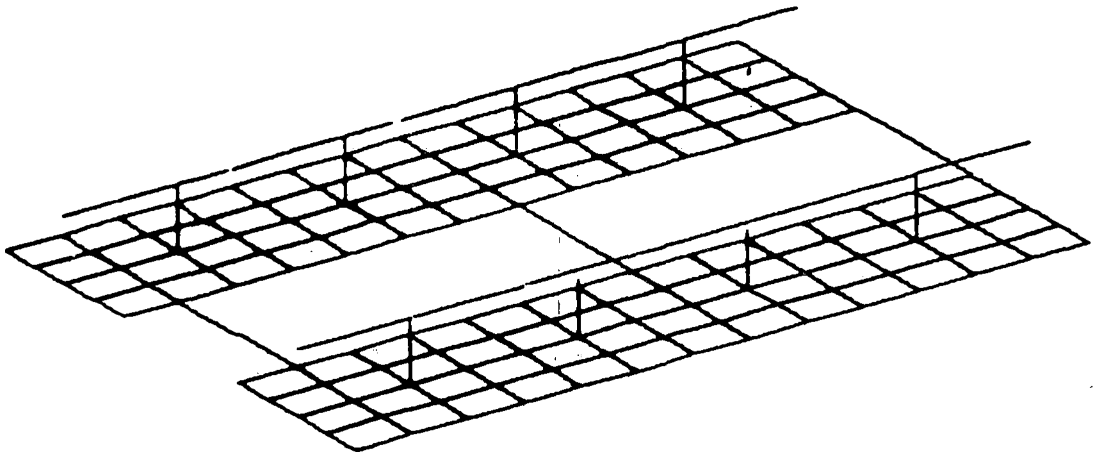
7) At this time the design was completed, based on the rationale discussed herein, and the adequacy of the sections was evaluated for the results of the computer analyses.

Since the finite element method is a lower bound solution to this analysis, a factor of 1.5 was applied to the computer results to account for approximation and modeling convergence. No factor was applied to displacements or subgrade stress due to the nature of the subgrade model used.

Matrix solutions were made by NASTRAN with a general accuracy of 10^{-10} resulting for all analyses.



SLAB



BEAM

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NASTRAN CHECK PLOTS
OF ELEMENT CONNECTIVITY

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FIGURE 7.1.1

TABLE 7.1.1

STRUCTURAL SECTION LOADINGS

IN PERCENT OF SECTION CAPACITY

TYPE	Longitudinal Bending	Infinite Structures: Uniform Subgrade Support Max	Avg.	Infinite Structures: Abnormal Subgrade Support Max	Avg.	Ends of Structures: Uniform Subgrade Support Max	Avg.	Ends of Structures: Abnormal Subgrade Support
SLAB	Positive	46% WSD*	31%	129% WSD	111%	55% WSD	43%	NOT CRITICAL
		31% USD+	21%	86% USD	74%	41% USD	32%	
	Negative	107% WSD	104%	212% WSD	208%	128% WSD	113%	OVERSTRESSED
		72% USD	73%	154% USD	146%	97% USD	81%	
BEAM	Positive	42% WSD	36%	120% WSD	107%	48% WSD	39%	NOT CRITICAL
		30% USD	26%	85% USD	76%	34% USD	28%	
	Negative	143% WSD	140%	284% WSD	284%	117% WSD	93%	OVERSTRESSED
		100% USD	98%	200% USD	200%	83% USD	61%	

* Working Stress Design
\+ Ultimate Strength Design

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8) The preprocessor was used to generate models for the reduced subgrade support condition. Section capacities were compared to resulting forces and stresses.

All analyses were performed using loads exclusive of impact, and the results were increased by 100% to account for impact at the time of design.

Table 7.1.1 contains a summary of structural section loadings for reference. These values were developed under the rationale of Section 8.1, and are discussed in succeeding subsections.

7.2 Slab

7.2.1 Stresses

Review of the computer analysis of the slab revealed few unusual or unexpected results. As anticipated, the positive bending moments in the "infinite" portion of the slab were relatively low, with uniform subgrade support. Based on a cracked section, and assuming the moment to be distributed over the full cross section, these moments result in a maximum tensile steel stress of approximately 7500 psi. If the concrete does not crack, and the same moments exist, only about 160 psi tension will be developed in the concrete, and approximately 1000 psi tension will be developed in the positive reinforcing. It should be noted that these figures include an impact factor of 100%, plus a modeling and convergence factor of 50%. Concrete with a $f'c$ = 4500 psi would not normally be expected to crack at tensile loads of 160 psi.

Stresses in the negative steel are considerably higher, near 25,000 psi, an approximate 1000 psi overstress. The main reason for this is that considerably less steel is provided for negative bending than for positive bending. The computed negative moments are considered to be high, however, because of homogeneous, orthotropic stiffness properties input to the computer. A homogeneous longitudinal stiffness (EI) of 4×10^{10} lb/in^2 was used, based on cracked sections. However, this is true only for positive bending. The stiffness for negative bending is about one fourth of this value. This would have the effect of reducing negative moments and increasing positive moments, as the structure would redistribute the load.

Shear stresses in the slab are also high, though not above allowable limits. No vertical reinforcing was provided in the slab and consequently, shear stresses in the concrete must be limited to 75 psi. Calculating shear stresses at a distance d/2 from the edge of the loaded fastener yields a maximum shear stress of 64 psi. Utilization of the shear value at this point is in accordance with the current AREA specifications, assuming the structure to be analogous to a footing. The AREA code specifically refers to a "footing acting essentially as a wide beam, with a potential diagonal crack extending in a plane across the entire width", which is the exact case. It should be noted that unless vertical design loads may be reduced, the present slab dimensions are about the minimum which may be used without providing shear reinforcing or over-stressing the concrete in shear. This maximum shear stress occurs near the end of the slab, while stresses in the "infinite" portion are slightly less.

Bending moments at the end of the structure are approximately 50% higher than in the "infinite" portion. This necessitated an increase in negative reinforcing, but stresses in the positve steel remain low.

Transverse bending stresses in the infinite portion of the slab are moderate, with maximum stress in the negative steel of approximately 17,000 psi, based on a cracked section. The positive bending stresses are considerably higher for about the first 30 inches from the end of the slab. However, the computed transverse bending moments are known to be conservative because the computer model transferred all loads from the fasteners as concentrated loads, rather than distributed over the full width and length of the fastener. Also, computer output gave loads and moments at a point, which in the case of transverse bending does not allow for any distribution which will occur, i.e., transverse moments averaged over a 40 inch length are considerably less than the maximum moment output for a single 10 inch element.

It should be noted that the stresses measured near the instrumentation cutouts will vary from the stresses given above, because the reduced depth of section necessitated changes in reinforcing in order to maintain the specified stiffness. The stress in the positive longitudinal steel will be approximately 88% of that in the full section, or a maximum of about 6500 psi. Stresses in the negative longitudinal steel will be about 10% higher than in the full section. These values are based on a cracked section. Both positive and negative transverse bending stresses will also be about 10% higher because of the reduced effective depth.

To determine the effect of possible variations in subgrade modulus, a 15 foot length of subgrade was given a modulus equal to 25% of that of the remainder of the subgrade. This severe condition results in an overstress of all reinforcing steel. Most effected is the negative reinforcing which would be stressed to about 85% of yield. This is not considered to be a legitimate design criteria, but an evaluation of adequacy, as it is impractical to design for all possible events.

A "step decrease" of 75% in subgrade modulus is highly unlikely in terms of both magnitude and occurrence.

7.2.2 Contact Pressures and Deflections

As was expected, the maximum slab deflection, and hence contact pressures, occur at the end of the slab, under combined vertical and lateral loads. The effect of the lateral load was very slight however, amounting to only a small increase over pressures due to vertical load only. The maximum pressure of 38 psi drops off very quickly, becoming 22 psi within 6 feet. For the remainder of the length of the slab, 22 psi is the maximum pressure developed for the design load. These pressures are under the loaded rail and include impact. Under vertical loads there is little variation in pressure across the width of the slab. The maximum variation amounts to little over 2 psi. The variation under combined vertical plus lateral load is about 10 psi. Both longitudinal and transverse distributions of contact pressures are shown on Figure 7.2.1, exclusive of impact. Deflections may be determined from the figures as noted.

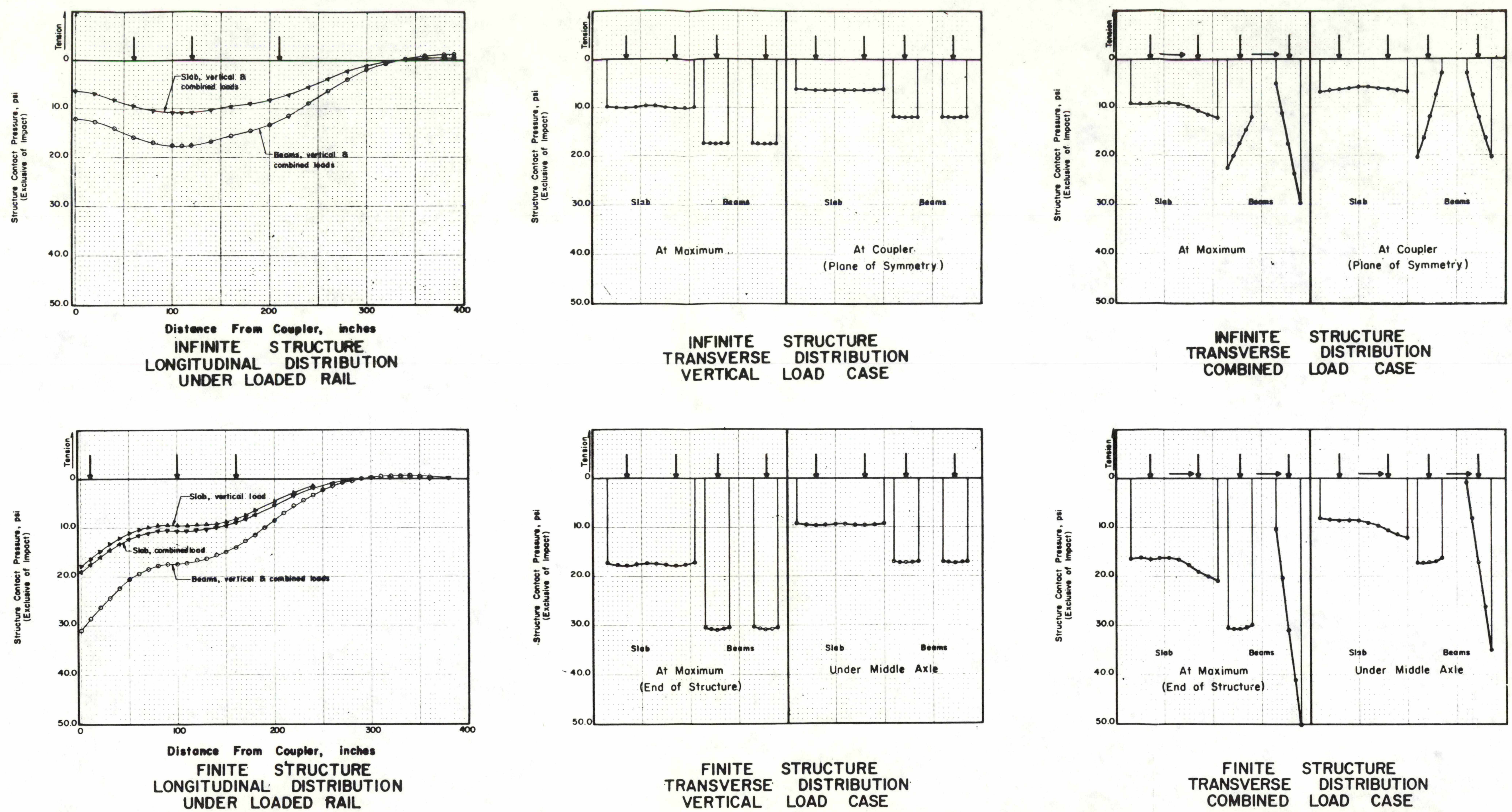
Deflections for the case of subgrade softening are shown in Figure 7.2.2. These values are exclusive of impact, and may be converted to pressures as noted.

7.3 Beams

7.3.1 Stresses

The beams were found to be more highly stressed than the slab in all cases, with the exception of positive longitudinal bending. The positive bending stresses are relatively low, with a maximum stress of slightly less than 9,000 psi occuring in the "infinite" beam. More critically, the negative bending stresses reach approximately 33,000 psi. While significantly above the allowable working stress of 24,000 psi, this is considered acceptable. As discussed earlier, the lower stiffness of the beam under negative bending will reduce the actual moment as compared to the computer output. Further reasoning behind the design of this steel will be discussed in Section 8.1.

As expected, the combination of shear and torsional loads in the beams was the controlling condition. As with the slab, the critical section for design was taken at a distance of d/2 from the edge of the loaded fastener. Using these values, the shear stress in the concrete and stress in the tied stirrups were within allowable limits. It should be noted that requirements for shear reinforcement, under



NOTES:

1. ↓ 38 Kip Vertical Load
2. → 0.6 (38 Kip) = 22.8 Kip Lateral Load
3. Multiply pressure and displacements by 2 to obtain calculated values for impact loadings.
4. To convert pressure to displacement divide by appropriate vertical subgrade constant.

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CALCULATED STRUCTURE CONTACT PRESSURES		
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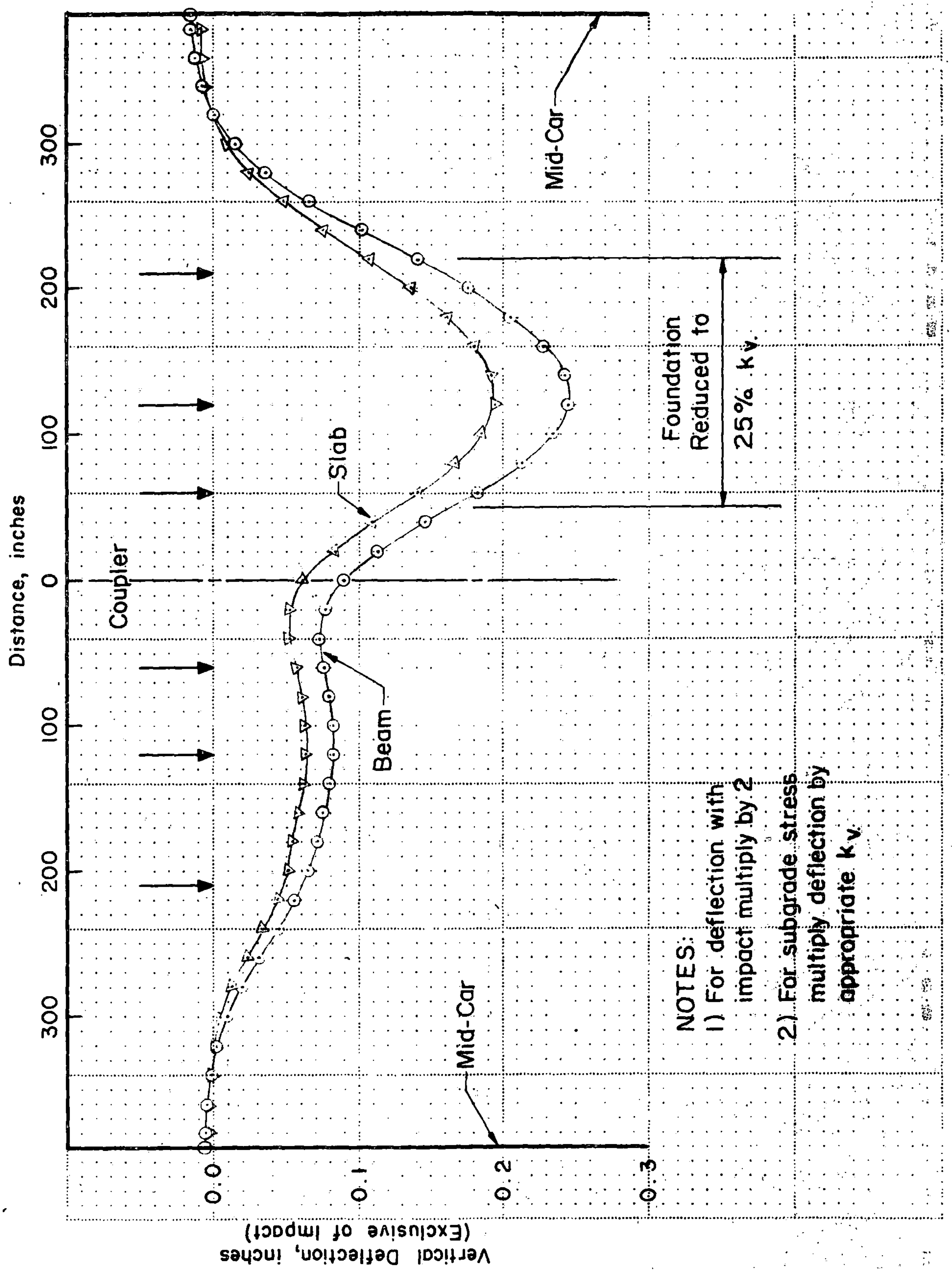
FIGURE 7.2.1



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SANTA FE / DOT

VERTICAL DEFLECTION OF
STRUCTURES WITH SOFTENED SUBGRADE

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FIGURE 7.2.2

conditions of combined vertical shear and torsion, are very sensitive. A relatively small increase in either shear or torsion has a large effect on the required stirrup size and spacing.

While the computer output indicated the existence of transverse bending moments, these are a modeling feature and may be neglected in design. The rail fastener has a physical width and length, while the computer model represented fastener loads as concentrated loads.

All stresses at the ends of the beam were slightly higher than for the "infinite" beam, similar to the slab. Positive bending stresses increased to slighly above 9,000 psi. Again, the negative bending was sufficient to require additional steel, resulting in a final stress of 22,000 psi. Torsion and shear stresses remained within allowable limits.

For the case of an uncracked beam section, assuming the same maximum moments, the maximum concrete stress would be approximately 500 psi, for the "infinite" beam under negative bending. Concrete with an $f'c$ = 4500 psi would not normally be expected to carry this tensile stress, and would result in a cracked section.

As with the slab, the beams are overstressed under a condition of reduced subgrade modulus. The positive steel is the least effected, showing stresses of less than 26,000 psi. However, negative steel would be stressed to near yield and the shear and torsion would exceed the capacity of a 30 inch by 18 inch concrete section regardless of the stirrup size and spacing. As previously stated, this condition is considered to be severe and was not taken as a design requirement.

As a result of the analysis, it may be stated that the present beam sections are near the optimum concrete section for the given design loading conditions. This is due to the control of the design of the section by shear and torsional stresses, resulting from the loading condition, which are to be resisted by the concrete.

7.3.2 Contact Pressures and Deflections

Contact pressures under the loaded rail were considerably higher for the beams than for the slab. For the "infinite" beam the maximum pressure was 36 psi, while beam pressures under lateral loads reach 50 psi; these values include impact.

At the ends of the beams, contact pressure at centerline of rail is 62 psi for vertical loadings and the addition of lateral loads increases edge pressure to over 100 psi. It has been noted in Section 9.2.1 that because of this high contact pressure, the ballast at the ends of the beams could be an area of high maintenance.

Both longitudinal and transverse distributions of soil pressures are shown on Figure 7.2.1, exclusive of impact. Deflections may be determined from the figure as noted.

Deflections for the softened subgrade case are shown on Figure 7.2.2, exclusive of impact. They may be converted to contact pressures as noted.

7.3.3 Beam Spread

At the time of analysis and design the selected fastener had not yet undergone acceptance testing, hence, the final properties were not known for the fastener. Since the rail deflections are primarily due to fastener characteristics, DOT required that only the beam spread be evaluated for design. Figure 7.3.1 shows the beam spread resulting from the loading conditions analyzed, exclusive of impact, and the contribution is not considered excessive.

8. DESIGN

8.1 Design Specifications and Methods

As originally conceived, the usual working stress design methods, as provided in the most current AREA Specifications, were to be used for both slab and beam structures. Ultimate strength, or "load factor", design methods were considered but were not utilized at that time because they did not appear to offer any particular benefits. The reason for this was that the reinforcing required to meet the specified longitudinal stiffness requirements of DOT could not be reduced whether working stress or ultimate strength design methods were used.

Upon analysis of computer output, it became apparent that an ultimate strength design approach did indeed hold a number of advantages over the working stress approach. With the exception of positive bending reinforcement, an excessive amount of reinforcement, based on experience, resulted by using working stress methods. In addition it was recognized that an increase in reinforcement, particularly in the top of the structures, would complicate the already difficult problem of assuring full contact between the concrete-fastener sub-assembly interface, and might impair fastener anchorage.

Upon further consideration of the conservative design loads, it was

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BEAM SPREAD - COMBINED LOAD CASE

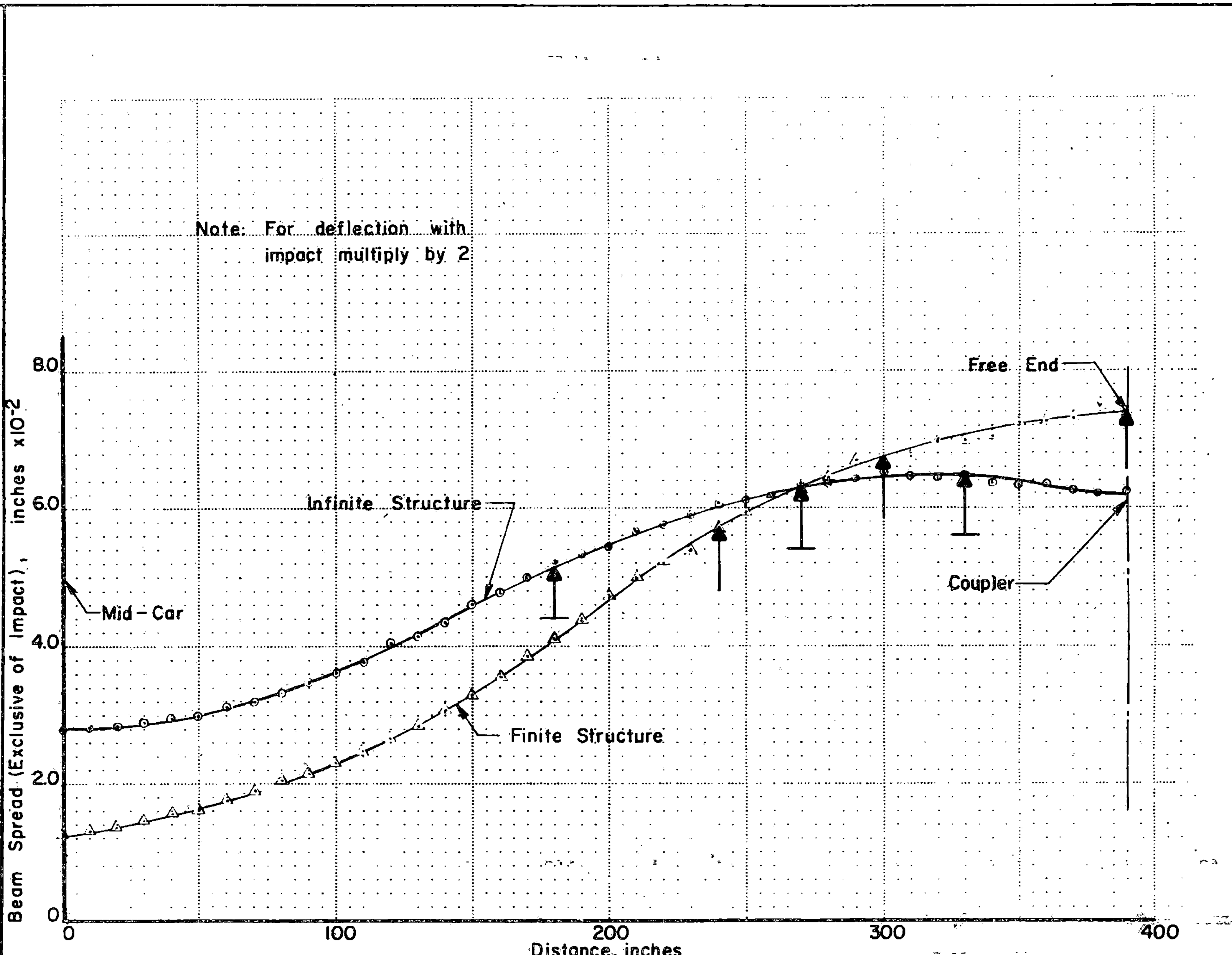

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FIGURE 7.3.1

decided to use these loads (with appropriate load factors) in conjunction with an ultimate strength design approach. This was further influenced by the fact that there is no widely accepted method of design for combined longitudinal bending, vertical shear and torsion using working stress methods. The current "Building Code Requirements for Reinforced Concrete", ACI Standard 318-71, has specific design methods for such loading conditions, however, based on an ultimate strength approach.

The current edition of the AREA Specifications, Chapter 8, Concrete Structures and Foundations, does not recommend ultimate strength and ultimate load design methods. As this is a research project, and not intended to be in rigorous service over an extended period of decades, an ultimate strength design is considered appropriate and adequately conservative. However, in addition to meeting the requirements of the ACI specifications for ultimate strength design, it was decided to require the structures to meet the requirements of the AREA specifications Chapter 8, part 19, "Rules for Rating Existing Concrete Bridges", based on working stress design. This permits an increase in reinforcing steel stress of 50%, from 24,000 psi to 36,000 psi for Grade 60 steel. Concrete stress is not critical under either design method. Table 8.1.1 contains structural capacities for the slab and beams calculated by the above methods.

In summary, the structures are designed to meet the following two conditions simultaneously:

(1) Ultimate strength design with loads = computed load x 1.5 (model factor) x 2 (100% impact) x 1.7 (load factor).

(2) Working stress design (using rating stress of 36,000 psi in reinforcing) with loads = computed load x 1.5 (model factor) x 2 (100% impact).

Both the model factor and the impact factor are considered to be reasonable and conservative.

8.2 Materials

8.2.1 General

The literature abounds with evidence that all of the materials utilized in the project (steel, concrete, soil, elastomers) have strength properties which are significantly load-rate and load-frequency dependent, and, to varying extents, temperature dependent. This fact should be considered during future data analysis

TABLE 8.1.1

COMPARISON OF STRUCTURAL CAPACITIES

CALCULATED BY VARIOUS METHODS

STRUCTURE	BENDING CAPACITY					
	fs = 24,000*		fs = 36,000*		Ultimate**	
	Pos.	Neg.	Pos.	Neg.	Pos.	Neg.
Slab - "Infinite"	1,008	508	1,512	762	1,555	716
- Transverse***	4.5	4.9	6.7	7.3	6.2	6.8
- @Instruments	1,152	472	1,728	708	1,463	657
- @Transitions	996	602	1,494	903	1,359	850
Beams - "Infinite"	607	196	910	294	838	273
- @Instruments	790	218	1,185	327	1,028	245
- @Transitions	607	378	910	567	838	535

All bending moments are given in in.-kips.

* Capacity reduced by a factor of 3 = 2 X 1.5 (to account for 100% impact and 50% model factor) for direct comparison with computer output.

** Capacity reduced by a factor of 5.1 = 3 X 1.7 (as above plus load factor of 1.7).

*** Capacity given for 1" strip.

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and subsequent design projects. Although the scope of design on this project is limited to static analysis, the actual structures will be subjected to dynamic loadings, and will respond to them according to their respective dynamic properties. General notes applying to design loadings and allowable stresses are shown on Sheet 3 of the Design Plans.

8.2.2 Concrete

The specified 28 day compressive strength for concrete is a minimum of 4,500 psi. This will provide a modulus of rupture of approximately 650 psi and a direct tensile strength of about 380 psi. While compressive strength is in itself not a critical item, the tensile strength is important for two reasons. First, a high tensile strength increases the probability of the concrete sections remaining uncracked, thereby resulting in greater stiffness per inch of structure depth. At the same time, the probability of random cracking due to temperature stresses or other causes is proportionately reduced. Concrete will be mixed and placed by construction methods specified in the standard specifications of the Santa Fe Railway Company, supplemented by the Special Provisions of the contract documents. Because of the expected difficulty in casting the concrete in a manner which will minimize voids at the interface between the concrete and the fastener sub-assemblies, the Special Provisions provide for the construction of several 10 foot test beams. The purpose of these test beams is to determine the optimum construction procedures and appropriate concrete mix proportions. The desirability of using an admixture in the concrete to increase workability will also be determined at this time.

8.2.3 Steel

The reinforcing steel utilized is ASTM Designation A615, A616 or A617, Grade 60 with a minimum yield strength of 60,000 psi and an allowable design stress of 24,000 psi under AREA specifications. This steel was initially used because it is the standard reinforcing used by the Santa Fe Railway. At a later date it was determined to be necessary because of its high allowable stress.

8.2.4 Ballast

Standard Santa Fe crushed, basic, Pueblo Slag will be used as a ballast under the structures. Tests have been performed showing the ballast to conform to AREA specifications, and that it is probably non-expansive and suitable as a base course for the cast-in-place concrete structures. Observation of field performance, and a series of one-point modified Proctor tests, indicated that resistance to crushing under dynamic loads is probably satisfactory, although no criteria exist with which to assess this condition. Reference is made to Figures C.1 and C.2, and the Summary of Test Results in Appendix C.

8.3 Control Joints

Control Joints, as shown on Sheet 6 of the Design Plans, will be constructed at ten foot intervals in both the slab and beam structures. Joint formers will be provided at the top and bottom of the control joint to prevent cracks from forming at undesirable locations and interfering with fastener performance. Plastic sheathing will be provided for 12 inches on either side of the control joints to prevent significant bonding between the steel and concrete.

8.4 Precast Beams

Precast beams are to be cast in units of approximately 40 feet in length. A cast-in-place section between abutting precast sections will provide for full moment and shear transfer, as shown on Sheet 6. Moment transfer will be achieved by field welding of the reinforcing bars using standard A.W.S. welding procedures. Precast beams are conventionally reinforced and will have control joints at the same spacing as the cast-in-place beams and slab.

Beams systems, both cast-in-place and precast, are provided with gage members at 5 foot intervals which are intended to resist beam overturning under lateral loads.

The precast beams will be provided with lifting stirrups, which may also be utilized for beam handling and placing, at the ends and the 10 foot points of each beam.

8.5 Transitions

Transitions consist of a mesh reinforced concrete slab below the ballast and the concrete ties as shown on Sheet 8. As may be seen, varying the depth of the transition slab creates a gradual increase in stiffness from the "soft" tie system to the "stiff" slab or beam system. Allowing a minimum of eight inches of ballast above the transition slab provides a means of tie adjustment by conventional track surfacing methods. The 8 inch depth is also considered adequate to minimize any tendency for the ballast to break up under the concrete ties. The top of the transition slab will be crowned for lateral drainage, and it will be artificially roughened. The variable spacing of ties through the transition is designed to aid in gradually increasing the track stiffness.

8.6 Gage Members

A 1-3/4 inch diameter steel bar, anchored to the two beam sections, is provided as a gaging member at a spacing of 5 foot intervals. In addition to controlling gage spread this member serves to restrain the beams against overturning. This bar will be highly stressed, under the design lateral loads, because the location of the bar in the beam will result in both tensile and bending stresses in the bar. Fatigue may also become a problem since a reversal of direction of the lateral loads results in a stress reversal in the bar. This bar has been designated in Section 9.2 as a possible area of high maintenance. To reduce this possibility and to increase its resistance to corrosion, the bar will be of ASTM A-588 steel.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 Areas of Probable Construction Problems

9.1.1 Fastener Installation

There are two main areas of anticipated construction problems. The foremost of these is the procedure for placing the fastener sub-assemblies. This in itself consists of two inter-related problems.

First, the sub-assembly must be located within close tolerances regarding alignment, grade, and cross-slope. Secondly, while maintaining position regarding alignment, grade, and cross slope, concrete must be placed in a manner which will prevent the formation of voids at the interface between the concrete and the sub-assembly, and provide adequate bond between the two.

Until such time as this project is completed and in operation, it will not be known how successfully this problem has been anticipated and provided for. Several alternative solutions have been suggested and will be examined in a number of 10 foot test beams to be constructed before the start of the actual construction operations. The procedures to be adopted for final construction will be determined by these tests.

9.1.2 Precast Beam Bedding

Proper bedding of the precast beams onto the ballast material is the second probable area of difficulty. The procedures described in the Special Provisions of the contract documents give a suggested method of bedding. The success of these procedures will again be determined only when the project has been completed and in service. The bedding of these beams is considered to be a possible high maintenance item.

Due to the unusual nature of the project, there are undoubtedly other areas where the contractor will be faced with new construction problems and procedures. While not of the same magnitude as the problems with the sub-assemblies and precast beams, these areas will probably be different from the "normal" type of construction and will require special consideration and close observation. These areas may include placement of the control joints; the welded field splices of the precast beams; and the placement of the large number of closed stirrups in the beam sections.

9.2 Possible High Maintenance Areas

9.2.1 Transitions

The transition zones of both the slab and beams may be expected to be areas of high maintenance. While all possible effort has been made to smooth the transition from tie supported track to structure supported, contact pressures at the end of the structures remain high. While computed pressures include an impact factor of 100%, this is not considered excessive near the transition areas. Periodic maintenance work on the ballast material may be expected. This is primarily applicable to the beam structures.

Fasteners and rails may be subjected to significantly increased loadings near the end of the sections, and may become maintenance prone in these regions.

9.2.2 Gage Members

The gaging members of the two beam systems may also be an area of high maintenance. If the design lateral loads are actually developed, the gage members will be subjected to both tensile and bending stresses, which will combine to give high fibre stress. Nearly complete reversal of the maximum fibre stress is possible depending upon the direction and magnitude of the lateral loads. Fatigue may therefore be a factor in the performance of the gage member. At this time there is no data regarding magnitude, direction, and frequency of lateral loads on which to base calculations on the probability of fatigue failure during the life of this project.

The steel gage members are also relatively exposed to the elements and are therefore liable to corrode. Special steel (A-588) has been used to reduce this likelihood, however, its corrosion resistance is not considered effective with respect to corrosive agents such as brine or roadway salts. Thus the amount of deterioration will be primarily dependent upon the amount and type of leakage from railway traffic.

9.2.3 Precast Beams

The precast beam structures present a third area of possible maintenance problems. This is wholly dependent upon the success of the initial bedding procedures during the construction period. Maintenance work on the ballast material may be required, especially in the early life of the test program.

9.2.4 Concrete

Since this is a test program, design with reasonable conservatism but with many variables which are not completely defined, some cracking and/or spalling of the concrete structures may result. However, it is not anticipated that such cracking will require repair procedures, because of the relatively short life requirement. If cracking or spalling should occur to any extent, the cause should be extensively investigated and eliminated in any future track structures. Particular attention should be paid to this area in order to allow an assessment of the effects of placement procedures, admixtures, and curing processes during the life of the test.

9.2.5 Rails

Rails will be subjected to higher stresses due to their direct fixation by the fastener. It is possible that past successful performance of the rail section may insure its adequacy for the non-conventional structures, particularily at the ends of the structures abutting the transition zones.

9.3 Inspection Program

An established program for inspections should be adopted to assure that the full benefit of the project results can be realized. It is suggested that the Santa Fe track patrolman, during his normal daily patrol duties, carefully observe the project for apparent low spots, high spots, line spots, sun kinks, bent bolts, damaged fasteners, evidences of dragging equipment, broken or distressed gage members, plugged lateral drain channels, broken, cracked, or spalling concrete, subgrade deterioration, or other obvious, unusual conditions.

It is recommended that the signal maintainer test the electrical resistance between rails with the frequency that he tests insulated joints and at the same time. The resistance between rails should be at least as high as that required for conventional cross-tie track, and tests should be made under snow, wet, and dry conditions.

It is recommended that a detailed walking inspection be made at one month intervals by the Santa Fe engineer. He should make the same checks as the track patrolman, but in more detail, and in addition he should examine the structure closely for cracks, rail fastener bedding behavior, spalling or broken concrete, or other evidence of distress from loads or weather, fatigued gaging members, roadbed drainage, water pockets, slope erosion, porosity of ballast shoulders, lateral or vertical dislocation of the structures, if any, and troublesome vegetation tending to impede drainage. His findings should be logged, perhaps reduced from a voice recorder tape, and photographs should be taken.

After six month intervals, or at other times deemed to be advisable depending on the results on the monthly Santa Fe inspections, an inspection should be conducted by a team comprised of representatives of Santa Fe, Department of Transportation, Portland Cement Association, Fastex Division of Illinois Tool Works, and Westenhoff and Novick, Inc. All of the items covered by the Santa Fe engineer's monthly inspection should be inspected by the team, in addition to other items requiring review and evaluation. The findings of the team inspection should be logged and photographed.

9.4 Recommended Maintenance Procedures

If high subgrade pressures at the beam ends result in settlement of the subgrade, periodic tamping of the ballast will be required. A more permanent solution, however, would involve a method of reducing the subgrade pressures. This may be accomplished by excavating the ballast and some subgrade from beneath the beams and casting a slab the full width between the outside of the beams and for several feet from the end of the beams. This would provide a greater bearing area for the beams, thus reducing subgrade pressures. An alternative method would be to place several concrete ties beneath the beams, using a non-shrink grout or epoxy between the beams and the ties.

Tamping of the ballast material will also be required if the precast concrete beams are improperly bedded initially. This tamping should be required only in the early stages of the test period, or until the beams are bedded through tamping and by daily rail traffic.

If fatigue in the gage members does, indeed, result in failure, complete replacement of the unit is the only procedure. If deterioration due to corrosive agents appears to be excessive, the gage members can be wrapped in a clear polyethylene sheeting to prevent direct contact with the corrosive agent, yet allow visual inspection.

9.5 Accident Investigation Procedures

A derailment on the test track is not expected. In fact, such an occurrence from a track-related cause is highly improbable, due to the extraordinary care, scrutiny, and caution addressed to every phase of planning, design, and construction. Neglect of the possibility, however remote, would be imprudent and unrealistic in light of the many possible causes which could precipitate a derailment.

The rail traffic operating over the test track will be comprised principally of conventional rolling stock fulfilling DOT/FRA inspection requirements, but varying in age, wear and maintenance condition, and will be operated according to prescribed operating rules and procedures. But such inspections, rules and procedures do not positively preclude the possibility that defective or improperly lubricated equipment may be included in a train consist, or that train handling and braking may be imperfect, or that emergency braking may be necessary to avoid an obstruction on the track or at a road crossing.

It is suggested that any investigation of a derailment at or near the non-conventional track structures be conducted according to procedures prescribed by Santa Fe, or by DOT/FRA if circumstances require. It is suggested that engine tapes be impounded promptly to prevent their being lost or mislaid, so that they will be available for examination by all directly concerned, in the event a direct physical cause cannot be determined. In such an event, all offices and firms directly concerned with the test track project should be notified immediately and be given an opportunity to promptly examine all available evidence and statements.

9.6 Instrumentation

The instrumentation originally conceived for this project was primarily concerned with the measurement of pressures at the fastener-structure interface, subgrade pressures, the measurement of strains in the longitudinal reinforcing steel, and structural deflections. Upon completion of computer analyses, it became apparent that additional instrumentation is required for the beam structures if meaningfull results are to be obtained from this test track.

As originally conceived, only the positive and negative longitudinal steel was to be provided with strain gages. This will yield the service stresses and provide a means for verifying longitudinal bending moments, however, results of computer analysis indicate high shear and torsional loads in the beams when lateral loads are applied. Several stirrups in the beams should be instrumented to provide some means of determining the actual shear and torsional loads being carried by the beams. Since it is suspected that the lateral loads used for design, which include a 100% impact factor, are conservative, determination of the actual resulting torsional stresses is highly desirable. The actual distribution of vertical shear near the fasteners is also required in order to verify the shear values. This could also be done with properly instrumented stirrups.

Torsional loads are not a factor in design of the slab structure. Here, positive and negative longitudinal bending moments are of principal interest, and are instrumented. However, the slab is also subjected to significant transverse bending stresses, and it is desirable that these also be verified by additional instrumentation.

Instrumentation of one or more gage members would allow verification of stresses, stress reversals, and also provide data on frequency and direction of the actual applied lateral loads. This would provide a basis for future design.

9.7 Instrumentation Calibration Under Field Loading

It is recommended that a series of known static loads and dynamic loads be applied to the structures to observe the response of the system.

Static loads could be applied very easily, by jacking the rails apart, dead loading, etc., and would give load deflection relations for the structures which could be compared with existing theories, and the design analysis. Also a comparison could be made on a section to section basis to determine relative responses throughout the duration of the test.

Dynamic loads could be applied to the structure by means of a lumped or rotating mass vibrator attached to, or resting on, the rail. Separate application of dynamic loads to the structure directly would allow the influence of the rail and fastener to be determined for future evaluation. Since the load frequency and magnitude would be known, the instrumentation could be evaluated, calibrated, and the response could be compared with readouts from more complex loading conditions to form a decision base. In addition, the "natural frequency" of the total track system could be measured and damping characteristics determined for use in future analysis and design.

The contribution of these efforts to the project, and to future design efforts for more sophisticated applications, would outweigh additional costs. A properly planned and executed addition would not create a significant delay in the program, and a very high return would be realized. This effort should be made for all test sections, and is considered necessary to provide evaluation of design criteria and to assess its application for future design.

9.8 Future Design

The basis of future design of safe, economical track structures is primarily dependent upon three factors: (1) verification of the need for a structural stiffness (EI) of 4×10^{10} lb-in^2, based on a cracked concrete section; (2) verification of an impact factor of 100%; (3) verification of the required magnitude of lateral loads to be applied, which in this case consisted of 60% of the vertical load, plus 100% impact. This last mentioned criteria is applicable mainly to the beam structures. Should it be possible to reduce any or all of the three factors listed above, significant savings could be obtained in reinforcing steel, concrete, or both.

At this time, as a result of the analysis and design, it would appear that the slab design holds the most promise for future development, depending on the specific application intended. In comparison with the slab, both beam systems required more longitudinal steel, and are sensitive to both shear and torsional loads. Both beam systems require a gaging system, which is a possible area of high maintenance, as well as an additional initial cost as compared to the slab. The precast units have the additional disadvantage of requiring field splices between the beams, an additional cost item.

Elimination of the field splices of the precast beams would reduce initial cost, however, discontinuities in the beam system would magnify their inherent weaknesses. Without the torsional stiffness provided by a continuous system, the resistance to overturning due to lateral loads is greatly reduced. To prevent this, a larger, stiffer, and hence more expensive gaging system would have to be provided. More importantly, high subgrade pressures would occur at the ends of each beam, unless some provision for shear transfer is provided. Even so, the points of shear transfer would tend to be areas of high maintenance.

10. DESIGN PLANS AND SPECIAL PROVISIONS

10.1 Design Plans

LIST OF DRAWINGS IN DESIGN PLAN SET

Drawing Number	Sheet Number	Title
EOT-6110-I	1 of 8	LOCATION MAP, INDEX, & SUMMARY OF QUANTITIES
EOT-6110-I	2 of 8	GENERAL PLAN DIAGRAM, PROFILE & TYPICAL SECTIONS
EOT-6110-I	3 of 8	INSTRUMENTATION LOCATIONS & GENERAL NOTES
EOT-6110-I	4 of 8	BOTTOM REINFORCING - SLAB & BEAMS
EOT-6110-I	5 of 8	TOP REINFORCING - SLAB & BEAMS
EOT-6110-I	6 of 8	CONCRETE & REINFORCEMENT DETAILS
EOT-6110-I	7 of 8	CONCRETE & MISCELLANEOUS DETAILS
EOT-6110-I	8 of 8	TRANSITION & MISCELLANEOUS DETAILS

U.S. DEPARTMENT OF TRANSPORTATION

NON-CONVENTIONAL TRACK STRUCTURES

KANSAS TEST TRACK

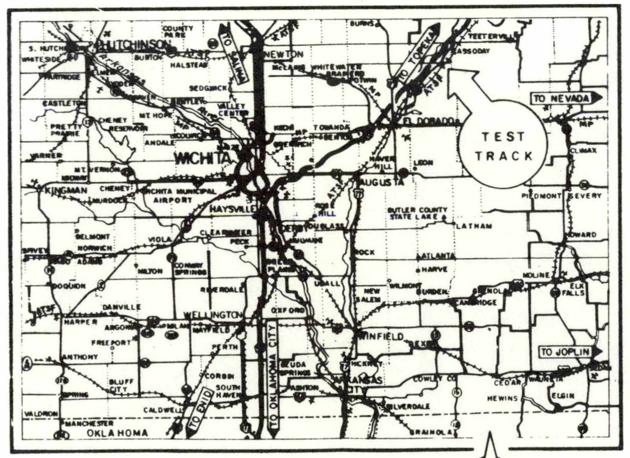
BETWEEN AIKMAN AND CHELSEA, KANSAS

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY

EASTERN LINES, TOPEKA, KANSAS

INDEX TO DRAWINGS

DRAWING NO. ATSF NO.	SH. NO.	DESCRIPTION
	1	LOCATION MAP, INDEX & SUMMARY OF QUANTITIES
	2	GENERAL PLAN DIAGRAM, PROFILE & TYPICAL SECTIONS
	3	INSTRUMENTATION LOCATIONS & GENERAL NOTES
	4	BOTTOM REINFORCING - SLAB & BEAMS
	5	TOP REINFORCING - SLAB & BEAMS
	6	CONCRETE & REINFORCEMENT DETAILS
	7	CONCRETE & MISCELLANEOUS DETAILS
	8	TRANSITION & MISCELLANEOUS DETAILS



LOCATION MAP

0 5 10 20 30

ONE INCH EQUALS APPROXIMATELY 17 MILES

N W E S

SUMMARY OF QUANTITIES

DESCRIPTION	QUANTITY	UNIT
CONCRETE - CAST IN PLACE BEAMS	222	C.Y.
" " " " SLAB	400	C.Y.
" PRECAST BEAMS	222	C.Y.
PLACE REINFORCING - CAST IN PLACE BEAMS	64,096	LB.
" " " " " SLAB	66,832	LB.
" " PRECAST BEAMS	60,555	LB.
CONTROL JOINTS - CAST IN PLACE BEAMS	160	EA.
" " " " " SLAB	79	EA.
" " PRECAST BEAMS	158	EA.
INSTALL FASTENER SUB-ASSEMBLLES		
CAST IN PLACE BEAMS	160	EA.
" " " SLAB	160	EA.
PRECAST BEAMS	160	EA.
BALLAST - CAST IN PLACE BEAMS	700	TON
" " " " SLAB	500	TON
" PRECAST BEAMS	700	TON
GAGE MEMBERS - CAST IN PLACE BEAMS	160	EA.
" " - PRECAST BEAMS	160	EA.
FIELD JOINTS - PRECAST BEAMS	´38	EA.
TRANSITIONS	4	EA.
TEST BEAMS	3	EA.

6405

David Novak

KANSAS PROFESSIONAL ENGINEER NO 6405

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS

DATED: May 4, 1972 APPROVED: C. L. Holman
ASST GEN. MGR. - ENGINEERING

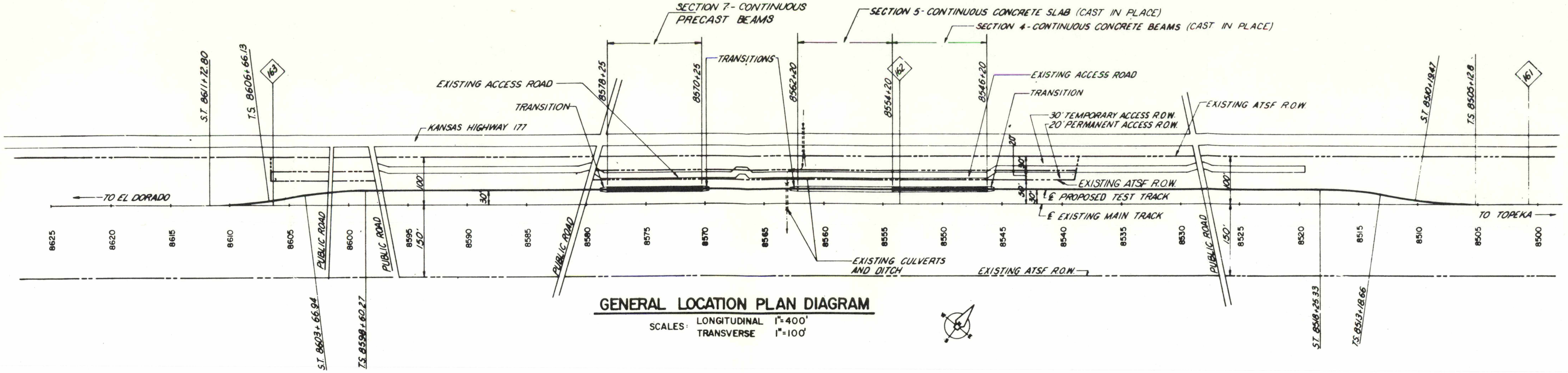
U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

NON CONVENTIONAL TRACK STRUCTURES

LOCATION MAP, INDEX & SUMMARY OF QUANTITIES

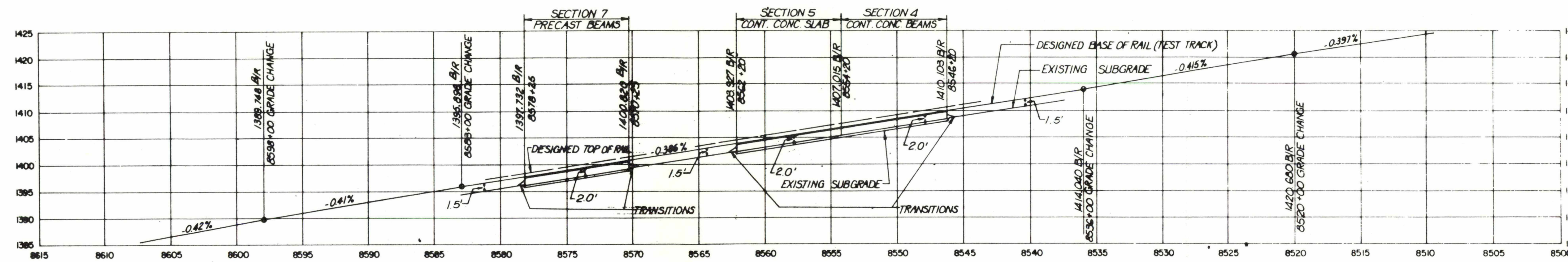
WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS

SHEET 1 OF 8
ATSF DRWG NO.
EOT-6110-I



GENERAL LOCATION PLAN DIAGRAM

SCALES: LONGITUDINAL 1"=400'
TRANSVERSE 1"=100'

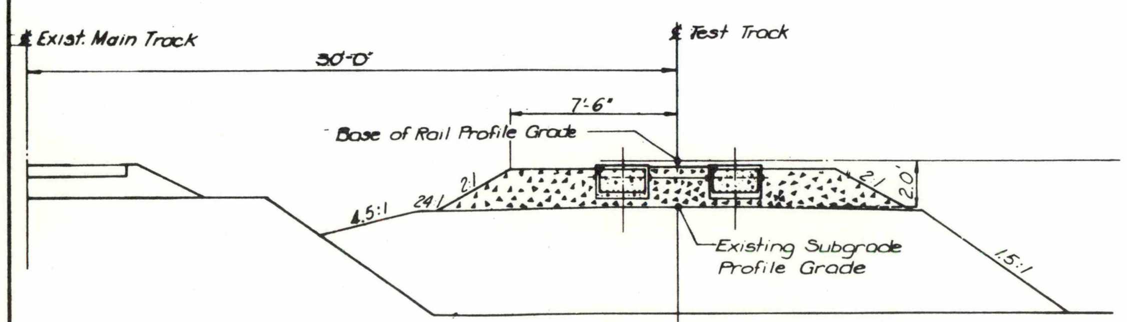


PROFILE

SCALES: HORIZ. 1"=400'
VERT. 1"= 10'

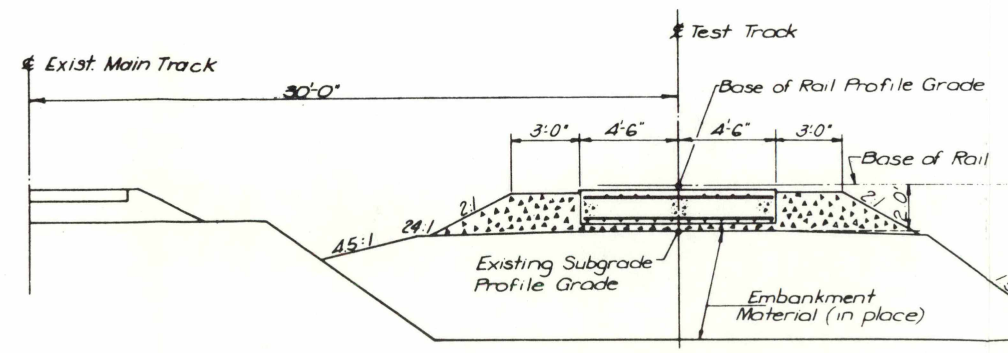
NOTE

BASE OF RAIL (PROFILE GRADE) TO BOTTOM OF SLAB = 1'-8 1/8"
BASE OF RAIL (PROFILE GRADE) TO BOTTOM OF BEAMS = 1'-7 3/8"



SECTION THRU BEAMS

SCALE: 1/4"=1'-0"



SECTION THRU SLAB

SCALE: 1/4"=1'-0"

DAVID NOVICK LICENSED 6405 KANSAS PROFESSIONAL ENGINEER

David Novick
KANSAS PROFESSIONAL ENGINEER NO. 6405

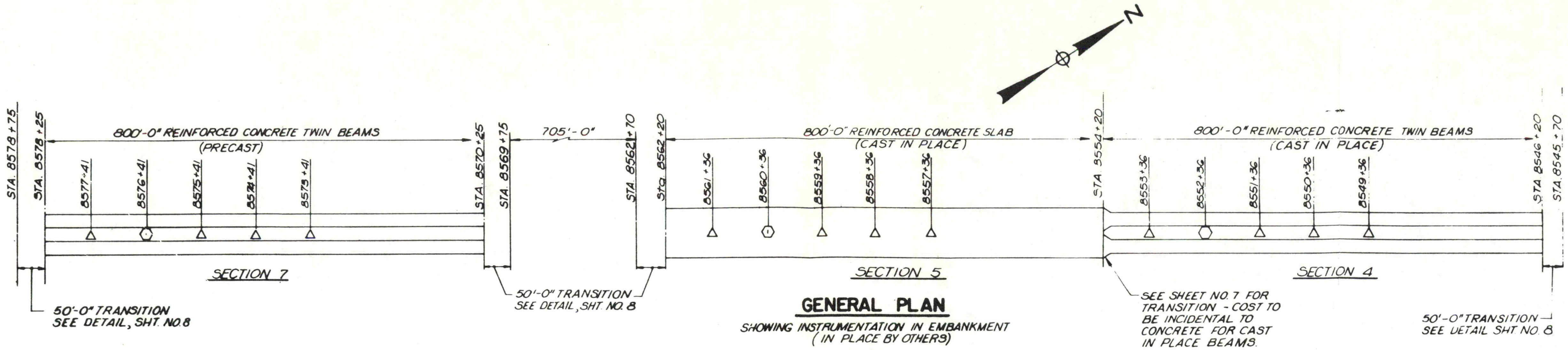
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS
DATED: MAY 4, 1972 APPROVED: C. L. Holman
ASST. GEN. MGR. - ENGINEERING

U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

GENERAL LOCATION PLAN DIAGRAM, PROFILE & TYPICAL SECTIONS

WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS

SHEET 2 OF 8
ATSF DRWG NO. EOT-6110-1



GENERAL PLAN

SHOWING INSTRUMENTATION IN EMBANKMENT
(IN PLACE BY OTHERS)

NO SCALE:

LEGEND

EMBANKMENT INSTRUMENTATION

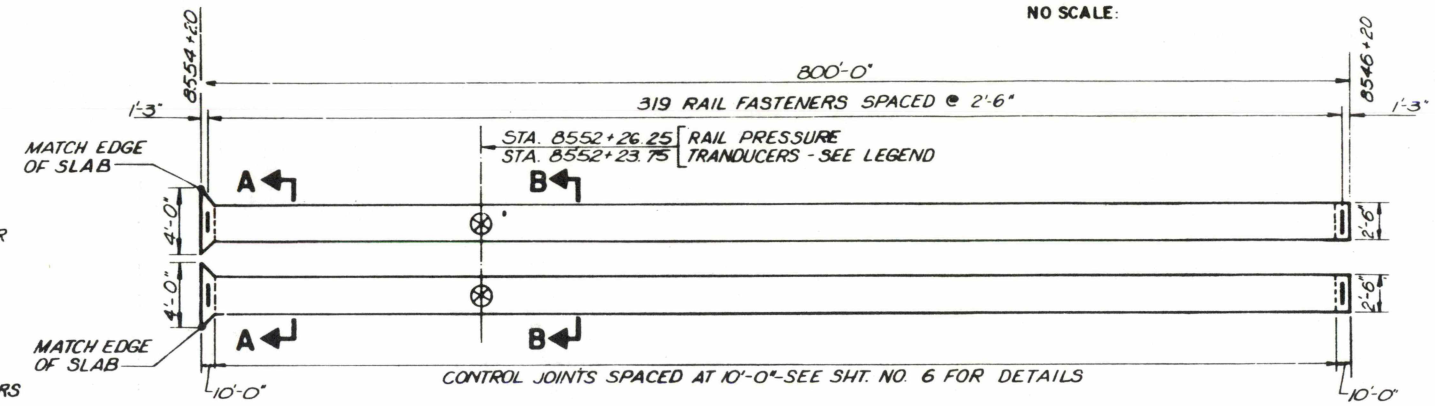
- □ PCA PRESSURE TRANSDUCER (BY OTHERS SEE DETAIL A)
- ⬡ MAIN INSTRUMENT (IN PLACE)
- △ SINGLE POSITION (IN PLACE)

STRUCTURE INSTRUMENTATION

- ⊗ RAIL PRESSURE TRANSDUCERS (INSTRUMENT BY OTHERS - CUT OUT BY CONTRACTOR (SEE DETAIL A)

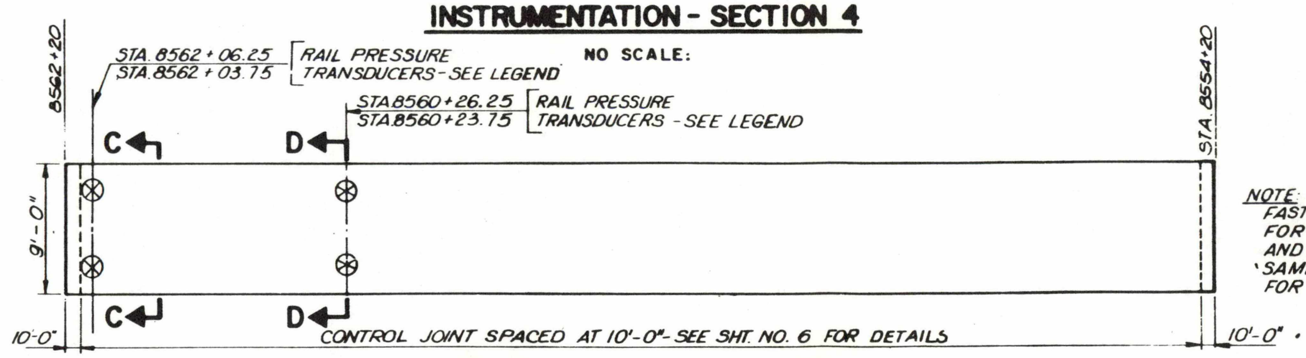
NOTE:
FOR DETAILS OF CUT OUTS FOR PRESSURE TRANSDUCERS, AND LOCATIONS OF OTHER INSTRUMENTATION NEAR TRANSDUCERS, SEE "PART LONGITUDINAL SECTIONS" ON SHEET NO. 6

FOR SECTIONS A-A, THRU D-D, SEE SHEET NO. 6



INSTRUMENTATION - SECTION 4

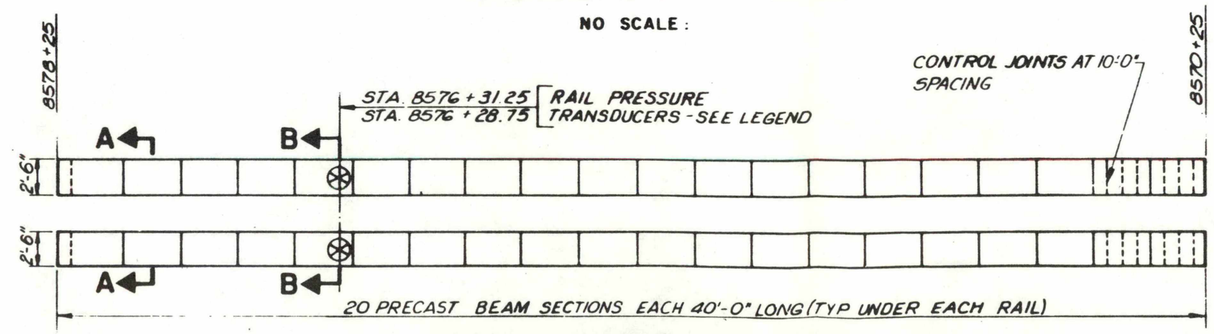
NO SCALE:



INSTRUMENTATION - SECTION 5

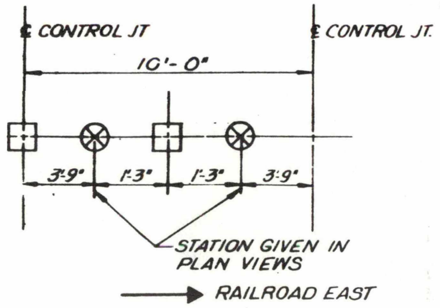
NO SCALE:

NOTE:
FASTENER SPACING FOR SECTION 5 AND SECTION 7 SAME AS SHOWN FOR SECTION 4



INSTRUMENTATION - SECTION 7

NO SCALE:



DETAIL "A"

TYPICAL INSTRUMENTATION LAYOUT

GENERAL NOTES

DESIGN SPECIFICATIONS:
AREA MANUAL FOR RAILWAY ENGINEERING - CHAPTER 8
CONCRETE STRUCTURES AND FOUNDATIONS - LATEST EDITION (1971)

DESIGN CRITERIA:
LIVE LOAD - LOCOMOTIVES - 38K PER WHEEL - SPACING VARIES
FREIGHT CARS - 35K PER WHEEL - SPACING VARIES.
IMPACT - 100% OF LIVE LOAD.
LATERAL LOADS ON RAIL - 60% (LL+I)

DESIGN STRESSES:

REINFORCED CONCRETE	$f'c$	= 4,500 PSI
	f_c	= 2,000 PSI
	v_c	= (WITHOUT WEB REINFORCEMENT) = 75 PSI
	v_c	= (WITH WEB REINFORCEMENT) = 335 PSI
	N	= 8
REINFORCEMENT:	f_s	= 24,000 PSI

NOTE:
EXCEPT AS SHOWN OR NOTED ON THE DRAWINGS AND SPECIFICATIONS, ALL DETAILS OF THE PRECAST BEAMS IN SECTION 7 ARE SIMILAR TO THE CAST IN PLACE BEAMS IN SECTION 4.

KANSAS PROFESSIONAL ENGINEER NO.

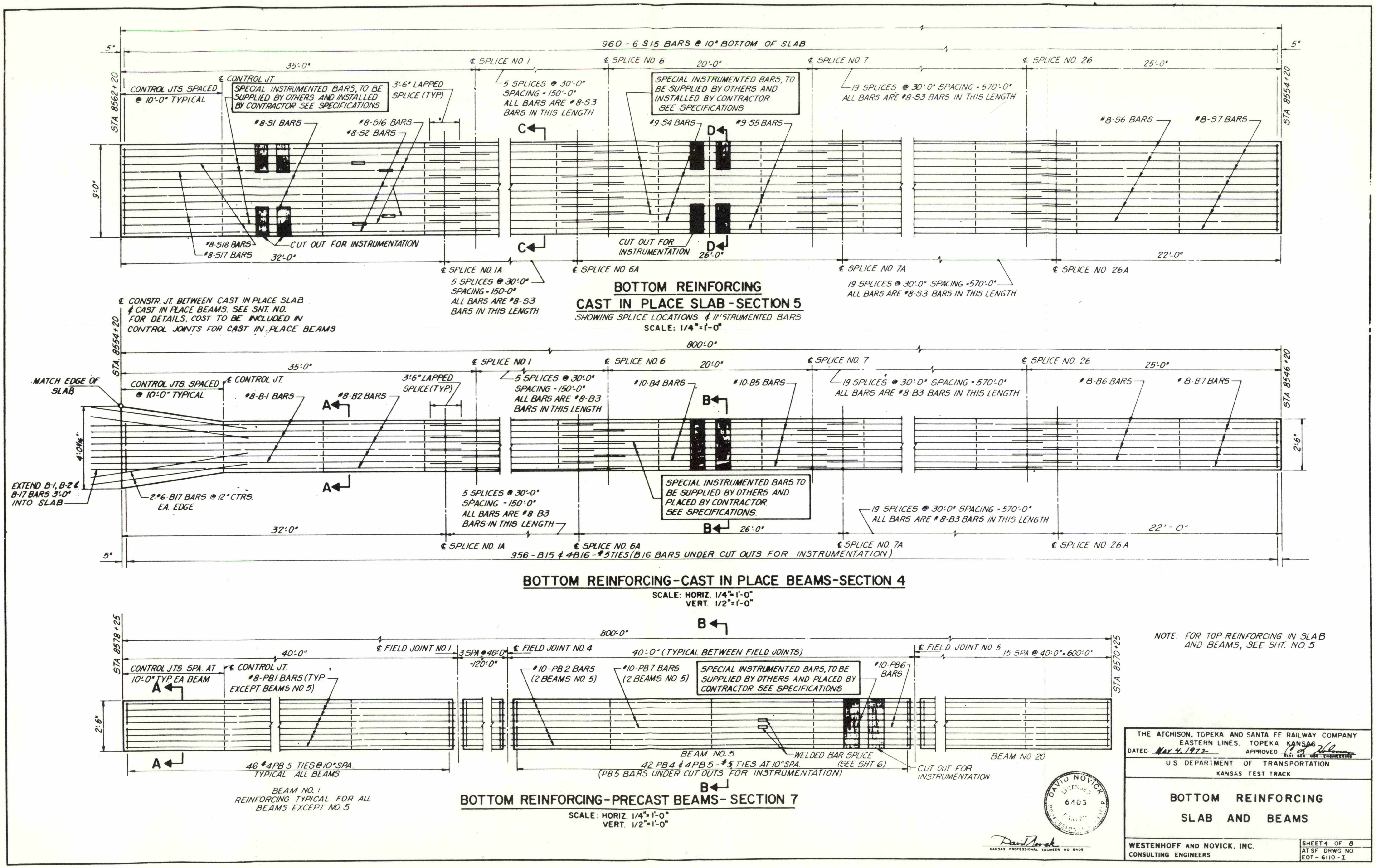
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS
DATED: MAY 4, 1972 APPROVED:

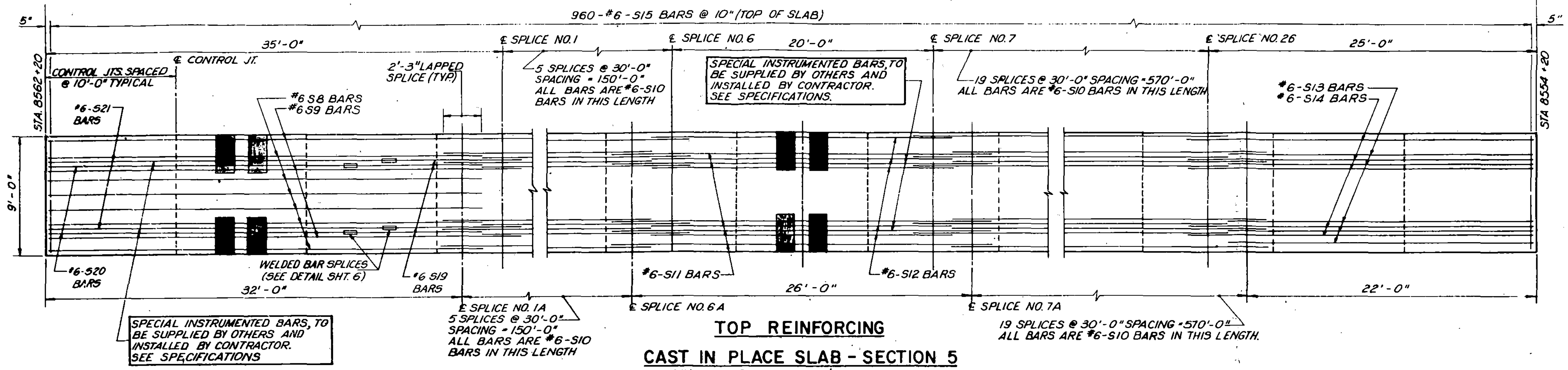
U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

INSTRUMENTATION LOCATIONS & GENERAL NOTES

WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS

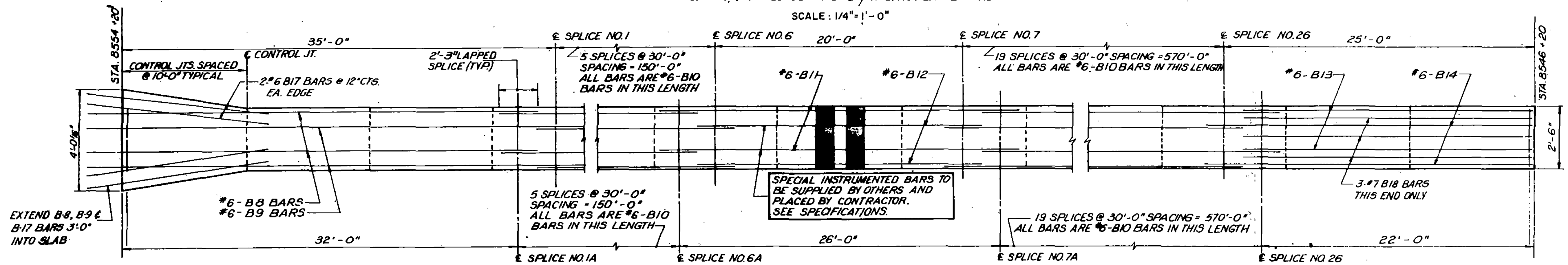
SHEET 3 OF 8
ATSF DRWG NO.
EOT-6110-I


960 - 6 S15 BARS @ 10" BOTTOM OF SLAB
5"
5"
35'-0"
℄ SPLICE NO 1
℄ SPLICE NO 6
20'-0"
℄ SPLICE NO 7
℄ SPLICE NO 26
25'-0"
STA 8562+20
STA 8554+20
℄ CONTROL JT.
CONTROL JTS. SPACED @ 10'-0" TYPICAL
SPECIAL INSTRUMENTED BARS, TO BE SUPPLIED BY OTHERS AND INSTALLED BY CONTRACTOR SEE SPECIFICATIONS
3'-6" LAPPED SPLICE (TYP)
5 SPLICES @ 30'-0" SPACING - 150'-0" ALL BARS ARE #8-S3 BARS IN THIS LENGTH
SPECIAL INSTRUMENTED BARS, TO BE SUPPLIED BY OTHERS AND INSTALLED BY CONTRACTOR SEE SPECIFICATIONS
19 SPLICES @ 30'-0" SPACING - 570'-0" ALL BARS ARE #8-S3 BARS IN THIS LENGTH
#8-S1 BARS
#8-S16 BARS
#8-S2 BARS
C
#9-S4 BARS
D
#9-S5 BARS
#8-S6 BARS
#8-S7 BARS
9'-0"
#8-S18 BARS
#8-S17 BARS
CUT OUT FOR INSTRUMENTATION
32'-0"
C
CUT OUT FOR INSTRUMENTATION
D
26'-0"
22'-0"
℄ SPLICE NO. 1A
℄ SPLICE NO. 6A
℄ SPLICE NO 7A
℄ SPLICE NO 26A
5 SPLICES @ 30'-0" SPACING - 150'-0" ALL BARS ARE #8-S3 BARS IN THIS LENGTH
19 SPLICES @ 30'-0" SPACING - 570'-0" ALL BARS ARE #8-S3 BARS IN THIS LENGTH
BOTTOM REINFORCING
CAST IN PLACE SLAB - SECTION 5
SHOWING SPLICE LOCATIONS & INSTRUMENTED BARS
SCALE: 1/4"=1'-0"
℄ CONSTR. JT. BETWEEN CAST IN PLACE SLAB & CAST IN PLACE BEAMS. SEE SHT. NO. FOR DETAILS. COST TO BE INCLUDED IN CONTROL JOINTS FOR CAST IN PLACE BEAMS
STA 8554+20
800'-0"
35'-0"
℄ SPLICE NO 1
℄ SPLICE NO. 6
20'-0"
℄ SPLICE NO. 7
℄ SPLICE NO 26
25'-0"
STA 8546+20
MATCH EDGE OF SLAB
CONTROL JTS. SPACED @ 10'-0" TYPICAL
℄ CONTROL JT.
#8-B1 BARS
A
#8-B2 BARS
3'-6" LAPPED SPLICE (TYP)
5 SPLICES @ 30'-0" SPACING - 150'-0" ALL BARS ARE #8-B3 BARS IN THIS LENGTH
#10-B4 BARS
B
#10-B5 BARS
19 SPLICES @ 30'-0" SPACING - 570'-0" ALL BARS ARE #8-B3 BARS IN THIS LENGTH
#8-B6 BARS
#8-B7 BARS
4'-0 1/16"
2'-6"
EXTEND B-1, B-2 & B-17 BARS 3'-0" INTO SLAB
2-#6-B17 BARS @ 12" CTRS. EA. EDGE
A
5 SPLICES @ 30'-0" SPACING = 150'-0" ALL BARS ARE #8-B3 BARS IN THIS LENGTH
SPECIAL INSTRUMENTED BARS TO BE SUPPLIED BY OTHERS AND PLACED BY CONTRACTOR. SEE SPECIFICATIONS.
19 SPLICES @ 30'-0" SPACING - 570'-0" ALL BARS ARE #8-B3 BARS IN THIS LENGTH
32'-0"
B
26'-0"
22'-0"
℄ SPLICE NO. 1A
℄ SPLICE NO. 6A
℄ SPLICE NO 7A
℄ SPLICE NO 26A
5"
956 - B15 & 4B16 - #5 TIES (B16 BARS UNDER CUT OUTS FOR INSTRUMENTATION)
BOTTOM REINFORCING - CAST IN PLACE BEAMS - SECTION 4
SCALE: HORIZ. 1/4"=1'-0"
VERT. 1/2"=1'-0"
B
800'-0"
STA 8578+25
40'-0"
℄ FIELD JOINT NO. 1
3 SPA @ 40'-0" -120'-0"
℄ FIELD JOINT NO. 4
40'-0" (TYPICAL BETWEEN FIELD JOINTS)
℄ FIELD JOINT NO 5
15 SPA @ 40'-0" = 600'-0"
STA 8570+25
CONTROL JTS. SPA. AT 10'-0" TYP. EA BEAM
℄ CONTROL JT.
#8-PB1 BARS (TYP EXCEPT BEAMS NO 5)
A
#10-PB2 BARS (2 BEAMS NO. 5)
#10-PB7 BARS (2 BEAMS NO. 5)
SPECIAL INSTRUMENTED BARS, TO BE SUPPLIED BY OTHERS AND PLACED BY CONTRACTOR. SEE SPECIFICATIONS
#10-PB6 BARS
2'-6"
A
BEAM NO. 5
WELDED BAR SPLICE (SEE SHT. 6)
CUT OUT FOR INSTRUMENTATION
BEAM NO 20
46 #4PB 5 TIES @ 10" SPA. TYPICAL ALL BEAMS
42 PB4 & 4PB5 - #5 TIES AT 10" SPA. (PB5 BARS UNDER CUT OUTS FOR INSTRUMENTATION)
B
BEAM NO. 1 REINFORCING TYPICAL FOR ALL BEAMS EXCEPT NO. 5
BOTTOM REINFORCING - PRECAST BEAMS - SECTION 7
SCALE: HORIZ. 1/4"=1'-0"
VERT. 1/2"=1'-0"
NOTE: FOR TOP REINFORCING IN SLAB AND BEAMS, SEE SHT. NO. 5
DAVID NOVICK LICENSED 6403 PROFESSIONAL ENGINEER KANSAS
KANSAS PROFESSIONAL ENGINEER NO 6403
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS
DATED May 4, 1972
APPROVED
U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK
BOTTOM REINFORCING
SLAB AND BEAMS
WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS
SHEET 4 OF 8
ATSF DRWG NO.
EOT - 6110 - I



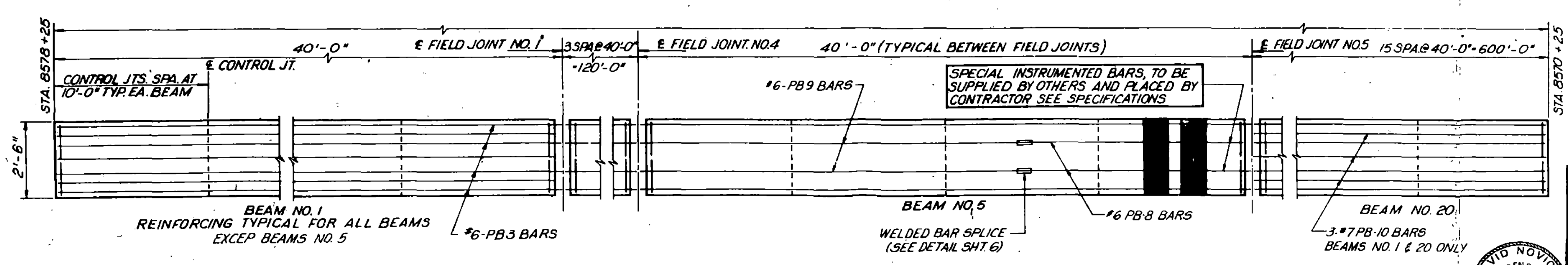
TOP REINFORCING
CAST IN PLACE SLAB - SECTION 5
SHOWING SPLICE LOCATIONS & INSTRUMENTED BARS

SCALE: 1/4" = 1'-0"



TOP REINFORCING - CAST IN PLACE BEAMS - SECTION 4
SCALE: HORIZ. 1/4" = 1'-0"
VERT. 1/2" = 1'-0"

NOTE: FOR BOTTOM REINFORCING IN SLAB AND BEAMS, SEE SHT. NO. 4



TOP REINFORCING - PRECAST BEAMS - SECTION 7
SCALE: HORIZ. 1/4" = 1'-0"
VERT. 1/2" = 1'-0"

DAVID NOVICK LICENSED 6405 KANSAS PROFESSIONAL ENGINEER

David Novick
KANSAS PROFESSIONAL ENGINEER NO. 6405

THE ATCHISON TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS
DATED: May 4, 1972 APPROVED: [signature] ASST. GEN. MGR. - ENGINEERING

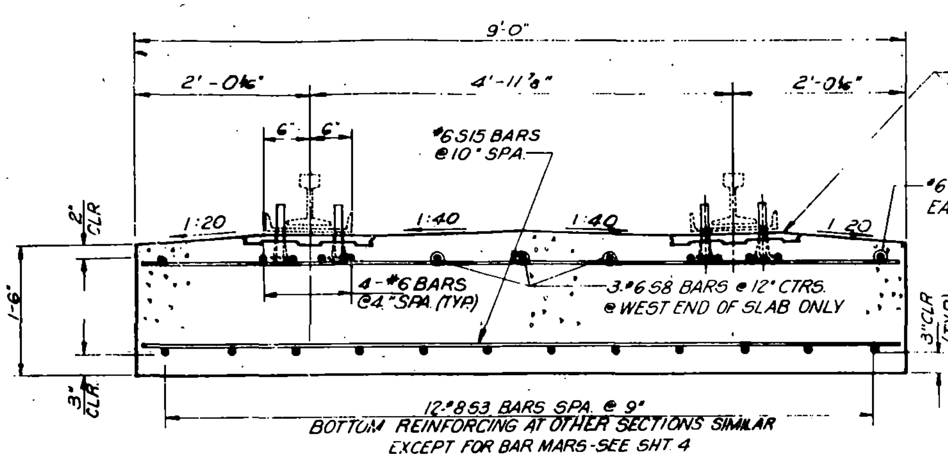
U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

NON - CONVENTIONAL TRACK STRUCTURES

TOP REINFORCING
SLAB AND BEAMS

WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS

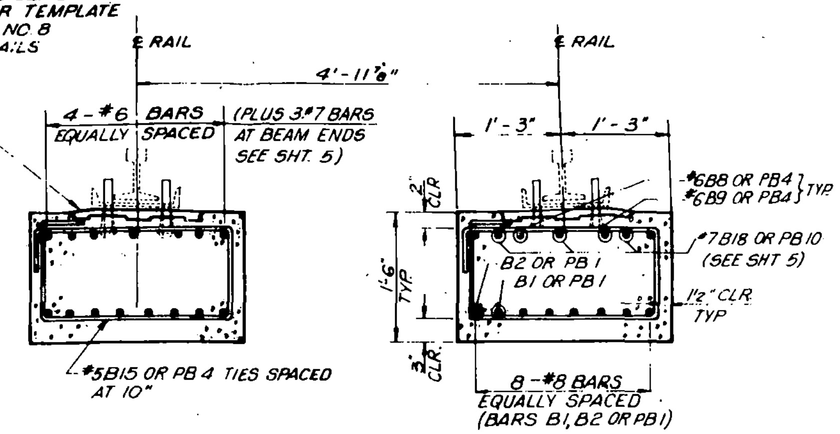
SHEET 5 OF 8
AT SF DRWG NO. EOT - 6110 - I



SECTION C-C

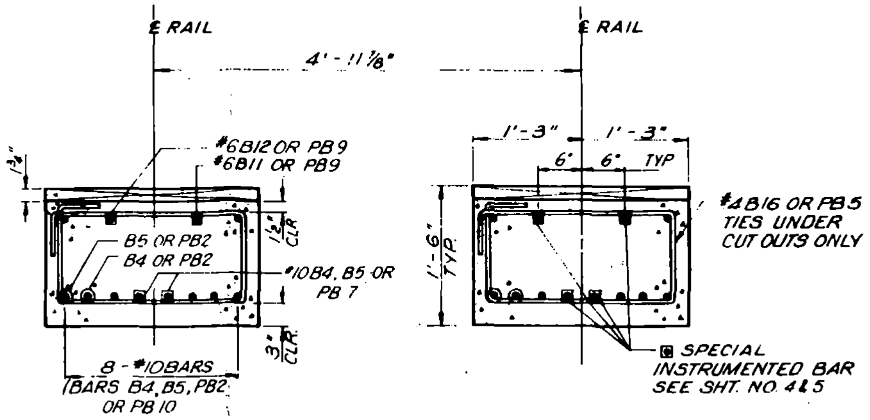
(TYPICAL SLAB SECTION)

SCALE: 1" = 1'-0"



SECTION A-A

SCALE: 1" = 1'-0"



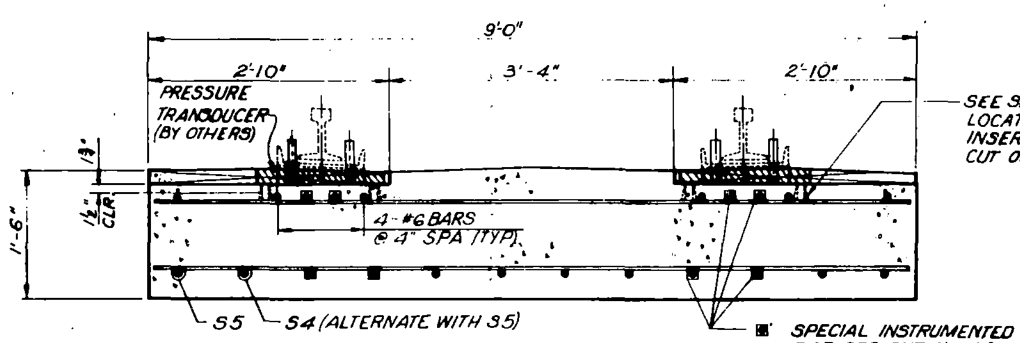
SECTION B-B

SCALE: 1" = 1'-0"

INSTRUMENTED BARS

IN SLAB 2 EA. - #8 S17, S18; #9 S4, S5; #6 S20, S21, S11, S12 - TOTAL 16
IN CAST IN PLACE BEAMS - 2 EA. - #10 B4, B5; #6 B11, B12 - TOTAL 8
IN PRECAST BEAMS - 4 EA. - #10 PB6; #6 PB8 - TOTAL 8

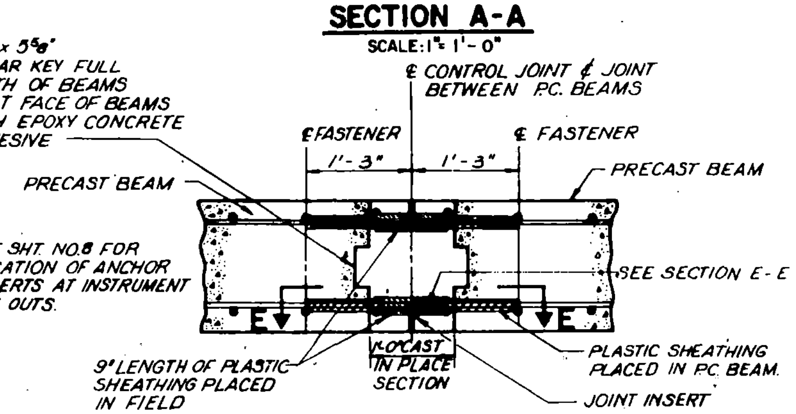
TOTAL 32 BARS



SECTION D-D

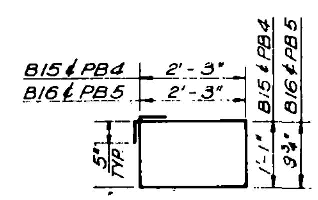
(SLAB SECTION AT CUT OUT FOR INSTRUMENTATION)

SCALE: 1" = 1'-0"



JOINT BETWEEN PRECAST BEAMS

SCALE: 1" = 1'-0"



BARS B15, B16, PB4 & PB5

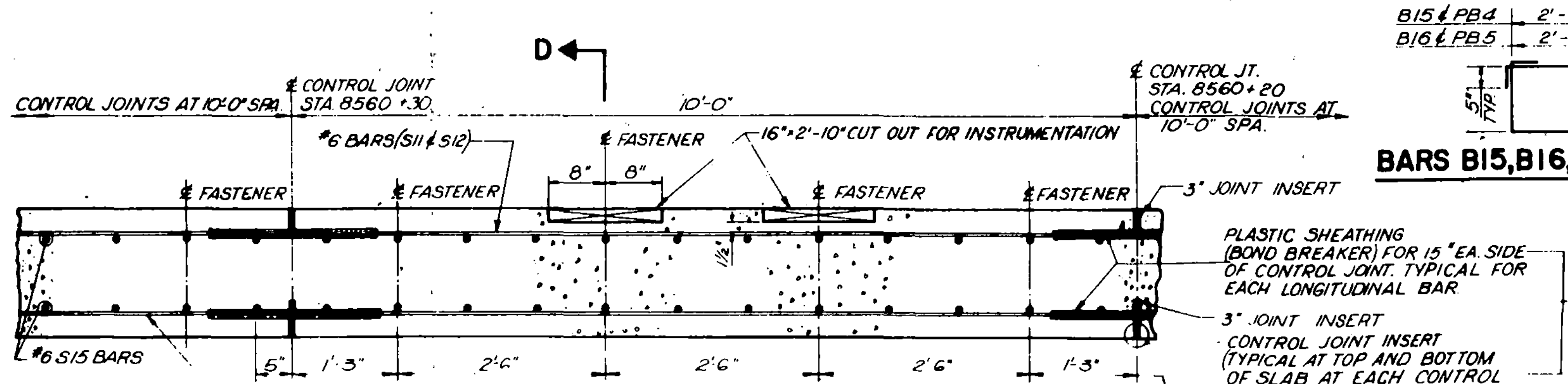
CAST IN PLACE BEAM REINFORCING				
MARK	LOCATION	SIZE	NO.	LENGTH
B1	BOTT. BEAM-LONGIT.	#8	8	36'-9"
B2	" " "	#8	6	39'-9"
B3	" " "	#8	384	33'-6"
B4	" " "	#10	8	29'-6"
B5	" " "	#10	8	23'-6"
B6	" " "	#8	6	23'-9"
B7	" " "	#8	8	26'-9"
B8	TOP BEAM-LONGIT.	#6	4	36'-3"
B9	" " "	#6	4	39'-3"
B10	" " "	#6	192	32'-3"
B11	" " "	#6	4	28'-3"
B12	" " "	#6	4	22'-3"
B13	" " "	#6	4	23'-0"
B14	" " "	#6	4	26'-0"
B15	BEAM TIES	#5	1912	7'-6"
B16	" "	#5	8	7'-0"
B17	EDGE BARS	#6	8	15'-0"
B18	TOP BEAM-LONGIT.	#7	6	23'-0"
PRECAST BEAM REINFORCING				
MARK	LOCATION	SIZE	NO.	LENGTH
PB1	BOTT. BEAM-LONGIT.	#8	304	39'-10"
PB2	" " "	#10	12	39'-10"
PB3	TOP BEAM LONGIT.	#6	156	39'-10"
PB4	BEAM TIES	#5	1912	7'-6"
PB5	" "	#5	8	7'-0"
PB6	BOTT. BEAM-LONGIT.	#10	4	14'-11"
PB7	" " "	#10	4	24'-11"
PB8	TOP BEAM-LONGIT.	#6	4	14'-11"
PB9	" " "	#6	4	24'-11"
PB10	" " "	#7	12	39'-0"

SLAB REINFORCING				
MARK	LOCATION	SIZE	NO.	LENGTH
S1	BOTT. SLAB-LONG.	#8	4	33'-9"
S2	" " "	#8	4	36'-9"
S3	" " "	#8	288	33'-6"
S4	" " "	#9	6	29'-6"
S5	" " "	#9	6	23'-6"
S6	" " "	#8	6	23'-9"
S7	" " "	#8	6	26'-9"
S8	TOP SLAB-LONGIT.	#6	6	33'-0"
S9	" " "	#6	3	36'-0"
S10	" " "	#6	240	32'-3"
S11	" " "	#6	5	28'-3"
S12	" " "	#6	5	22'-3"
S13	" " "	#6	5	23'-0"
S14	" " "	#6	5	26'-0"
S15	TOP & BOTT-TRANSV.	#6	1920	8'-8"
S16	BOTT SLAB-LONGIT.	#8	4	10'-0"
S17	" " "	#8	2	26'-9"
S18	" " "	#8	2	23'-9"
S19	TOP SLAB-LONGIT.	#6	4	10'-0"
S20	" " "	#6	2	23'-0"
S21	" " "	#6	2	26'-0"

NOTES:

SEE SHTS. NO. 3, 4 & 5 FOR LOCATION OF SECTIONS A-A THRU D-D.

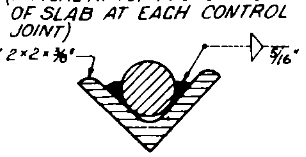
SECTIONS A-A AND B-B ARE IDENTICAL FOR CAST IN PLACE AND PRECAST BEAMS, EXCEPT FOR REINFORCING BARS MARKS.



PART LONGITUDINAL SECTION AT SLAB INSTRUMENTATION

SHOWING INSTRUMENT CUT OUTS, CONTROL JOINTS AND TYPICAL FASTENER SPACING LONGITUDINAL SECTION THRU BEAMS SIMILAR EXCEPT FOR REINFORCING

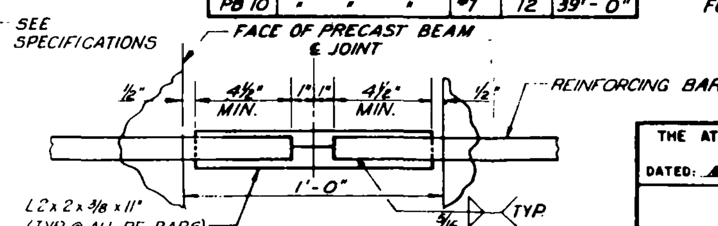
SCALE: 1" = 1'-0"



WELDED BAR SPLICE

TYPICAL AT JOINTS BETWEEN PRECAST BEAMS AND SPLICES IN INSTRUMENTED BARS.

SCALE: HALF SCALE



SECTION E-E

PLAN VIEW SHOWING TYPICAL WELD DETAIL SEE ALSO "WELDED BAR SPLICE", THIS SHEET.

SCALE: 1" = 1'-0"

DAVID NOVICK LICENSED 6405 KANSAS PROFESSIONAL ENGINEER

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS
DATED: May 4, 1972 APPROVED:

U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

CONCRETE & REINFORCEMENT DETAILS

WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS

SHEET 6 OF 8
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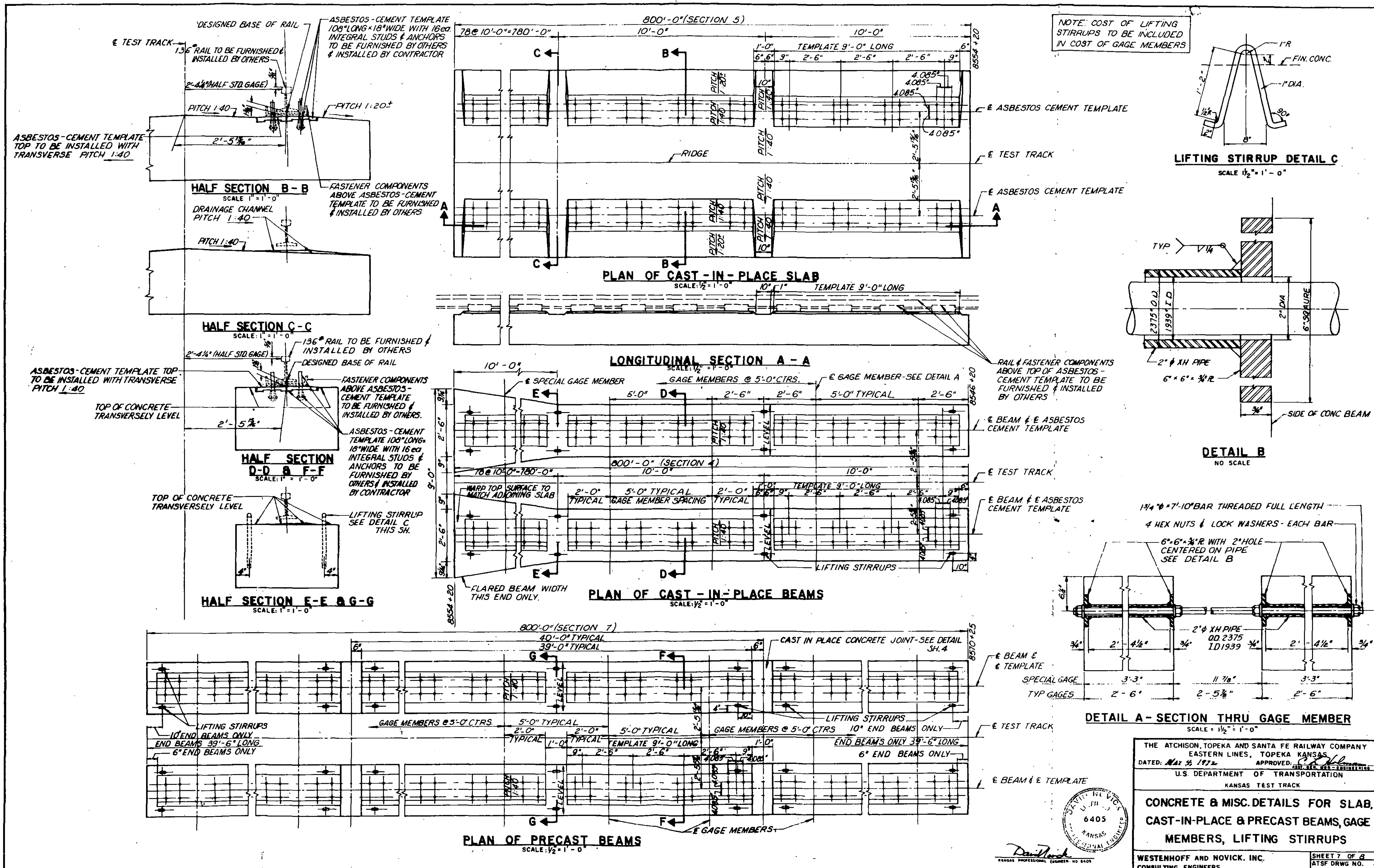
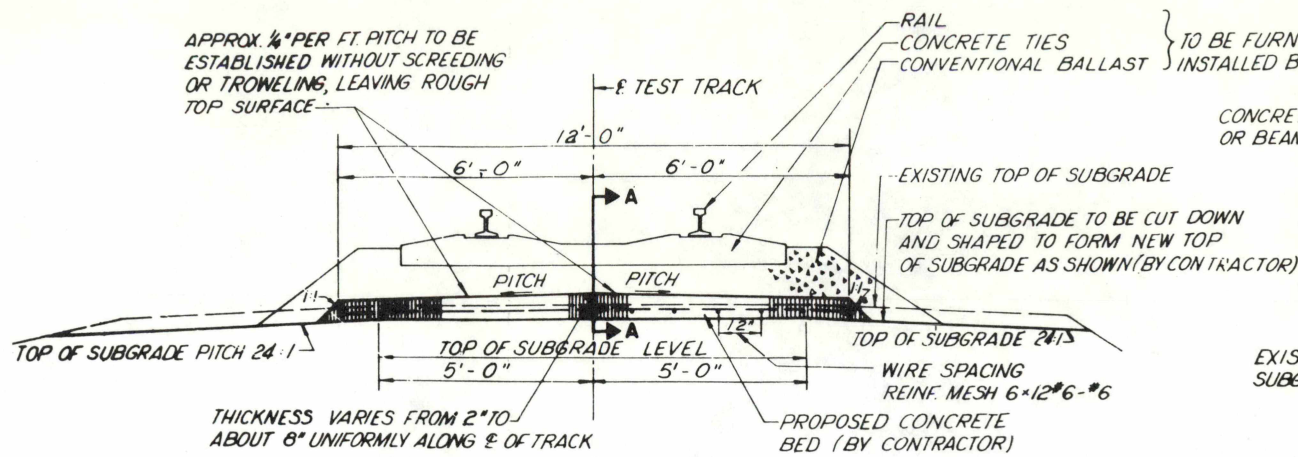
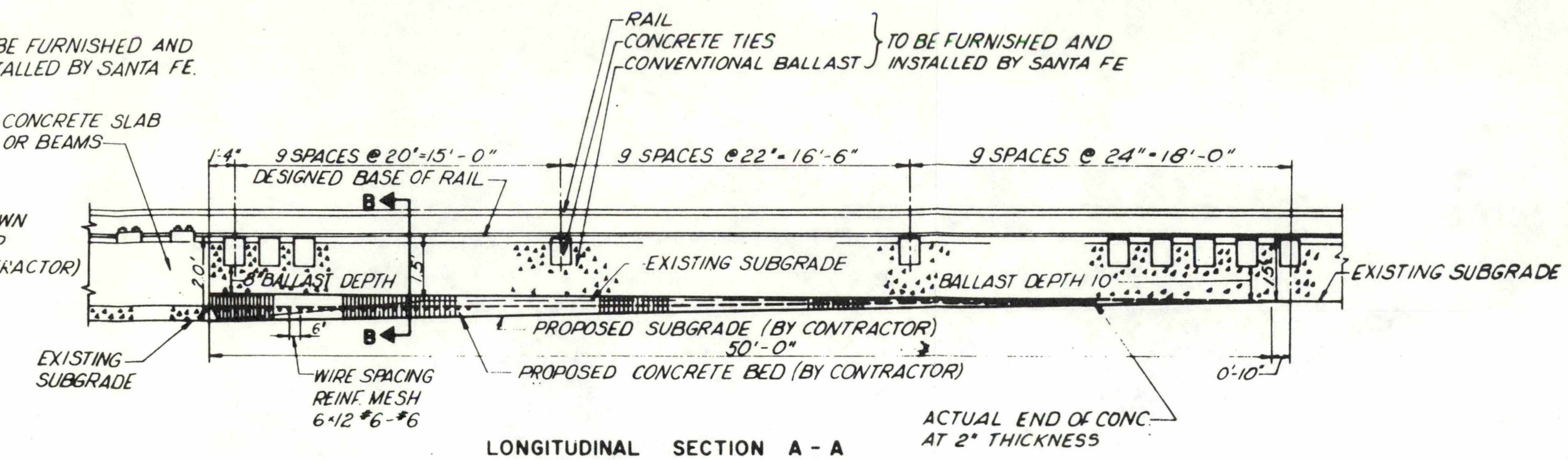

HALF SECTION B-B
SCALE 1" = 1'-0"
HALF SECTION C-C
SCALE: 1" = 1'-0"
HALF SECTION D-D & F-F
SCALE: 1" = 1'-0"
HALF SECTION E-E & G-G
SCALE: 1" = 1'-0"
PLAN OF CAST-IN-PLACE SLAB
SCALE: 1/2" = 1'-0"
LONGITUDINAL SECTION A-A
PLAN OF CAST-IN-PLACE BEAMS
SCALE: 1/2" = 1'-0"
PLAN OF PRECAST BEAMS
SCALE: 1/2" = 1'-0"
LIFTING STIRRUP DETAIL C
SCALE 1 1/2" = 1'-0"
DETAIL B
NO SCALE
DETAIL A - SECTION THRU GAGE MEMBER
SCALE = 1 1/2" = 1'-0"
NOTE: COST OF LIFTING STIRRUPS TO BE INCLUDED IN COST OF GAGE MEMBERS
800'-0" (SECTION 5)
800'-0" (SECTION 4)
800'-0" (SECTION 7)
78 @ 10'-0" = 780'-0"
TEMPLATE 9'-0" LONG
℄ ASBESTOS CEMENT TEMPLATE
℄ TEST TRACK
RIDGE
DESIGNED BASE OF RAIL
ASBESTOS-CEMENT TEMPLATE 108" LONG × 18" WIDE WITH 16 ea. INTEGRAL STUDS & ANCHORS TO BE FURNISHED BY OTHERS & INSTALLED BY CONTRACTOR
136# RAIL TO BE FURNISHED & INSTALLED BY OTHERS
2'-4 1/4" (HALF STD. GAGE)
PITCH 1:40
PITCH 1:20±
ASBESTOS-CEMENT TEMPLATE TOP TO BE INSTALLED WITH TRANSVERSE PITCH 1:40
FASTENER COMPONENTS ABOVE ASBESTOS-CEMENT TEMPLATE TO BE FURNISHED & INSTALLED BY OTHERS
DRAINAGE CHANNEL PITCH 1:40
TOP OF CONCRETE TRANSVERSELY LEVEL
LIFTING STIRRUP SEE DETAIL C THIS SH.
RAIL & FASTENER COMPONENTS ABOVE TOP OF ASBESTOS-CEMENT TEMPLATE TO BE FURNISHED & INSTALLED BY OTHERS
℄ SPECIAL GAGE MEMBER
GAGE MEMBERS @ 5'-0" CTRS.
℄ GAGE MEMBER - SEE DETAIL A
℄ BEAM & ℄ ASBESTOS CEMENT TEMPLATE
WARP TOP SURFACE TO MATCH ADJOINING SLAB
FLARED BEAM WIDTH THIS END ONLY
LIFTING STIRRUPS
CAST IN PLACE CONCRETE JOINT - SEE DETAIL SH. 4
40'-0" TYPICAL
39'-0" TYPICAL
10 END BEAMS ONLY
END BEAMS 39'-6" LONG
6" END BEAMS ONLY
℄ BEAM & ℄ TEMPLATE
℄ GAGE MEMBERS
1 3/4" φ × 7'-10" BAR THREADED FULL LENGTH
4 HEX NUTS & LOCK WASHERS - EACH BAR
6" × 6" × 3/4" ℞ WITH 2" HOLE CENTERED ON PIPE SEE DETAIL B
2" φ XH PIPE OD 2375 ID 1939
SPECIAL GAGE
TYP GAGES
SIDE OF CONC BEAM
FIN. CONC.
8554+20
8546+20
8570+25
THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS
DATED: MAY 4, 1972 APPROVED:
U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK
CONCRETE & MISC. DETAILS FOR SLAB, CAST-IN-PLACE & PRECAST BEAMS, GAGE MEMBERS, LIFTING STIRRUPS
WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS
SHEET 7 OF 8
ATSF DRWG NO. EOT-6110-I



TRANSVERSE SECTION B - B

SCALE: 1/2 = 1'-0"

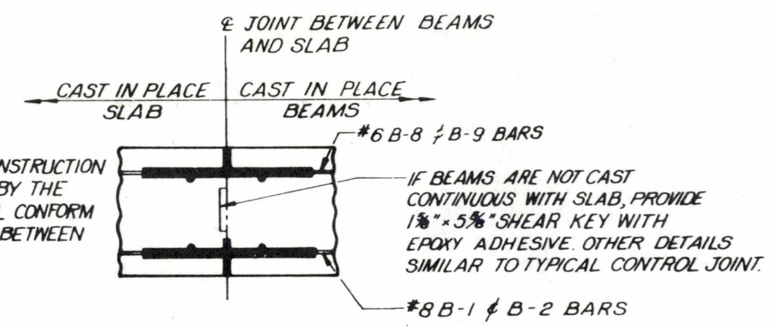


LONGITUDINAL SECTION A - A

SCALE: HORIZ. 1/4" = 1'-0"
VERT. 1/2" = 1'-0"

TRANSITION DETAILS (4 REQD.)

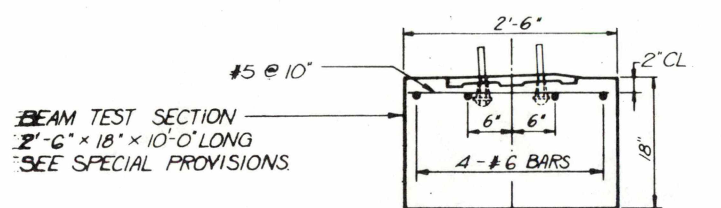
SCALES: AS SHOWN



NOTE: LOCATION OF ANY OPTIONAL CONSTRUCTION JOINTS SHALL BE APPROVED BY THE ENGINEER AND DETAILS SHALL CONFORM TO THOSE SHOWN FOR "JOINT BETWEEN BEAMS AND SLAB".

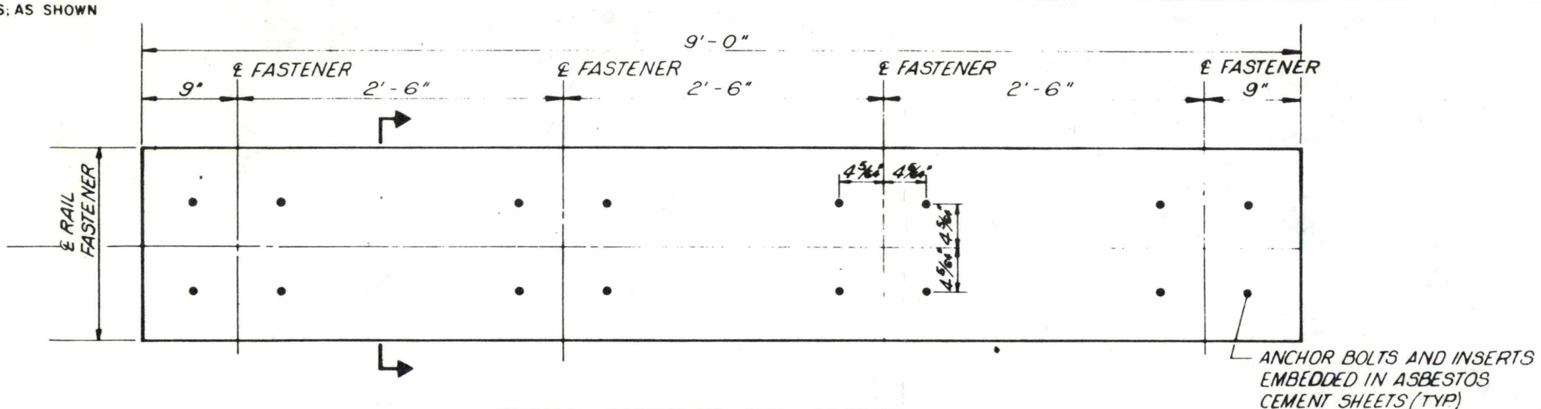
JOINT BETWEEN BEAMS AND SLAB

SCALE: 1" = 1'-0"



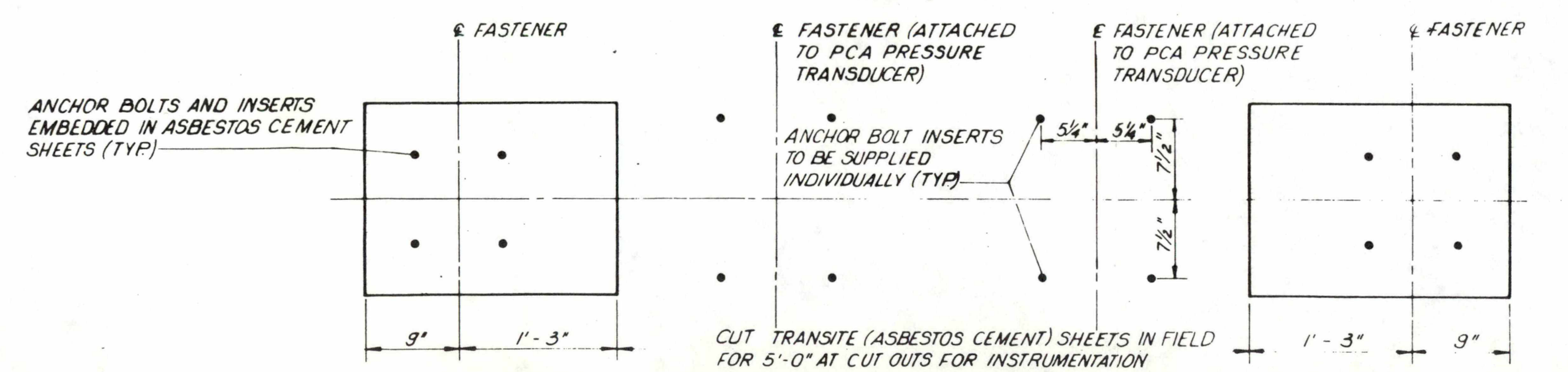
BEAM TEST SECTION

SCALE 1" = 1'-0"



TYPICAL FASTENER SUB - ASSEMBLY

SCALE: 1½" = 1'-0"



SUB - ASSEMBLIES AT INSTRUMENT CUT - OUTS

SCALE: 1½" = 1'-0"

NOTE:
FASTENER SUB-ASSEMBLIES, INCLUDING TRANSITE SHEETS WITH EMBEDDED ANCHOR INSERTS, BOLTS, NUTS, AND WASHERS, WILL BE SUPPLIED TO CONTRACTOR BY THE RAILWAY COMPANY, BUT WILL BE INSTALLED BY CONTRACTOR.

DAVID NOVICK LICENSED 6405

David Novick
KANSAS PROFESSIONAL ENGINEER NO. 6405

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS
DATED: May 4, 1972 APPROVED: [signature]

U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

TRANSITION MISCELLANEOUS DETAILS

WESTENHOFF AND NOVICK, INC.
CONSULTING ENGINEERS

SHEET 8 OF 8
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10.2 Special Provisions

Section	SP-1	General
Section	SP-2	Concrete, Test Sections 4, 5 and 7
Section	SP-3	Reinforcing Steel, Test Sections 4, 5 and 7
Section	SP-4	Control Joints, Test Sections 4, 5 and 7
Section	SP-5	Fastener Sub-Assemblies, Test Sections 4, 5 and 7
Section	SP-6	Ballast, Test Sections 4, 5 and 7
Section	SP-7	Gage Members, Test Sections 4 and 7
Section	SP-8	Field Construction Joints, Test Section 7
Section	SP-9	Transitions
Section	SP-10	Beam Test Sections

SPECIAL PROVISIONS

The work embraced herein shall be performed in accordance with the ATSF Standard Specification for Earthwork and Structures, CES 50153-58. All Sections of the Specifications which are cited are applicable except as they may be modified, amended, or added to by these Special Provisions. Unless otherwise noted, references herein refer to the Standard Specifications.

Section SP-1-GENERAL

Modification, amplification and additions to Section I - General Provisions of the Standard Specifications.

SP-1.1 General. The provisions of the Standard Specifications, Section 1, shall apply except as modified and supplemented herein.

SP-1.2 Description of the Entire Project. The proposed construction under this contract, as generally described in the Invitation for Bids, is a part of a single line test track to be constructed by the Railway Company in cooperation with the U. S. Department of Transportation. The entire project, including work already completed, includes an embankment with instrumentation, the track support structures, ballast, ties, rail fasteners, rail, and instrumentation for the test track structures.

The embankment and structural instrumentation will be monitored over a period of several years under main line traffic operations for the purpose of evaluating the performance of the three track support structures. It is essential for the success of the entire project to produce structures which are accurately placed with respect to line, grade and cant of rail, as well as high quality construction in general. Field observations will be made by the Engineer to assure accurate placement, in addition to materials testing.

SP-1.3 Time of Completion. The Contractor shall start construction within ten (10) days of notice to proceed and complete the work within seventy-five (75) calendar days unless time for completion is extended as provided elsewhere in the contract.

SP-1.4 Program of Construction. The Contractor shall submit to the Engineer in writing, within five (5) days of Notice to Proceed, his program of construction, which shall show the dates at which the various operations and parts will be started and completed, in sufficient detail to enable the Engineer to judge the adequacy of such program. This program may be modified from time to time as may be necessary in order to secure such results.

The Contractor shall coordinate his operations with the installation of instrumentation by the Portland Cement Association as further described in Section SP-1.6.

SP-1.5 Contractor's Methods. The Contractor's procedure and methods of construction may be of his own selection except as specified herein, provided they will, in the judgement of the Engineer, secure results which satisfy the requirements of the Plans and Contract Documents. Permission by the Engineer to use any particular device or method of construction shall not relieve the Contractor from full responsibility for any failure or inaccuracies resulting from the use thereof. Approval of details of construction, or of lists or bills of material, shall not relieve the Contractor from full responsibility to secure results in conformity with the Plans and Contract Documents. It is expressly understood that the Contractor is in all respects an independent contractor for this work, notwithstanding under certain conditions he is bound to follow the directions of the Engineer, and is in no respect an agent, servant, or employee of the Railway Company.

SP-1.6 Cooperation of the Contractor. It will be noted that certain instrumentation is to be placed both in the existing embankment and within the track support structures. Some of this will be done before the start of construction operations, and other instrumentation will be placed during the construction period.

It is anticipated that all soil pressure sensors and junction boxes, to be place by PCA within the embankment, will be installed prior to the construction operations. The four areas of test installation will be properly marked. It will be the obligation of the contractor to exercise the necessary precaution to prevent damage to the in-place instrumentation.

The instrumentation to be placed during the construction operation is affixed by others to the reinforcing bars. Precaution must be taken during the placing of the bars not to damage the gages and wrappings affixed to the bars. After the bars have been placed, it will be necessary for Portland Cement Association representatives to work in the instrumented areas for about one day for each of the four test areas, to install lead wires.

The Contractor is instructed to insure that no damage occurs to these gages and lead wires during construction. This will require that the concrete be placed carefully by hand in all instrumented areas. In vibrating the concrete, precaution must be taken to avoid contact between the vibrator and the instrumented portions of the reinforcing steel or the lead wires.

During the Contractor's casting operations, the sensors will be monitored by others and the Contractor will be informed of damaged instrumentation.

SP-1.7 <u>Damage to Existing Instrumentation</u>. In addition to the instrumentation mentioned above, the existing embankment already contains delicate and sensitive instrumentation whose approximate locations are shown on the plans. Wiring for these instruments is embedded in the embankment and runs to readout stations on the side of the embankment.

The Contractor will be required to take proper measures to protect this existing and all new instrumentation to avoid damage due to his operations. In case of damage, he shall make good such damage, at his own expense, in a manner acceptable to the Engineer. It will be understood that "instrumentation" will include all measuring devices within the embankment and structures, all wiring leading to recording devices, and the recording devices themselves.

SP-1.8 <u>Inspection</u>. The Engineer will provide a full time staff to observe construction and to sample and test materials. Materials testing will include tests of concrete constituents; property tests of plastic concrete;

flexural and compressive tests of the concrete to determine short term strength and strength gain characteristics, and tensile tests of reinforcing steel. The Contractor shall provide provisions for storage on the job site of concrete specimens for long term testing, as directed by the Engineer. Close observation will be made of operations involving placement of fastener sub-assemblies to assure accurate placement.

The Contractor shall provide the services of one laborer to assist the Engineer as required, for the term of this contract.

SP-1.9 Information to be Furnished by Contractor. Delete Section 1.6.4 of the Standard Specifications.

The Contractor shall furnish a program of construction within five (5) days of Notice to Proceed as provided elsewhere in these Special Provisions.

The Contractor shall furnish with his proposal a list of equipment which he expects to place on this project and he shall quote rental rates per hour on each unit of equipment for use in paying for Force Account Work. Such rental rates shall be fully operated rates and include machine, operator, supplies, fuel, repairs, taxes, overhead, etc. Rental will be paid only for time machines are engaged in doing Force Account Work. The list of equipment shall be provided on the prescribed format included with the contract documents.

The Contractor shall promptly furnish the Engineer, when requested, a force report and rates of pay.

The Contractor shall prepare whatever detailed working drawings are necessary to enable him to construct all parts of the work in conformity with the plans and specifications. For all general construction, working drawings shall illustrate details and methods of construction which are left to the Contractor's choice or which are not shown on the plan drawings. Working drawings shall show the project name and location, the Contractor's and Railway Company's names, and the location of the parts of the work. They shall be signed or initialed to indicate they have been checked, and shall be submitted to the Engineer for review in triplicate. One set will be returned to the Contractor with approvals, or showing changes or corrections required, and copies in triplicate shall be resubmitted after correction until they are approved. If additional copies have been specified in the plans or in these specifications they shall be furnished.

The Contractor shall report any errors found in the plans to the Engineer, who will make or approve the necessary corrections. The Company shall not be responsible for errors on the Contractor's drawings, even though approved, or for minor discrepancies of the plan drawings. Payment for working drawings, revisions thereof, and for copies furnished shall be included in the amounts bid for materials or work. The Contractor shall furnish as many sets of prints of working drawings as the Company may need for the work.

SP-1.10 Office and Utilities. Before starting work at the site, the Contractor shall provide one field office for laboratory testing and suitable furniture and equipment for two field offices.

A second field office, less furniture and equipment, will be furnished by others. The Contractor shall maintain both of these offices thereafter until the completion of the work to be done under this Contract. These offices shall be separate buildings located, as directed, where they will not interfere with the progress of the work. An approved, suitably constructed and equipped trailer of adequate size and design for the purpose or satisfactory existing rented quarters may be furnished as the field office for testing.

The field office for testing shall be of suitable height and of ample size to accommodate the furniture and equipment listed below, without crowding (at least 160 sq. ft. of floor area). It shall be weathertight; the walls and roof shall be insulated with at least 1/2-in. insulating board suitably ventilated; and the floor shall be tight and of double-thick construction. The office shall have at least three screened windows which can be both opened and locked shut, and the door shall have a cylinder lock with two keys. There shall also be a screen door.

The Contractor shall provide acceptable toilet facilities for the exclusive use of the Resident Engineer.

The Contractor shall furnish the following furniture, equipment, supplies and services for both offices:

One plan table or sloping plan shelf, about 3 ft. by 5 ft., with a reasonably smooth top, and one suitable stool.

Two chairs.

Shelves as directed.

Electric lights and outlets as directed. The Contractor shall pay all charges for the energy used.

Stove or other approved source of heat, for which the Contractor shall supply all fuel used.

Lockers for clothes and equipment.

Broom and dustpan.

Desk, about 3 ft. by 5 ft., with desk chair.

Plan rack, as directed.

Four-drawer, legal-size, vertical filing cabinet with lock.

Private-line telephone. The Contractor shall pay all charges for local calls.

Carbon-dioxide-type fire extinguisher of at least 4-lb. capacity.

Ten-inch oscillating fan.

Typewriter on a substantial stand.

Adding machine.

Calculating machine.

Refrigerant-type air conditioner of sufficient capacity to maintain room temperature at no more than 80 degrees Fahrenheit.

Supply of drinking water in a suitable cooler or other approved container.

Janitor service.

Paper cups, paper towels, liquid soap, and toilet paper; each with suitable dispenser or holder.

First Aid Kit.

The field office for laboratory testing shall also be as designated above except for the following:

The facilities shall include water supply and sufficient storage facilities for curing concrete test specimens under water for a period of 28 days.

Items from the above list of furniture, equipment, supplies and services may be de eted only as directed by the Engineer.

On the completion of the Contract, the Contractor shall raze and remove the office provided by him, and remove all furniture and equipment furnished by him. All such temporary facilities are to be removed from the site, the same to become his property (except the office provided by others), and leave the premises in a condition acceptable to the Engineer.

SP-1.11 <u>Alterations of Plans or Change in Amounts of Work.</u> The Engineer shall have the right, at any time during the progress of the work, to make such increases or decreases in quantities and such alterations in the details of construction, including alterations in the grade or alignment of the embankment or structure or both, as may be found necessary or desirable. Such increases or decreases and alterations shall not invalidate the Contract nor impair the bond or release the sureties thereof, and the Contractor agrees to accept the work altered, the same as if it had been a part of the original work.

Unless increases and decreases in quantities and alterations in Plans materially change the character of the work to be performed or the cost thereof, the altered work shall be performed as a part of the Contract and will be paid for at the same contract prices as for other parts of the work. The term "materially change", above, for purposes of intent under the Contract shall be construed to appy only to the following circumstances:

(1) When the character of the work as altered differs in kind or nature from that involved or encountered in the original proposed construction.

(2) When the total amount increase or decrease in quantities of a major Contract bit item affected by the work as altered varies from those same individual items in the Proposal by more than twenty (20) percent.

If the character of the work or the unit costs thereof are "materially changed" as defined above, an allowance will be made on such basis as may have been agreed to in advance of the performance of the work involved, or in case no such basis has been previously agreed upon, then an equitable adjustment in the contract price will be made.

No claim shall be made by the Contractor for any loss of anticipated profits because of any such alteration, or by reason of any variation between approximate quantities and the quantities of work as done.

When the accepted quantities of work vary from the quantities in the proposal the Contractor shall accept as payment in full, so far as Contract items are concerned, payment at the original contract unit prices for the accepted quantities of work done. If the altered or added work is of sufficient magnitude as to require additional time in which to complete the project, the Assistant General Manager, Engineering, may extend the time of completion by the amount of additional time which, in his judgement, is required.

SP-1.12 Fair Employment Practice. The Contractor shall not discriminate against any applicant for employment under this Contract or against any employee performing work under this Contract because of the race, color, sex, religion, ancestry, or nation origin of such person. The Contractor shall notify all labor unions which represent employees working under this Contract of its policy of non-discrimination in employment and shall publicize his policy of non-discrimination to all his supervisors, foremen and employment agencies which refer applicants for employment under this Contract to the Contractor. The Contractor shall include the provisions of this Paragraph SP-1.12 with the substitution of the word "Subcontractor" for the "Contractor" in all his subcontracts hereunder.

The Contractor and Subcontractors shall execute the Fair Employment Practices Certification included with the contract documents and furnish any other evidence of compliance with the Fair Employment Practices requirements as may be required by the Engineer.

SP-1.13 Traffic Delays. Certain operations by the Contractor may be delayed by railroad traffic. The Contractor shall take such delays into account in preparation of bid prices and shall agree that he will make no claim for extra compensation due to such delays.

SP-1.14 Clearing of Track. The Contractor shall provide sufficient personnel and equipment to promptly clear the tracks of debris, hauling across the tracks, or any other cause, and shall keep the tracks clear.

As directed by the Engineer, the Railway Company will furnish flagmen at no cost to the Contractor to assist the Contractor in maintaining safe and unimpeded rail operation.

SP-1.15 The contract prices shall include full compensation for all costs incurred under this section.

SPECIAL PROVISIONS

Section SP-2. CONCRETE, TEST SECTIONS 4, 5 and 7.

Section 6 of the Standard Specifications shall apply except as herein modified.

SP2.1 Compressive Strength. All concrete, including that used for the precast beams in Section 7, shall have a minimum 28-day compressive strength of 4500 psi. The proportions of the concrete mix shall be as specified in Section 6.3 of the Standard Specifications. The Contractor shall perform certified tests as required to determine all proportions not specified in Section 6.3. Certified tests shall also be performed as required to determine the proper proportions of any admixtures, if any, to be used in the final work or in the Beam Test Sections as specified in section SP-10. The results of these tests shall be supplied to the Engineer and the final mix submitted to him for approval.

SP-2.2 Slump. Section 6.3.7 of the Standard Specifications shall be modified to provide a minimum slump of 3 inches and a maximum slump of 5 inches, as determined by "Slump Cone" per A.S.T.M. Designation: C143. The "Kelly Ball" test, A.S.T.M. C360, will not be allowed.

SP-2.3 Placing. Section 6.5.2 of the Standard Specifications shall apply, except that in the area of the fastener sub-assemblies, concrete shall be placed from one side of the sub-assembly only. The concrete shall be placed by shovel or by other means acceptable to the Engineer, in order to achieve a complete bond between the concrete and the bottom of the sub-assemblies, free of voided areas unfilled with concrete.

If inspection through the "knock-outs" provided in the asbestos cement sheets indicates voided areas, the Contractor shall place non-shrink grout through these inspection holes to fill such voided areas. The non-shrink grout shall be as specified in Section SP-7 for Field Construction Joints between the precast beams.

SP-2.4 Compacting. In addition to the provisions of Section 6.5.4 of the Standard Specifications, finger vibrators shall be provided as required by the Engineer to assure proper compaction and elimination of void areas at the underside of the fastener sub-assemblies. As provided in Section SP-10, the Contractor will be required to provide test beams to determine the adequacy of the proposed construction methods in eliminating these voided areas.

SP-2.5 Handling of Precast Beams. The precast beam used in test Section 7 shall not be lifted or otherwise moved from their casting beds until the concrete attains a minimum compressive strength of 3000 psi, or have cured a minimum of 14 days. This time period may be reduced if high early strength cement or other methods are used to accelerate the strength gain characteristics of the mix. Any such methods proposed by the Contractors shall be submitted in writing to the Engineer for approval.

The Contractor shall exercise extreme care at all times in handling the precast beams to avoid excess cracking or other damage from improper or inadequate handling. Particular care shall be taken so as to prevent lifting of the beams at their center section only. Additional lifting stirrups may be provided by the Contractor as deemed necessary and as approved by the Engineer. The Contractor shall be paid only for those beams accepted at the job site by the Railway Company.

Minimal cracking may be expected at the control joints and should not be cause for rejection. The Engineer shall have final authority as to acceptance of all beams.

SP-2.6 Casting of Precast Beams. The precast beams shall be cast upon 1 inch of ballast to be furnished by the Railway Company in order to provide an artifically roughened undersurface. Any ballast adhering to the bottom of the beam upon lifting shall remain. Other methods of providing an artifically roughened surface will be acceptable upon approval by the Engineer. The beams may be precast on the job site, but they may not be precast in their final positions.

SP-2.7 Bedding of Precast Beams. The Contractor shall be responsible for the accurate placement, uniform sound bedding (preserving drainage through the ballast) and complete installation of the precast beam system as shown on the drawings, all in a manner satisfactory to the Engineer. This includes but is not necessarily limited to adjustment for true alignment and surface, accurate placement on station, gaging system, and continuous joint systems. The gaging system and continuous joints shall be as specified and paid for in SP-7 and SP-8.

Before starting this work, the Contractor shall furnish in writing to the Engineer a detailed description of the methods and procedures he proposes to use. Work shall not be started prior to the Contractor's receipt of the Engineer's written approval of the proposed methods and procedures. The Engineer's approval in no way relieves the Contractor of any responsibility.

The precast beams shall be accurately placed as described in Section SP-5.1 on line and station, leaving a 12 inch (plus or minus 1/4 inch) opening between abutting beams. Gage members shall then be installed, with care to avoid beam movement. Loose standard ballast shall then be placed to a depth of about 5 inches above the bottom of the beams across the full width between beams and for a width of 2 feet along the field side of each line of beams. Adjustment in alignment and surface as required to assure true line and grade shall then be performed by utilizing the lifting stirrups and oak blocks and wedges. Manually operated vibrating tampers shallthen be used at about ten foot intervals to flow the sub-ballast in tight against the bottom of the beams to provide firm support for the beams in their adjusted alignment and surface position. Reinforcing bars at the beams ends shall then be spliced by welding as shown on the drawings and specified in Section SP-8.2.1. The joints between abutting beams shall then be completed, using non-shrink grout. The final tamping for the entire beam length shall then be done using manually operated bibrating tampers to provide uniform compacted sub-ballast support to the beams longitudinally and transversely, while maintaining proper alignment and grade.

SP-2.8 Method of Payment. Concrete shall be paid for at the contract unit price per cubic yard of concrete in place, for the three types of track structure as indicated in Bid Items 1,2 and 3. This price shall be full compensation for furnishing all materials, unless otherwise specified, and for all equipment, tools, falsework, curing, forms, bracing, labor for surface finish and all other times of expense required to complete the concrete work shown on the plans, with the exception of reinforcing steel. This price shall also include the cost of hauling and accurately placing and bedding the precast beams.

The unit price cost for concrete shall not include the cost of furnishing or installing gage members, lifting stirrups, control joints, or field construction joints, all as specified elsewhere in these Special Provisions.

The cost of any grouting required after inspection shall not be measured for payment but shall be incidental to the cost of Bid Items 1, 2 and 3.

SPECIAL PROVISIONS
SECTION SP-3 - PLACING REINFORCING, TEST
SECTIONS 4, 5 & 7

The applicable provisions of the Standard Specifications, Section 6, Concrete Construction shall apply except as herein modified.

SP-3.1 General. All steel reinforcing, except in test beams will be furnished to the Contractor by the Railway Company. The steel will be furnished on cars at the Aikman, Kansas siding. The applicable provisions of Section 1.5 of the Standard Specifications shall also apply.

Where there is anticipated delay in depositing concrete, the reinforcing steel shall be protected from the weather as directed by the Engineer. Before depositing concrete, the reinforcement shall be reinspected and cleaned when necessary.

As described in Section SP-1 of these Special Provisions, the Contractor shall coordinate placing of instrumented bars with the Portland Cement Association, and take special care in protecting such bars from the weather and his construction operations.

SP-3.2 Welding of Instrumented Bars. Where shown on the Plans, the Contractor shall field weld splices in the instrumented bars. The welding shall be in accordance with the latest provisions of the "Recommended Practice for Welding Reinforcing Steel Metal Inserts and Connections in Reinforced Concrete Construction" (AWS D 12.1-61) by the American Welding Society.

SP-3.3 Basis of Payment. The provisions of Section 6.15.4 of the Standard Specifications shall apply except the cost of furnishing the reinforcing shall not be included. Any additional cost of protecting reinforcing from the weather or precautions to protect instrumented bars shall be incidental to the unit price cost for Bid Items 4, 5 and 6.

SPECIAL PROVISIONS
SECTION SP-4 - CONTROL JOINTS TEST
SECTIONS 4, 5 & 7

SP-4.1 General. Control joints shall comply with the details shown on the plans and shall be spaced at the interval so indicated. All control joints shall be perpendicular to the centerline of the slab or beam and to the top or bottom surface of the member.

Control joints may consist of forming the joint by placing a strip of polyethylene sheeting across the full width of the beam or slab. This sheeting shall have a minimum thickness of 10 mils (0.010 inch) and a minimum width of 3 inches for the top insert and 3 inches for the bottom insert. The joint material must be such that it will not react adversely with chemical constituents of the concrete. Use of any other joint material shall require prior approval by the Engineer.

The joint insert material shall be such that when placed vertically in the concrete at both top and bottom of the slab or beams, it will not bond with the concrete and will form an effective weakened plane joint of 3 inches depth. The joint material shall be placed in such a manner that the edge of the strip is not more than 1/4 inch from the top or bottom surface of the slab or beam. The joint material shall not be deformed from a vertical position, either in the installation or in subsequent finishing operations performed on the concrete. Holes or slots shall be cut in both the top and bottom joint material where required so as to permit passage of the longitudinal reinforcing bars through the joint. The slots or holes shall be only sufficiently large so as to pass the reinforcing bars, without permitting free passage of wet concrete.

For a length as shown on the plans, all reinforcing bars passing through the joint shall be given a coating of an approved heavy cup grease, and an approved plastic sleeve of the length shown on the plans shall also be placed over each reinforcing bar. The greased portion of the reinforcing bars shall be free to slide in the plastic sleeve. Any excess grease on the sleeve or bar shall be removed.

SP-4.2 Method of Measurement. Control joints shall be measured per joint, which shall include insert material and plastic sheathing.

SP-4.3 Basis of Payment. Control joints shall be paid for at the contract unit price for each joint in place, or per Bid Items 7, 8 and 9. This price shall include full compensation for furnishing all labor, materials, tools and equipment, and performing all the work involved in forming the joint, including greasing of reinforcing bars, placement of plastic sheathing and inserting the joint material, and any other incidental work.

SPECIAL PROVISIONS
SECTION SP-5 - INSTALLATION OF FASTENER SUB-ASSEMBLIES

SP-5.1 General. The Contractor's attention is directed to the fact that the track support structures to be constructed will ultimately carry all main line traffic of the Railway Company. This combined with the non-conventional nature of the structures, will require extreme accuracy with respect to alignment, grade, and cant of the individual rails. These factors are in turn dependent upon the accuracy of placement of the Fastener Sub-Assemblies.

As described in the Invitation for Bids, the Railway Company shall establish and set horizontal and vertical control points, from which the test track centerline and base of rail profile grade will be determined.

The Railway Company shall also be responsible for all other lines, grades, and pitches including but not necessarily limited to, asbestos cement sheets and fastener studs and all anchor bolts, pitch for slab drainage, transverse level top for beam concrete, asbestos cement sheet cant, true alignment and surface.

Each and every one of the asbestos cement sheets 9 feet long will be furnished with seven (7) pre-cut "knock-outs" approximately 3-inch diamter to provide inspection points and for additional concrete grouting if required. It will be the Contractor's responsibility to remove these "knock-outs" at the site. Care shall be taken so as to prevent damage to the sub-assembly.

It shall be the Contractor's responsibility to accurately locate the fastener sub-assemblies. The longitudinal centerline between anchor bolts for each line of rail fasteners may be used as a reference with respect to location of the centerline of track. As shown on the plans, the top surface of the asbestos cement sheet shall be placed so as to have a 1:40 slope downward towards the centerline of track. A tolerance of $\pm$ 1/16 inch shall be allowed in both horizontal and vertical alignment in a longitudinal distance of 5 feet as measured along the centerline between anchor bolts for each line of rail fasteners at the top surface of the asbestos cement sheets. A tolerance in the 1:40 transverse slope of the asbestos cement sheets shall be allowed between 1:38 and 1:42.

Before concrete is poured, the location of all sub-assemblies to be located therein, shall be approved by the Engineer. The Contractor shall make any and all adjustments as required, to the satisfaction of the Engineer. Supporting jigs shall be removed from the sub-assemblies immediately after initial set of the concrete.

All fastener sub-assemblies will be furnished to the Contractor by the Railway Company. The sub-assemblies will be furnished on cars at the Aikman, Kansas siding.

After acceptance by the Railway Company at the site the Contractor shall exercise extreme care at all times in handling the sub-assemblies to avoid damage from improper or inadequate handling. The Contractor shall be paid only for those sub-assemblies accepted in place by the Railway Company.

SP-5.2 Installation at Instrumented Sections. As shown on the plan, eight sub-assemblies are effected by cut outs for instrumentation. At each point of instrumentation there will be two rail fasteners on short pieces of asbestos cement sheet and two fasteners attached directly to the instrumentation. The Contractor will be responsible for placing all anchor bolt inserts for the instrumentation as well as for the short lengths of asbestos cement sheets. The anchor inserts for the instrumentation will be furnished by others.

The Contract shall seal the fastener anchor bolt holes against entrance of surface water by applying a butyl caulking or other approved sealant.

SP-5.3 Method of Measurement. Installation of Sub-Assemblies shall be measured per 9 foot x 18 inch section of asbestos cement sheeting in place, including 16 anchor bolts and 16 anchor inserts. At the section to be instrumented, installation of two short pieces of asbestos cement sheeting and two instrumentation points (including a total of 16 anchor bolts and 16 anchor inserts) shall be counted as one sub-assembly.

SP-5.4 Basis of Payment. Installation of Sub-Assemblies shall be paid for at the contract unit price for each sub-assembly, in place, as per Bid Items 10, 11 and 12. This price shall include full compensation for furnishing all labor, materials, tools and equipment for installing the sub-assemblies. This shall include any jigs or equipment necessary to assure accurate placement of the sub-assemblies and any adjustments required by the Engineer to comply with the specified tolerances with respect to line, grade and transverse slope. Cost of cutting of asbestos cement sheets at the instrumentation points (as shown on the plans) as well as removing the "knock-outs" will be incidental to the contract unit price per sub-assembly.

SPECIAL PROVISIONS
SECTION SP-6 - BALLAST, TEST SECTIONS 4, 5 & 7

SP-6.1 General. If required the ballast shall be placed in a condition satisfactory to the Engineer by filling any holes or depressions, removing humps, and grading and compacting with vibrating compactors as required.

Prior to placing forms or pouring concrete the ballast in the areas of the cast in place beams and slab shall meet all dimensions and grades to the bottom surface of the structures to plus or minus 1/2 inch.

The Contractor shall exercise special care at all times to avoid damage to the existing sub-ballast and subgrade. No vehicles, trucks nor equipment will be permitted on the subgrade unless first approved by the Engineer. Materials to be placed on the sub-ballast and subgrade shall be placed carefully and in a manner that will not gouge, scar, deform the sub-ballast or subgrade or cause settlement.

After the precast and cast in place beams are completed in final position, Railway Company standard slag ballast shall be placed and compacted using vibrating compactors, between the beams (with care to avoid disturbance or damage to gaging members) and on the field side of the beams and slab to a depth 2 inches below the top of concrete, to the cross section shown on the drawings and to a width that will accommodate and support the Railway Company straddle buggy when subsequently used by the Railway Company for installing rail.

All ballast will be furnished to the Contractor by the Railway Company. The ballast will be furnished in cars at the Cassoday, Kansas siding.

SP-6.2 Ballast at Precast Beams. For test Section 7, Precast Beams, prior to placing beams on the existing sub-ballast, the top of the compacted sub-ballast shall be brought to precise profile and cross-section finish grade to provide uniform longitudinal and transverse bedding for the precast beams. To accomplish this any holes or depressions must be filled and humps removed, and the sub-ballast graded and compacted as may be required using vibrating compactors, to meet the dimensions shown on the drawings or as may be determined by the Engineer.

SP-6.3 <u>Method of Measurement</u>. Ballast, as per Bid Items 13, 14 and 15, will be measured by the ton placed in accordance with the plans and as directed by the Engineer.

SP-6.4 <u>Basis of Payment.</u> Ballast shall be paid for at the contract unit price per ton with the designated section, as shown on the plans. This price shall be full compensation for furnishing all labor, tools, equipment, and incidentals necessary for loading, transporting and placing ballast sections to the designated lines and grades.

The cost of filling holes, removing humps, or otherwise placing the existing ballast in a suitable condition, as determined by the Engineer, shall be incidental to the cost of Bid Items 13,14 and 15. The cost of furnishing the ballast shall not be included in these pay items.

SPECIAL PROVISIONS
SECTION SP-7 - GAGE MEMBERS, TEST SECTION 4 AND 7

SP-7.1 Extent. This work shall consist of furnishing all materials, labor, tools and equipment, and performing all operations necessary for the furnishing and installing of the steel gage members in the cast in place and precast beams in Test Sections 4 and 7. Also included shall be the furnishing and placement of lifting stirrups in both beam sections, all as shown on the plans.

SP-7.2 Materials. As shown on the plans, each gage member consists of a threaded rod, two pieces of pipe welded to four end plates, and four hex nuts with lock washers. Lifting stirrups shall be bent bars as shown on the plans.

The threaded rod and end plates for the pipes shall be mild steel conforming to the requirements of A.S.T.M. Designation: A-588. The pipe shall be extra strong steel pipe which shall conform to the requirements of A.S.T.M. Designation: A-53.

The threads used on the rods shall conform to the American National Course Thread Series, Class 2, free fit. The rods shall be threaded for their full length.

The lifting stirrups shall be as detailed on the plans and shall be of steel conforming to the requirements of A.S.T.M. Designation: A-36, or alternatively may be made from reinforcing bars of A.S.T.M. Grade 60 steel.

Welding of the pipes to the end plates shall conform to the requirements of the current "Specification for Welded Highway and Railway Bridges" of the American Welding Society.

SP-7.3 Installation. When installing gaging members between cast-in place beams or between precast beams, care shall be taken to assure that no lateral movement of the beams occurs. Particular care to guard against such movement shall be taken while nuts and lock washers are being drawn down tight.

SP-7.4 Method of Measurement. Gage members shall be measured by the installed unit which shall include threaded rods, pipes and end plates, and lock nuts and washers. Lifting stirrups shall not be measured for payment, but shall be considered incidental to the cost of Bid Items 16 and 17.

SP-7.5 Basis of Payment. Gage members shall be paid for at the contract unit price per unit in place, as shown on the plans. This price shall be full compensation for furnishing all labor, materials, tools, equipment and incidentals necessary for installing the members to the satisfaction of the Engineer.

SPECIAL PROVISIONS
SECTION SP-8 - FIELD CONSTRUCTION JOINTS
TEST SECTION 7

SP-8.1 Extent. This work shall consist of furnishing all materials, labor, tools and equipment necessary to construct the 12 inch by 30 inch field construction joints between precast beams. The control joint inserts, plastic sheathing on reinforcing bars, and greasing of reinforcing bars shall not be paid for under this item but shall be paid for under Bid Item 9, Control Joints in Precast Beams. The field joints shall be constructed by field welding all reinforcing bars, coating the face of the precast beams with an epoxy concrete adhesive, and then filling between the beams with a non-shrink grout, all as shown on the plans.

SP-8.2.1 Welding of Reinforcing Bars. Welding of reinforcing bars at the splices between precast beam sections shall be as shown on the plans. All welding shall be in accordance with the latest provisions of the "Recommended Practice for Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction" (AWS D 12.1-61) by the American Welding Society.

SP-8.2.2 Epoxy Bonding Compound. As shown on the plans, the faces of the precast concrete beams shall be coated with an epoxy concrete adhesive before the cast in place beam joints are poured.

The epoxy shall be a polysulfide polymer-base concrete adhesive, supplied as a two-package system. One package shall consist of a liquid epoxy resin. The second package shall contain a liquid polysulfide polymer and a curing agent.

The adhesive, as applied, shall contain approximately one part by weight of polysulfide liquid polymer to two parts of epoxy resin. The curing agent used shall provide a pot life of the mixed adhesive (100% solids) of 15 to 35 minutes as determined with a pint of adhesive in a standard pint container at 75 degrees farenheit ambient temperature, or with 20% solvent, a pot life of 50 to 90 minutes.

The adhesive shall be supplied as an unfilled, clear resin system. The adhesive as supplied shall not contain solvents. At the time that the two adhesive components are mixed, solvents in the ratio of not more than one part by volume of solvents to four parts by volume of adhesive shall be blended. At temperatures below 75 degrees farenheit, acetone shall be used as a solvent and at a temperature over 75 degrees farenheit, toulene shall be used as the solvent.

The two-package adhesive shall be furnished by the manufacturer in containers individually marked to clearly identify each component. The manufacturer shall supply mixing instructions, specifying how the components are to be combined in parts by weight or parts by volume.

The constituents of the two components shall comply with Federal Specification MMM-B-350 a Binder, Adhesive, Epoxy Resin, Flexible. These materials shall be furnished by a company that maintains stock tested and approved by the Corps of Engineers as meeting the stated Federal Specification and the materials furnished shall be equivalent to such stock.

Prior to approval and use of the two-package adhesive materials, the Contractor shall submit a notarized certification by the formulator of these materials, stating that they meet the requirements as set forth herein.

The surface to which the adhesive is to be applied shall be rendered free of oil, dirt, and any loose concrete by wire brushing. The surface shall be free of moisture before application of adhesive.

Immediately before application, the two adhesive components shall be combined in the proportion specified by the adhesive manufacturer. The components shall be intimately blended by hand or with a slow speed motor drive mixing device. The mixture of adhesive shall next be thinned by adding and blending the solvent into the adhesive. The amount of adhesive mixed at one time shall be limited to that quantity which can be conveniently applied within the pot life of the adhesive.

The two components and solvent shall not be mixed more than thirty minutes prior to use. The resulting adhesive shall be brushed onto the concrete in a layer 5 to 10 mils thick or an average of 100 to 200 square feet per gallon. After the adhesive has been applied, concrete shall not be placed against it until the solvent has evaporated. This period will be between 30 to 60 minutes depending upon weather conditions. The adhesive must be tacky and not dry at the time of concrete application. Areas that have been allowed to dry shall be recoated before concrete is placed.

Because of toxicity of the materials, including the solvents, safety hazards exist in the handling and use of the materials. They may cause serious rash in persons sensitive to the materials. Further, in the use of solvents as cleaning aids there exists a fire and flash hazard. The Contractor shall obtain from the formulator of the materials complete instructions as to the safety, health and handling precautions that must be exercised with respect to the materials to be used, and as to the procedure that should be followed in the event that workmen come in contact with the material. Before they are permitted to proceed with the work, the workmen shall be instructed as to the hazards to which they will be exposed, the necessary safety precautions, and the procedure to be followed in the event of accidental contact with the materials.

The epoxy concrete adhesive shall also be used on any construction joints proposed by the Contractor. No such construction joint shall be used unless the contractor receives written approval from the engineer. Proposed construction joints will be allowed only at the locations of control joints as shown on the plans. The contractor shall sumit all proposals for such construction joints sufficiently in advance so as to allow sufficient time for written approval to be received.

SP-8.2.3 Non-Shrink Grout. The non-shrink grout used for filling the 12 inch gaps between precast beams shall be "Embecco", "Vibro-Soil", "Ferrolith G", "DS Grout" or another approved equal, mixed and placed in a manner specified by the manufacturer for the temperature conditions existing at the time of placement.

SP-8.3 Method of Measurement. Field construction joints shall be measured by the completed joint between precast beams, which shall include welding of reinforcement bars; supplying and applying the epoxy bonding compound, and supplying and casting the non-shrink grout. Any additional field construction joints placed at the contractors option in the other test sections shall not be measured for payment, but shall be considered incidental to the contract.

SP-8.4 Basis of Payment. Field construction joints shall be paid for at the contract unit price per joint, completed, as shown on the plans. This price shall be full compensation for furnishing all labor, materials, tools, equipment and incidentals necesary for installing the joints to the satisfaction of the engineer.

SPECIAL PROVISIONS
SECTION SP-9 TRANSITIONS

SP-9.1 Extent This work shall consist of furnishing all materials, labor, tools and equipment, and performing all operations necessary for the construction of four concrete transitions at the locations shown on the plans.

The existing ballast and sugrade shall be re-graded for a 50 foot length as shown on the plans. A variable thickness concrete mat, reinforced with welded wire fabric, shall be sloped so as to drain away from the centerline of track. A one inch layer of ballast shall be placed on the mat while the concrete is still wet, so as to provide a roughened surface for placement of the additional ballast later.

SP-9.2 Materials

SP-9.2.1 Concrete The concrete used in the mats shall be as specified in Section 6, Concrete Construction, of the Standard Specifications. The minimum 28 day compressive strength shall be 3500 psi.

SP-9.2.2 Welded Wire Fabric The welded wire fabric shall be as noted on the plans and shall conform to the requirements of A.S.T.M. Designation A-185.

SP-9.3 Method of Measurement Transitions shall be measured by the completed unit, which shall include grading, welded wire fabric, concrete mat, and one inch layer of ballast. The ballast above this point, and the concrete ties as shown on the drawings will be placed by the Railway Company.

SP-9.4 Basis of Payment Transitions will be paid for at the contract unit price per transition, completed as shown on the plans. This price shall be full compensation for furnishing all labor, materials, tools, equipment and incidentals necessary for installing the transitions to the satisfaction of the Engineer.

SPECIAL PROVISIONS
SECTION SP-10 - BEAM TEST SECTIONS

SP-10.1 Extent. This work shall consist of furnishing all materials, labor, tools and performing all work necessary for the constructing and testing beam test sections ten feet long and of the cross section similar to the cast-in-place beams (Section 4) and the precast beams (Section 7), except that reinforcing steel shall be as shown on drawings for beam test section. Asbestos cement fastener sub-assemblies shall be furnished by the Company. Reinforcing steel for the beam test sections shall be furnished by the Contractor as shown on the plans.

SP-10.2 General. The ten feet long beam test sections will be constructed for the purpose of ascertaining acceptable construction procedures and concrete mixes that will assure adequate bedding of the asbestos cement fastener sub-assemblies with absence of voided areas at the interface between the concrete and the asbestos cement sheets. The number of beam test sections to be constructed shall be the least number required to ascertain construction procedures acceptable to the Engineer. A minimum of two test beams shall be constructed. At least one shall contain suitable pozzolanic material, the proportions of which are to be determined by certified test as provided in Section SP-2. Reinforcing steel shall be as shown on the drawings for beam test sections, so that construction of the test sections will closely duplicate construction of the cast-in-place and precast beams.

Beam test sections may be constructed and tested on the site of the project or other location approved by the Engineer.

SP-10.3 Testing. One transverse saw cut shall be made through the centerline of a fastener near the center of the beam. The saw cut shall be deep enough to allow breaking of the beam, to allow inspection of the asbestos cement-concrete interface. After the saw cut has been made, the asbestos cement sheet shall be removed from both beam sections to allow more complete inspection. Care should be taken to remove the sheeting as gently as possible to minimize damage to both the concrete and the asbestos cement sheets, to avoid unsatisfactory inspections.

SP-10.4 Method of Measurement. Beam test sections shall be measured by the furnished and tested unit. Concrete and reinforcing steel used in test beams shall not be measured for payment but shall be considered as incidental to this item.

SP-10.5 Basis of Payment. Beam Test Sections shall be paid for at the contract unit price per section tested. This price shall be full compensation for furnishing and testing Beam Test Sections to the satisfaction of the Engineer.

This design report, design plans, and special provisions to the Santa Fe "Standard Specifications" complete the design portion of the engineering services for non-conventional structures in the Kansas Test Track. The analysis which formed the basis for design was executed utilizing the physical and geometric properties of each component of the rail-fastener-structure-subgrade system as described herein. Significant departures from these properties will require a re-evaluation of the applicability of the analysis for the design, and may require reanalysis and redesign of the structure if the influences of the changes are found to affect the adequacy of the design, or to be contrary to structural or operating safety.

Respectfully submitted,

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APPENDIX A

Rail Torsional Properties

APPENDIX A

RAIL PROPERTIES

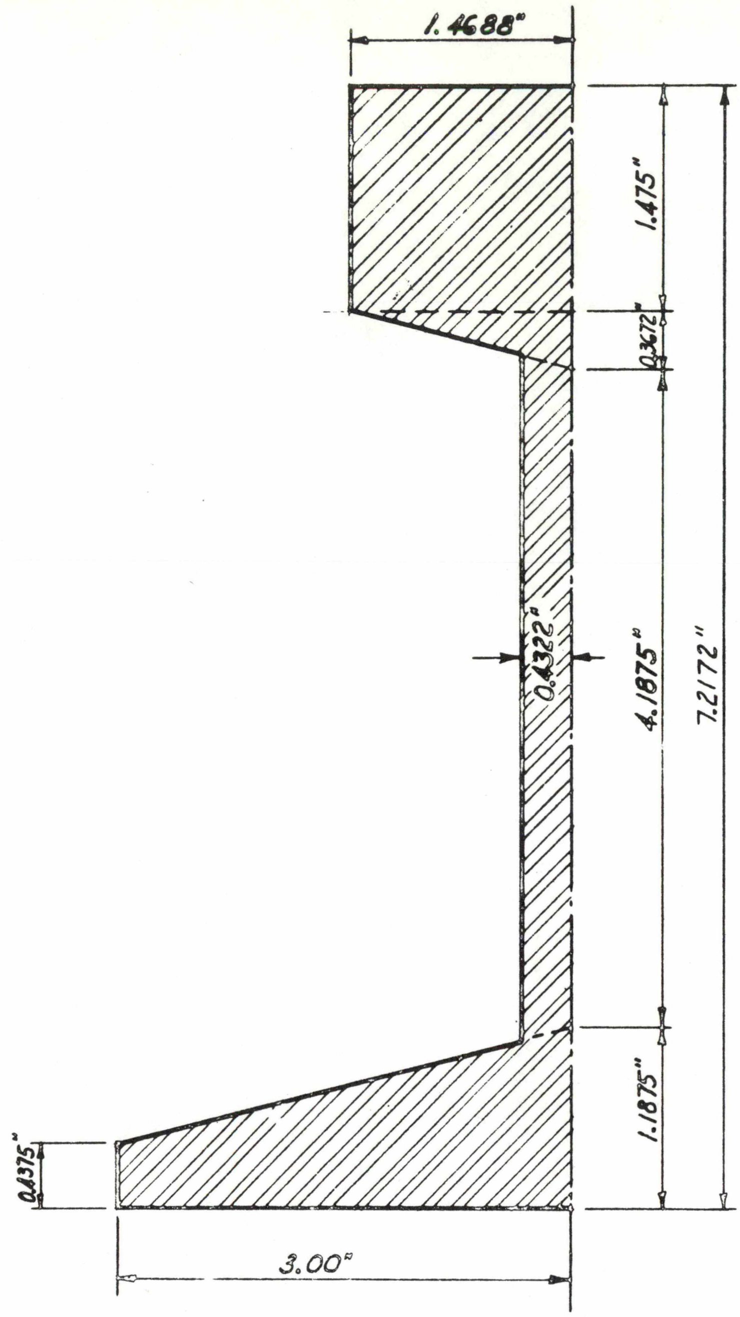
Values of rail properties were derived from three sources: 1) the literature, 2) transmitted by DOT, and 3) calculated from theory.

The literature yields a moment of inertia value of 94.9 in^4 (I_x) about a horizontal plane through the center of gravity. The moment of inertia about the vertical plane was stated to be 14.7 in^4 (I_y) by the DOT technical officer. This value was checked using the geometrical idealization shown in (a), Figure A.1. The resulting value was 14.78 in^4, which verifies the above value.

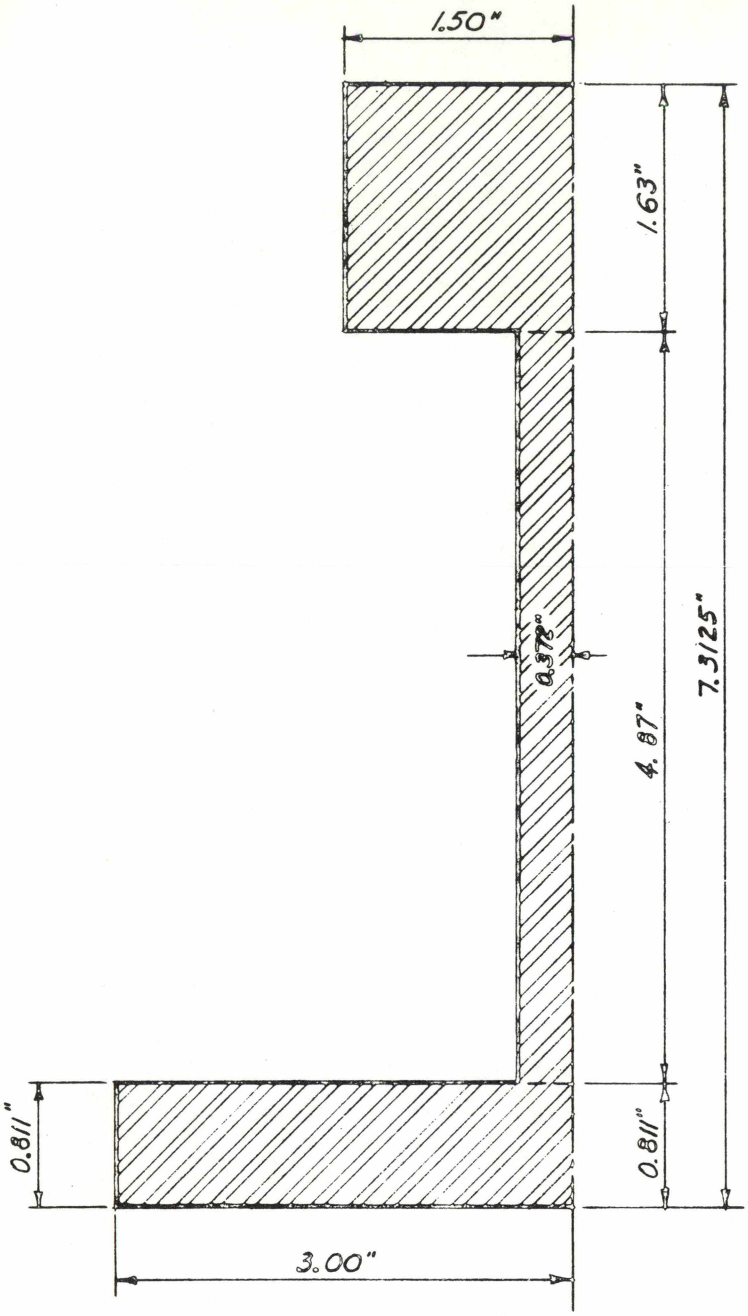
The torsional constant for the rail section was calculated using three different approaches. A lower bound value was calculated, according to the United States Steel, Steel Design Manual, using the geometrical approximation shown in (b) Figure A.1. The resulting value of 4.44 in^4 is a lower bound value due to the approximation for the head, web, and base fillet areas, which are very efficient in resisting torsional loadings.

An upper bound value was determined using St. Venant's formula, Hetenyi, and Timoshenko (Collected Papers). The resulting value was 7.24 in^4.

To determine a more exact value for use in analysis, the membrane analogy, Lyle and Johnson, and Timoshenko (Materials), was applied to the rail head, web, and base, with a resulting value of 5.68 in^4. This latter value is considered slightly conservative, and was used for design.



(a) SECTION FOR
LATERAL MOMENT OF INERTIA



(b) SECTION FOR
TORSIONAL CONSTANT
(LOWER BOUND)

SANTA FE RAILWAY - DEPT OF TRANSPORTATION KANSAS TEST TRACK		
IDEALIZED GEOMETRICAL RAIL SECTIONS		
WESTENHOFF AND NOVICK, INC. CONSULTING ENGINEERS	W1169	JUNE 1972

FIGURE A.1

APPENDIX B

Portland Cement Association Report of Static Fastener Tests

APPENDIX B

FASTENER PROPERTIES

The Portland Cement Association performed a series of static load tests on a Fastex Fastall fastener, for the DOT, to determine the load-displacement relationships of the fastener for use in analysis. These tests were performed on November 2, through November 4, 1971 at the PCA laboratory in Skokie, Illinois. A description of the test series, which was originally recommended by Westenhoff and Novick, written by PCA is contained on succeeding sheets of text and tables.

Subsequently, the data was analyzed by Westenhoff and Novick to develop the desired load-displacement and load-rotation relationships. In this analysis the centers of rotation, any movement of the centers during testing, and rigid body motion of the rail and fastener were considered. Graphs (a) through (f) on Figure B.1 show the resulting load-displacement relations. These curves assume the rail to act as a rigid body.

The resulting relations are non-linear, however, most of the properties are fairly well described by linear or bilinear approximations. In general, suitably selected tangent moduli or secant moduli may be used over the load range considered to represent "normal" operational loadings. A table of the properties used in the analysis is contained in the report.

RAIL FASTENER PERFORMANCE

The objective was to determine load-deflection relationships for a rail fastener supplied by the Fastex Division of Illinois Tool Works, Inc. To accomplish this, static loads were applied at 6 different locations and/or directions to a 136 lb. RE rail attached by the rail fastener to a concrete slab. During each load test, deflections were measured at several selected locations. This report describes the load tests and provides the load-deflection data.

Test Specimens

To conduct static load tests, the rail fastener was attached to a concrete slab. The rail fastener supplied by Fastex is fastened through four specially designed 7/8-in. diameter steel studs. The molded inserts holding these bolts and a 15 by 12 by 3/4-in. thick asbestos cement plate separating the studs were cast in an inverted position in a reinforced concrete slab 4'-0" by 2'-0" by 0'-6" thick. The concrete had a cement content of 6 bags per 1 cu. yd. and contained 3/4-in. maximum size gravel aggregate. The concrete slab was cast on October 18, 1971, and the fastener was load tested on November 2, 3, and 4, 1971. Load tests were conducted at the Cement and Concrete Research Institute laboratory in Skokie, Illinois, at an average temperature of 72 F. Before loading, a 30-in. length of a 136 lb. RE rail was attached to the concrete slab using the Fastex fastener. Each of the four 7/8-in. diameter nuts was tightened with a torque of 300 ft. lb.

Load Test Information and Results

The location of load application, locations of deflection dials, and test results are given for each load test.

Test 1

The load for Test 1 was applied vertically downward on the rail head at the center of the rail fastener. Two methods were used to apply the load. In the first method, load was applied by a hydraulic ram and measured by a calibrated load cell electrically connected to a strain indicator. In the second method, load was applied and measured by a Baldwin testing machine.

Test results from the first method using the calibrated load cell are given in Table 1. In this test, the rail was positioned in an east-west direction and

Dial 1 - measured vertical deflection of the rail head 6 in. east of load center, and

Dial 2 - measured vertical deflection of the rail head 6 in. west of load center.

After two 40,000 lb. loads were applied, the residual deflection was 0.012 in. The load-deflection relationship after these two 40,000 lb. loads is shown in Fig. 1.

Test results from the second method using the testing machine are given in Table 2. In this test, the rail was positioned in a north-south direction and

Dial 1 - measured vertical deflection of the rail head 6 in. north of load center, and

Dial 2 - measured vertical deflection of the rail head 6 in. south of load center.

Five 40,000 lb. loads were applied. The load-deflection relationships for the first and second load applications are shown in Fig. 2.

Test 2

The load for Test 2 was applied vertically downward on the rail head at a distance of 15 in. east of the center of the rail fastener. The load was applied by a hydraulic ram and measured by a calibrated load cell. Test results are given in Table 3. A load of 6,000 lb. was applied during the first application and 5,000 lb. during the second application. In this test, the rail was positioned in an east-west direction and

Dial 1 - measured vertical deflection of the rail head 6 in. west of the center of the fastener.

Dial 2 - measured vertical deflection of the rail head at the center of the fastener,

Dial 3 - measured vertical deflection of the rail head 6 in. east of the center of the fastener,

Dial 4 - measured vertical deflection of the southwest corner of the fastener clamp,

Dial 5 - measured horizontal deflection in the longitudinal or east-west direction of the southwest fastener bolt,

Dial 6 - measured horizontal deflection in the lateral or north-south direction of the northwest fastener bolt, and

Dial 7 - measured vertical deflection of the southwest fastener bolt.

Test 3

The load for Test 3 was applied laterally against the rail head at the center of the rail fastener. The load was applied by a hydraulic ram and measured by a calibrated load cell. Test results are given in Table 4.

A load of 4,000 lb. was applied during the first application and 8,000 lb. during the second application. In this test, the rail was positioned in an east-west direction, the lateral load was applied toward the north direction, and

Dial 1 - measured lateral deflection of the rail head 6 in. west of load center,

Dial 2 - measured lateral deflection of the rail head 6 in. east of load center,

Dial 3 - measured lateral deflection of the rail base 11 in. west of load center,

Dial 4 - measured lateral deflection of the rail base 11 in. east of load center,

Dial 5 - measured vertical deflection of the south side of the rail base 12 in. west of load center,

Dial 6 - measured vertical deflection of the north side of the rail base 12 in. west of load center,

Dial 7 - measured vertical deflection of the south side of the rail base 14 in. east of load center,

Dial 8 - measured vertical deflection of the north side of the rail base 14 in. east of load center,

Dial 9 - measured vertical deflection of the southwest corner of the fastener clamp,

Dial 10 - measured horizontal deflection in the lateral or north-south direction of the fastener base channel at the load center,

Dial 11 - measured horizontal deflection in the lateral or north-south direction of the southeast fastener bolt, and

Dial 12 - measured horizontal deflection in the lateral or north-south direction of the northeast fastener bolt.

Maximum deflection occurred at the rail head. Average measurements of Dials 1 and 2 are shown in Fig. 3.

Test 4

For Test 4, two lateral loads were applied in the same direction against the rail base. The loads were symmetrical 11 in. from the center of the rail fastener. Each load was applied by a hydraulic ram and measured by a calibrated load cell to be certain that the loads were equal during the test. Test results are given in Table 5. A load of 6,000 lb. was applied during the first application and 14,000 lb. during the second application. In this test, the rail was positioned in an east-west direction, the lateral loads were applied toward the north direction, and

Dial 1 - measured lateral deflection of the rail base 11 in. west of the center of the fastener,

Dial 2 - measured lateral deflection of the rail base 11 in. east of the center of the fastener,

Dial 3 - measured lateral deflection of the rail head 6 in. west of load center,

Dial 4 - measured lateral deflection of the rail head 6 in. east of load center,

Dial 5 - measured vertical deflection of the north side of the rail base 12 in. west of the center of the fastener,

Dial 6 - measured vertical deflection of the north side of the rail base 12 in. east of the center of the fastener,

Dial 7 - measured horizontal deflection in the lateral or north-south direction of the fastener base channel at its center, Dial 8 - measured horizontal deflection in the lateral or north-south direction of the northwest fastener bolt, and

Dial 9 - measured horizontal deflection in the lateral or north-south direction of the northeast fastener bolt.

Deflections of the rail head and base during the second load application are shown in Fig. 4.

Test 5

The load for Test 5 was applied laterally against the rail base at a distance of 15 in. east of the center of the rail fastener. The load was applied by a hydraulic ram and measured by a calibrated load cell. Test results are given in Table 6. In this test, the rail was positioned in an east-west direction, and

Dial 1 - measured lateral deflection of the north side of the rail base 11 in. east of the center of the fastener,

Dial 2 - measured lateral deflection of the south side of the rail base 11 in. west of the center of the fastener,

Dial 3 - measured horizontal deflection in the lateral or north-south direction of the northeast corner of the fastener base channel,

Dial 4 - measured horizontal deflection in the lateral or north-south direction of the northwest corner of the fastener base channel,

Dial 5 - measured horizontal deflection in the lateral or north-south direction of the southwest corner of the fastener base channel, Dial 6 - measured horizontal deflection in the lateral or north-south direction of the northeast fastener bolt, and

Dial 7 - measured horizontal deflection in the lateral or north-south direction of the southwest fastener bolt.

Deflection measured by Dials 1 and 2 were approximately equal. Average values are shown in Fig. 5.

Test 6

The load for Test 6 was applied longitudinally against the end of the rail base in the direction of the rail. The load was applied by a hydraulic ram and measured by a calibrated load cell. Test results are given in Table 7. In this test, the rail was positioned in a north-south direction, the longitudinal load was applied toward the south direction, and

Dial 1 - measured horizontal deflection in the longitudinal or north-south direction of the rail head,

Dial 2 - measured horizontal deflection in the longitudinal or north-south direction of the northeast fastener bolt,

Dial 3 - measured horizontal deflection in the longitudinal or north-south direction of the northwest fastener bolt,

Dial 4 - measured horizontal deflection in the longitudinal direction of the northeast corner of the fastener base channel,

Dial 5 - measured horizontal deflection in the longitudinal direction of the northwest corner of the fastener base channel,

Dial 6 - measured vertical deflection of the rail head 6 in. north of the center of the fastener, and

Dial 7 - measured vertical deflection of the rail head 6 in. south of the center of the fastener.

Longitudinal deflection of the rail is shown in Fig. 6. At a load of 5,500 lb., the rail began to slide. At the end of the test, the elastomeric pad extended 0.3 in. from the fastener at the end opposite the load application.

Torque in Fastener Bolts

Before conducting the load tests, each of the four fastener nuts was tightened with a torque of 300 ft. lb. After testing, the torques were 255, 240, 265, and 290 ft. lb.

TABLE 1

RESULTS OF TEST 1
VERTICAL LOAD DOWNWARD ON RAIL HEAD AT CENTER OF RAIL FASTENER USING CALIBRATED LOAD CELL

Load (Lb.)	Downward Deflection (0.001 in.) Dial 1	Dial 2	Average of Dials 1 & 2
0	0	0	0
0*	12	12	12
1000	13	14	13.5
2000	14	15	14.5
3000	15	15	15
4000	15	16	15.5
5000	16	17	16.5
10000	21	21	21
15000	27	27	27
20000	32	32	32
25000	37	37	37
30000	42	42	42
35000	47	45	46
40000	53	50	51.5
0	14	14	14

*After 2 Loads to 40,000 Lb.

TABLE 2

RESULTS OF TEST 1
VERTICAL LOAD DOWNWARD ON RAIL HEAD AT CENTER
OF RAIL FASTENER USING TESTING MACHINE

Load (lb.)	Downward Deflection (0.001 in.) Dial 1	Dial 2	Average of Dials 1 & 2
0	0	0	0
1000	1	1	1
2000	2	1	1.5
5000	5	4	4.5
10000	10	9	9.5
15000	16	16	16
20000	20	23	21.5
25000	25	30	27.5
30000	30	37	33.5
35000	35	43	39
40000	40	50	45
0	2	8	5
5000	7	12	9.5
10000	12	17	14.5
15000	17	24	20.5
20000	21	30	25.5
25000	25	36	30.5
30000	29	42	35.5
35000	34	48	41
40000	38	54	46
0	2	13	7.5
40000	37	57	47
0	2	14	8
40000	36	58	47
0	2	15	8.5
10000	11	24	17.5
20000	20	36	28
30000	28	49	38.5
40000	36	59	47.5
0	1	15	8

TABLE 3

RESULTS OF TEST 2
VERTICAL LOAD DOWNWARD ON RAIL HEAD
15" FROM CENTER OF RAIL FASTENER

Load (Lb.)	Deflection (0.001 in.) Dial 1 (upward)	Dial 2 (upward)	Dial 3 (downward)	Dial 4 (upward)	Dial 5 (toward load)	Dial 6 (away from rail)	Dial 7 (upward)
0	0	0	0	0	0	0	0
400	4	0	4	0	1	0	0
1000	8	0	8	0	1	0	0
2000	20	0	20	2	2	1	0
4000	67	8	54	3	2	3	0
6000	144	23	104	7	2	9	1

2nd LOAD APPLICATION

0	0	0	0	0	0	0	0
220	1	0	1	0	0	0	0
1000	12	0	11	0	0	0	0
2000	30	2	26	1	0	0	0
3000	65	8	49	2	0	2	0
4000	102	15	71	5	0	4	0
5000	147	25	96	7	0	7	0
0	4	0	5	0	0	0	0

TABLE 4

RESULTS OF TEST 3
LATERAL LOAD ON RAIL HEAD
AT CENTER OF FASTENER

Load (Lb.)	Dial 1 (away from rail)	Dial 2 (away from rail)	Dial 3 (away from rail)	Dial 4 (away from rail)	Dial 5 (upward)	Dial 6 (downward)	Dial 7 (upward)	Dial 8 (downward)	Dial 9 (upward)	Dial 10 (toward rail)	Dial 11 (away from rail)	Dial 12 (toward rail)	Average c Dials 1 & 2 (See Fig.
	DEFLECTION (0.001 in.)												
0	0	0	0	0	0	0	0	0	0	0	0	0	0
400	9	6	0	0	0	4	0	0	0	2	0	0	7.5
1000	20	17	2	0	0	9	6	3	0	3	1	0	18.5
2000	49	45	5	0	6	18	21	7	1	3	1	1	47
4000	125	117	9	5	34	28	57	14	2	1	3	1	121

2nd LOAD APPLICATION

Load (Lb.)	Dial 1	Dial 2	Dial 3	Dial 4	Dial 5	Dial 6	Dial 7	Dial 8	Dial 9	Dial 10	Dial 11	Dial 12	Average c Dials 1 & 2
0	0	0	0	0	0	0	0	0	0	0	0	0	0
340	6	5	2	0	2	2	1	2	0	0	0	1	5.5
1000	17	15	2	0	6	4	5	6	0	2	0	2	16
2000	36	34	3	1	15	9	12	14	0	4	0	3	35
4000	94	91	7	4	43	20	44	24	0	4	2	3	92.5
6000	165	164	12	12	93	25	88	34	2	4	4	3	164.5
8000	237	244	19	15	162	25	134	50	4	12	9	10	240.5

TABLE 5

RESULTS OF TEST 4
LATERAL LOAD ON RAIL BASE
AT CENTER OF RAIL FASTENER

DEFLECTION (0.001 in.)

Load (Lb.)	Dial 1 (away from rail)	Dial 2 (away from rail)	Dial 3 (away from rail)	Dial 4 (away from rail)	Dial 5 (upward)	Dial 6 (upward)	Dial 7 (away from rail)	Dial 8 (away from rail)	Dial 9 (away from rail)	Aver. of Dials 1 & 2	Aver. of Dials 3 & 4
0	0	0	0	0	0	0	0	0	0	0	0
770	7	0	2	1	2	1	0	0	0	3.5	1.5
1000	12	0	7	1	4	1	0	1	0	6	4
2000	20	1	11	2	7	1	0	1	0	10.5	6.5
4000	31	6	17	5	10	3	1	3	1	18.5	11
6000	38	15	22	11	13	7	3	4	3	26.5	16.5
2nd LOAD APPLICATION											
0	0	0	0	0	0	0	0	0	0	0	0
1020	7	8	3	5	1	1	0	0	0	7.5	4
2000	10	11	6	6	2	2	0	0	0	10.5	6
4000	19	19	13	11	6	5	1	1	1	19	12
6000	28	26	18	16	9	7	2	1	2	27	17
8000	33	36	21	22	11	10	3	2	3	34.5	21.5
10000	37	47	24	29	13	14	5	4	4	42	26.5
12000	25	89	15	53	13	29	14	9	16	57	34
14000	24	110	15	65	13	39	19	13	23	67	40
0	5	21	6	14	0	6	8	1	7	13	10

TABLE 6

RESULTS OF TEST 5
LATERAL LOAD ON RAIL BASE
15 IN. FROM CENTER OF RAIL FASTENER

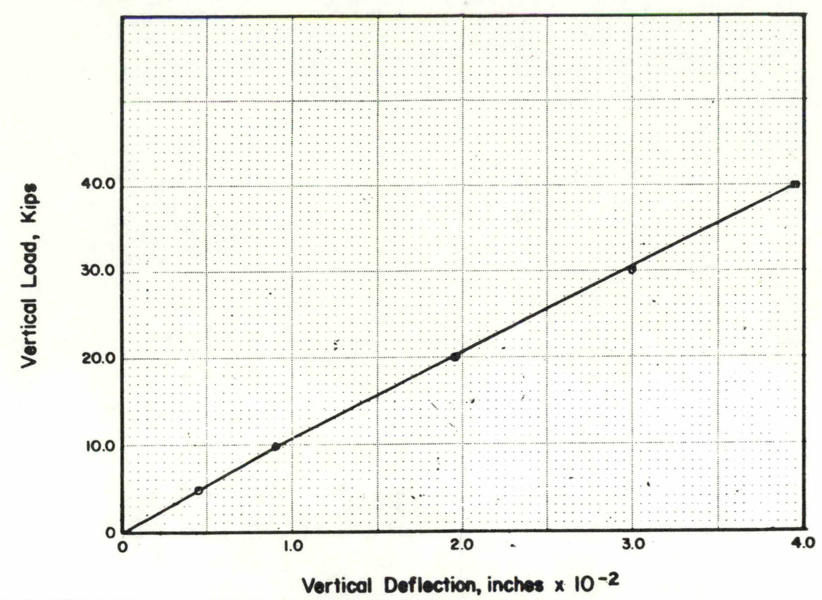
DEFLECTION (0.001 in.)

Load (Lb.)	Dial 1 (away from rail)	Dial 2 (away from rail)	Dial 3	Dial 4 (toward rail)	Dial 5 (toward rail)	Dial 6	Dial 7 (toward rail)	Average of Dials 1 & 2
0	0	0	0	0	0	0	0	0
290	12	13	0	0	0	0	1	12.5
400	17	18	0	0	1	0	1	17.5
600	24	25	0	0	1	0	1	24.5
800	28	29	0	1	1	0	0	28.5
1000	32	33	0	1	0	0	0	32.5
1500	45	44	0	1	0	0	0	44.5
2000	56	55	0	2	0	0	0	55.5
2500	71	70	0	2	0	0	0	70.5
0	7	11	0	0	0	0	0	9

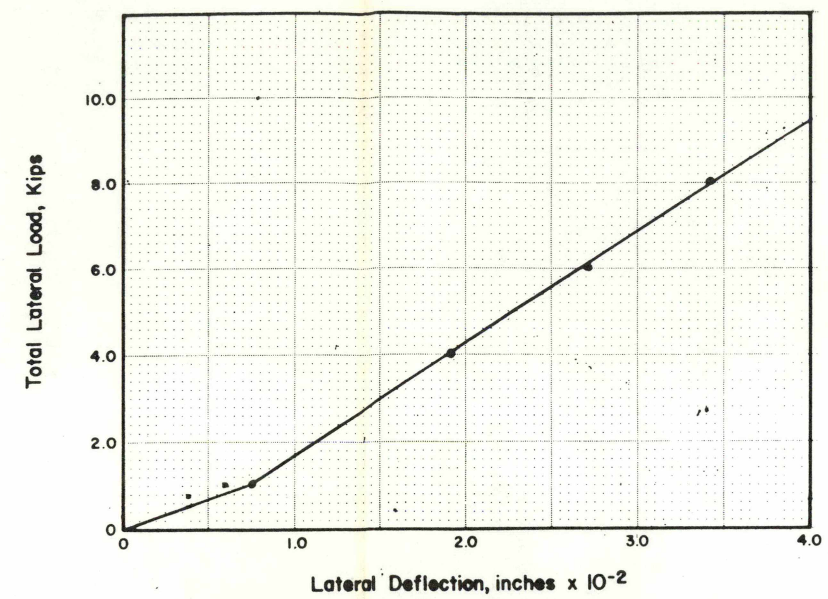
TABLE 7

RESULTS OF TEST 6
LONGITUDINAL LOAD ON RAIL BASE

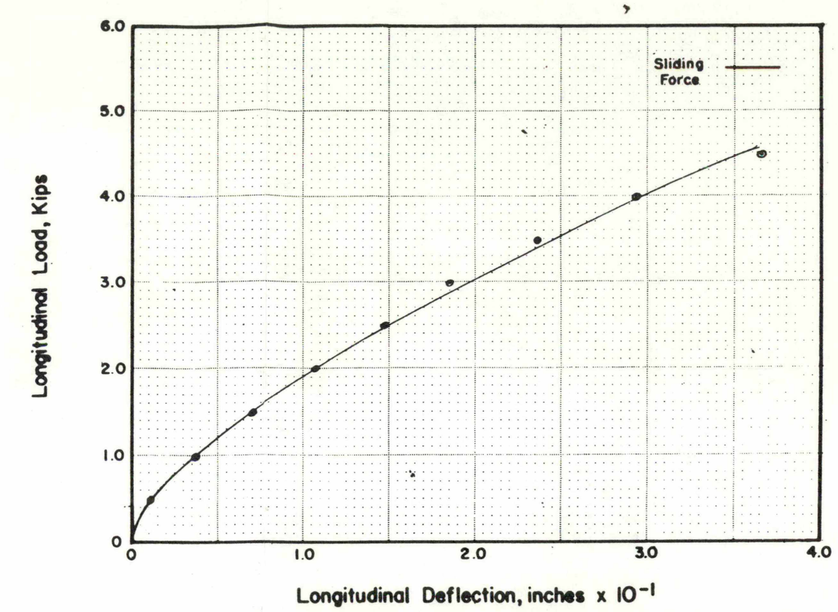
Load (lb.)	Dial 1 (away from load)	Dial 2 (away from load)	Dial 3 (away from load)	Dial 4	Dial 5	Dial 6 (upward)	Dial 7 (downward)
	Deflection (0.001 in.)						
0	0	0	0	0	0	0	0
500	11	0	0	0	0	0	0
1000	38	0	0	0	0	0	1
1500	71	0	0	0	0	2	3
2000	107	0	1	0	0	5	5
2500	147	0	1	0	0	7	7
3000	185	0	1	0	0	7	9
3500	236	0	1	0	0	7	11
4000	293	0	1	0	0	7	14
4500	365	1	1	0	0	8	17
5000	445	1	1	0	0	9	18
5500	Sliding						
0	182	0	0	0	0	6	18



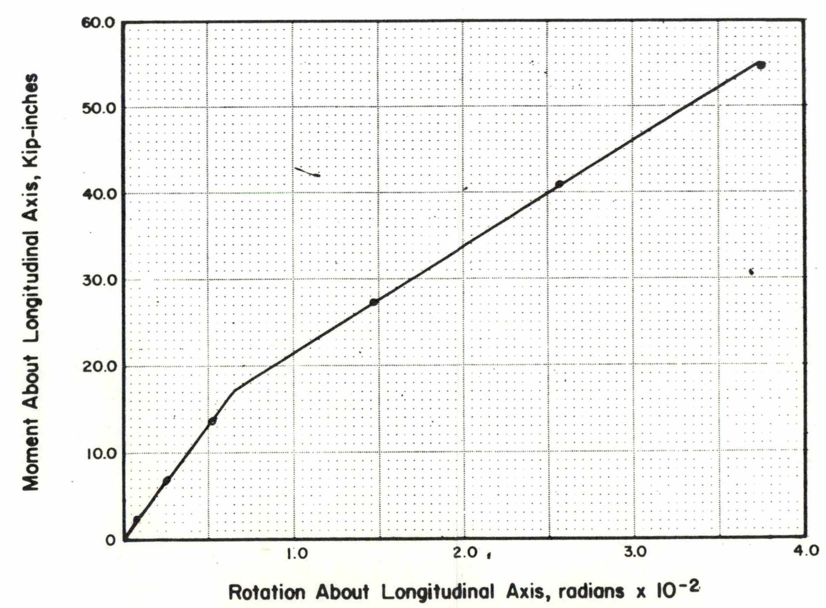
VERTICAL LOAD APPLIED TO RAIL HEAD
AT CENTER OF FASTENER



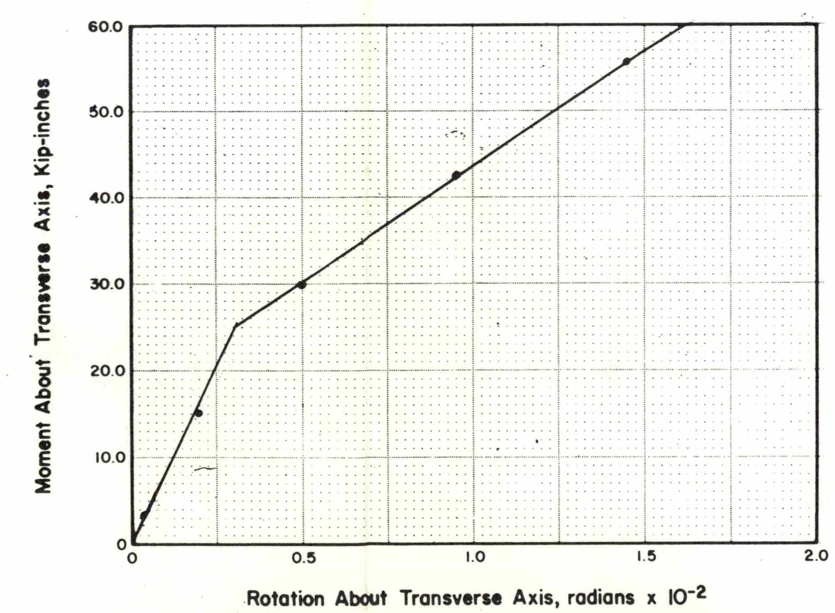
LATERAL LOAD ($\frac{P}{2}$) APPLIED TO RAIL BASE
AT A SHORT DISTANCE FROM FASTENER ENDS



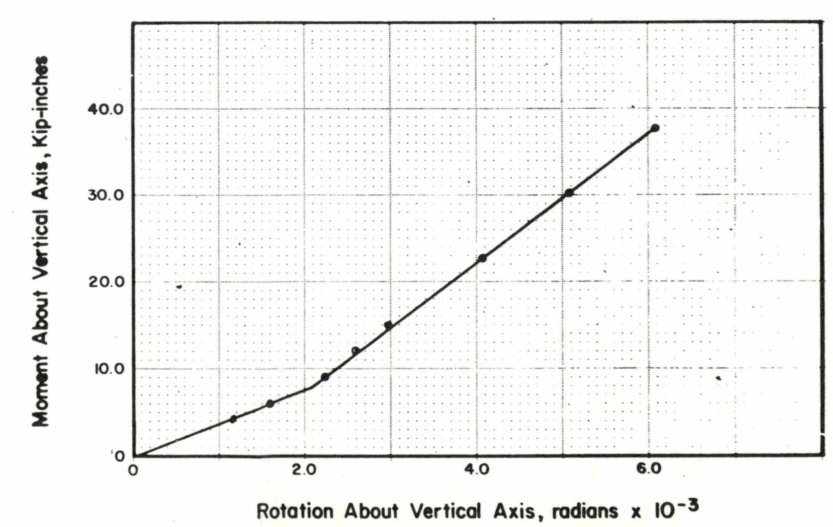
LONGITUDINAL LOAD APPLIED TO RAIL
AT CENTER OF RAIL BASE



LATERAL LOAD APPLIED TO RAIL HEAD
AT CENTER OF FASTENER



VERTICAL LOAD APPLIED TO RAIL HEAD
15 INCHES FROM FASTENER CENTER



LATERAL LOAD APPLIED TO RAIL BASE
15 INCHES FROM FASTENER CENTER

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
EASTERN LINES, TOPEKA KANSAS

U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

NON-CONVENTIONAL TRACK STRUCTURES
FASTEX FASTALL FASTENER
LOAD-DEFLECTION RELATIONS FROM
STATIC TESTS, NOV. 2-5, 1971

WESTENHOFF AND NOVICK, INC. CONSULTING ENGINEERS	W 1169	JUNE 1972

FIGURE B.1

APPENDIX C

Physical Properties of Slag Ballast

APPENDIX C

BALLAST PROPERTIES

The ballast used on this project becomes an integral part of the cast-in-place reinforced concrete sections and the transition support slabs. Resistance to lateral movement of the structures and restraint of longitudinal deflections caused by temperature will be supplied by shear across the structure-ballast interface. In addition, any chemical activity in the newly cast concrete initated by the ballast, or any tendency for the ballast to expand, could be detrimental to the structures.

Since the ballast will serve as a drainage blanket and a load bearing material, it needs to be properly graded to allow a free flow of water and to retard soil-ballast intrusion under loads. Durability and hardness criteria are also related to load carrying performance. Several laboratory tests and evaluations are performed to allow a determination of the suitability of the Santa Fe crushed slab ballast for use on sections 4, 5, and 7.

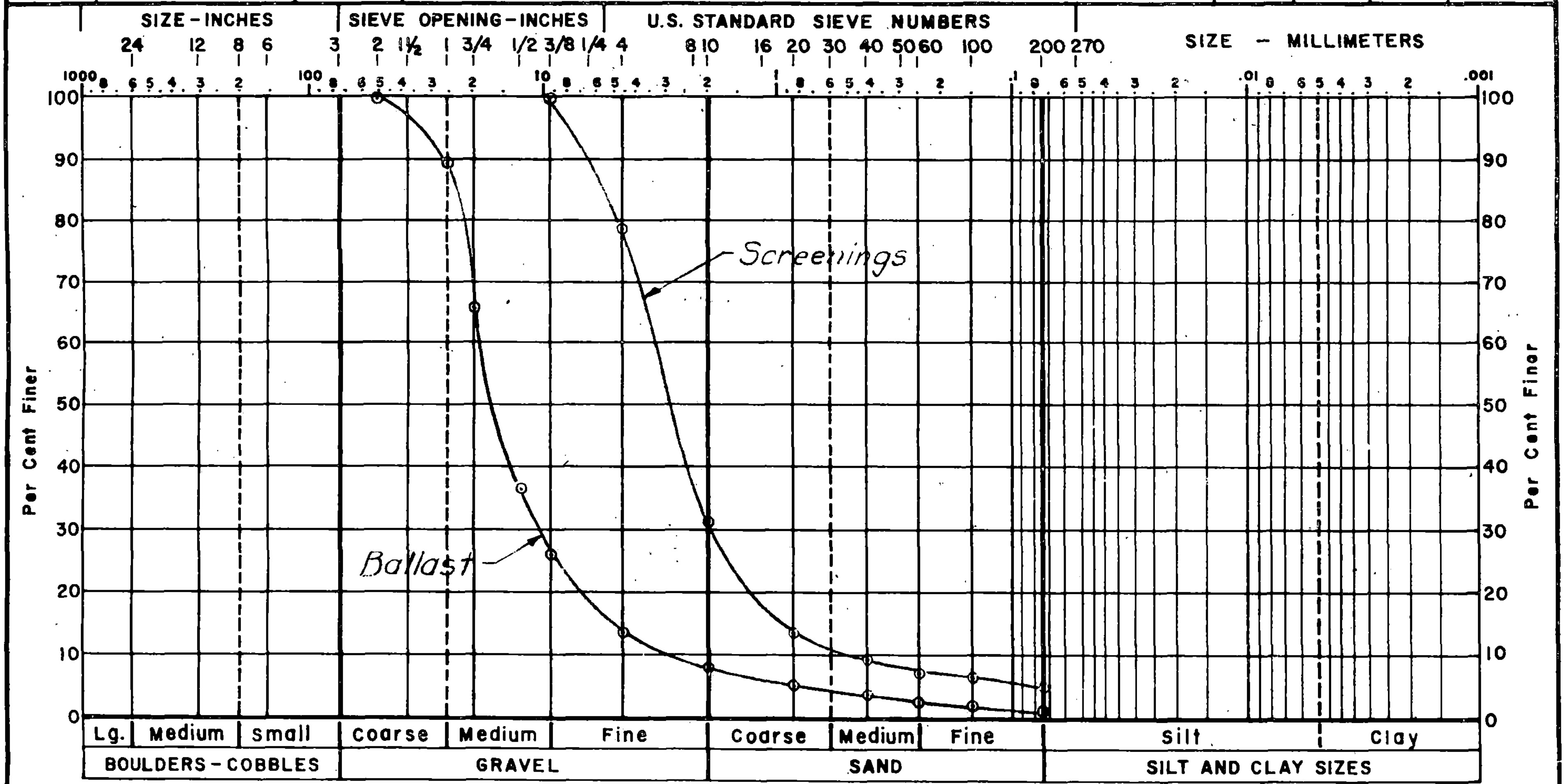
Sieve analyses were performed on two samples of ballast and one sample of screenings. Results of these determinations are shown on Figures C.1 and C.2. Gradation of the ballast is acceptable under AREA and AASHO specifications for sub-base material.

Results of several other evaluation tests are included in the enclosed report from R. W. Hunt Company. The ballast is acceptable under AREA specification for Specific Gravity, Hardness, Abrasion, etc. In addition, it was determined to be essentially inactive, and non-expansive.

Some question remained regarding the durability of the ballast under repeated load. Since no definite criteria were available, a series of one point Proctor Tests were performed with Sieve Analyses made prior to, and after compaction. Little breakdown in particle gradation was noted; the comparative plot is shown on Figure C.2.

The apparent satisfactory performance of the ballast in service on the existing track and the successful compliance with AREA and AASHO Specifications, with no definite, negative test responses to other tests, led to the decision to utilize the crushed slag ballast for the total test track, thereby eliminating another possible variable from consideration.

No.	Sample	Depth	Description	LL	PL	PI	pH
			Crushed PUEBLO SLAG Ballast				7.6
			Crushed PUEBLO SLAG Screenings				7.6



GRAIN SIZE DISTRIBUTION

Job No. 1169 Project A.T.& S.F. Rwy. Test Track

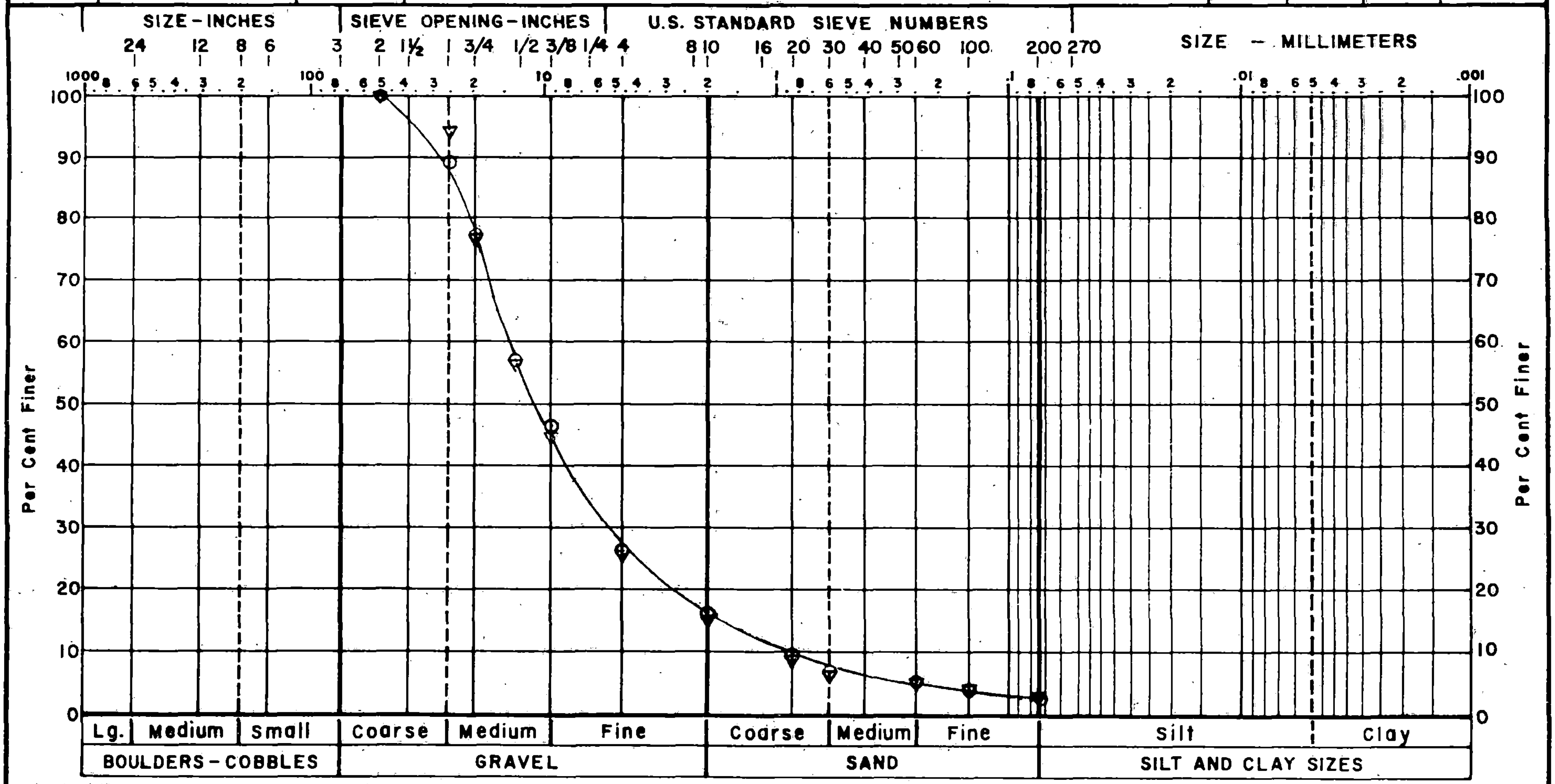
Client A.T.& S.F. Rwy. - D.O.T./C.H.S.G.T. - F.R.A.

WESTENHOFF AND NOVICK, INC. Consulting Engineers

Chicago, Illinois

By J.M. Date June 71 Figure C.1

No.	Sample	Depth	Description	LL	PL	PI	
○	A		Crushed PUEBLO SLAG as received				
▽	B,C,D,E		Crushed PUEBLO SLAG after one point Modified Proctor				



GRAIN SIZE DISTRIBUTION

Job No. 1169 Project A.T.& S.F. Rwy. Test Track

Client A.T.& S.F. Rwy. - D.O.T./O.H.S.G.T. - F.R.A.

WESTENHOFF AND NOVICK, INC. Consulting Engineers
Chicago, Illinois

By G.K. Date Sept 71 Figure C.2

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

August 31, 1971

FILE NO. 3670-6 REPORT 3930
ORDER 13-C-5664 PAGE 1

Tests On Slag

Westenhoff and Novick, Inc.
222 West Adams Street
Chicago, Illinois 60606

Attention: Mr. F. Mc Lean

Gentlemen:

We report test results on a sample of crushed slag picked up by our representative at your office on July 28, 1971.

Sample Marked- E-W- 1169

TEST RESULTS:

Specific Gravity (A.S.T.M. C127-68)	2.47
Absorption (A.S.T.M. C 127-68)	1.80
Hardness: (MoH's Scale)	8 to 9

Abrasion By Use of Los Angeles Machine (A.S.T.M. C 131-69)

Grading	"A"
Weight of Sample before test	5000
Weight of Sample after test (retained on No. 12 Sieve)	3450
Loss in weight	1550
Percentage of wear	31.00

Relative Density: (A.S.T.M. D 2049-69)

Minimum dry density using 0.1 cu. ft. mold and scoop to fill mold	88.8
Maximum dry density, dry method using 0.1 cu. ft. mold	115.40
Maximum dry density, wet method using 0.1 cu. ft. mold	110.90

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

August 31, 1971

FILE NO. 3670-6
ORDER 13-C-5664

REPORT 3930

Potential Reactivity (Chemical Method)
(A.S.T.M. C 289-66)

Trial	Quantity R_c Reduction in Alkalinity Millimole Per Litter	Quantity S_c Concentrated SiO_2 Millimole Per Litter	Classification
1	910	1.9	Innocuous
2	915	2.2	Innocuous
3	915	1.7	Innocuous

Soundness: (Sodium Sulfate 5-Cycles)

Sieve Size Passing	Sieve Size Retained	Grading of Original Sample Per Cent	Weight of Test Fractions Before Test Grams	Passing Finer Sieve After Test (Actual Per Cent Loss)	Weighted Average (Corrected Per Cent Loss)
1 1/2"	3/4"	37.1	1502	2.26	0.84
3/4"	3/8"	45.4	1002	2.20	1.00
3/8"	No. 4	17.5	300	2.33	0.41
Totals		100.0			2.25

Sieve Size Passing	Sieve Size Retained	Pieces in Test Fractions Before Test	Appearance of Material After Test: Pieces Affected – Split	Pieces Affected – Chipped or Flaked	Pieces In Good Condition
1 1/2"	3/4"	58	1	3	54

August 31, 1971

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Expansion Test:

Procedure:

A portion of the material aggregate was cut from the sample by use of a riffle box. This portion was reduced to a grading size of 100 % passing 1/2" sieve and 100% retained on a No. 4 sieve.

A six inch layer of the prepared sample was put into a tight container (4" diameter and 10" deep) on top of which was placed a surcharge equivalent to 18" of concrete. (21.0 lbs. approximate)

The distance from the top of the aggregate layer to the top of the container edge was measured prior to applying the surcharge.

The container, with aggregate and surcharge were autoclaved at 300 p.s.i. for two hours- after which the steam pressure was gradually reduced and the assembly allowed to cool over night.

The surcharge was removed and the distance between top of aggregate and container measured and recorded.

In the case of the slag submitted- no change was recorded indicating that the slag may not be expansive.

Respectfully submitted,
ROBERT W. HUNT COMPANY

G.E. Matoush, Manager
Cement and Concrete Department

GEM:rek-4

APPENDIX D

Field Tests of Ballast - Concrete Interface Shear Characteristics

APPENDIX D

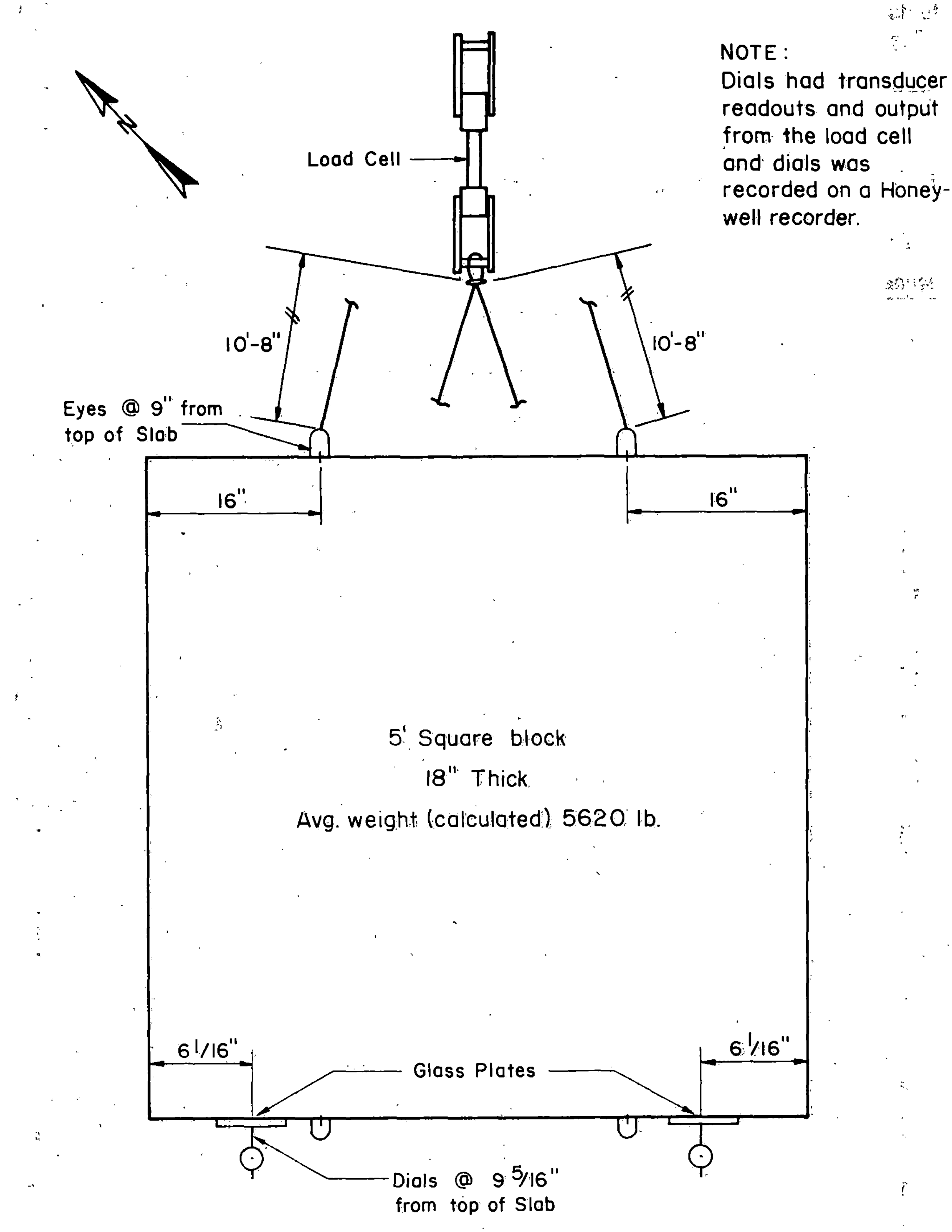
Ballast - Slab Interface

Shear Tests

Little information is available in the literature related to the shear strength or granular interlock properties or particles the size of ballast. Specifically, no information of any kind was available which would have allowed a rational, quantitative assessment of the ballast-concrete slab interface response under shear loadings. Since this is so important to a realistic analysis of the structures under lateral load conditions, Westenhoff and Novick recommended the performance of laboratory or field tests to determine value for use in analysis and design.

It was decided that the most economical and realistic determination of properties would result from large scale field tests. Westenhoff and Novick planned and observed the test series, which were performed by the grading contractor under the direction of the Technical Research and Development Department of the Santa Fe Railway, who supplied dead loads, instrumentation and recording equipment.

The tests were performed using two 18 inch thick by 5 feet square slabs cast on approximately 6 inches of compacted ballast. The slabs had embedded lifting stirrups, lying in the horizontal mid-plane, on two sides as shown on Figure D.1. A horizontal pulling force was exerted on the slab using a truck mounted crane. The load cell, clevis, and associated rigging are shown conceptually on Figure D.1.



KANSAS TEST TRACK | SANTA FE / DOT

TYPICAL SLAB ARRANGEMENT FOR BALLAST SHEAR TESTS

WESTENHOFF AND NOVICK, INC. CONSULTING ENGINEERS	W 1169	JUNE 1972

FIGURE D.1

Instrumentation and recording devices provided by the Santa Fe consisted of a load cell, a frame which mounted two dial indicators, that contacted two glass plates glued on the slab, and two electric transducers which were connected to the dial indicators. The dial indicators were used for calibration of the recorded deflection traces and visual control of the test sequence. Load cell and transducer signals were conditioned and then recorded on a Honeywell multiple track recorder, yielding a trace on light sensitive recording paper.

The size of the slabs was selected to give a fairly large surface area without requiring excessive dead load to obtain the contract pressures desired. An upper limit contact pressure of 10 psi was selected for the same reason. The lower pressure of 5 psi was chosen to yield a significant variation, without being small enough to allow other unknown variables to influence the tests.

Essentially five types of tests were performed. First, the cast-in-place slabs were pulled, at each contact pressure, to obtain a ballast-concrete shear relation for the cast-in-place structures. Second, the slabs were reseated with the rough faces in contact with the ballast, again at two contact pressures, to obtain relations for well bedded pre-cast units. Third, the slabs were inverted such that the finished face contacted the ballast to obtain a relationship for poorly bedded pre-cast units. Fourth, a series of pulls was made, at varying rates of load application, to isolate inertial effects. Fifth, ballast shoulders were placed against the slab, in the direction of pull, to determine the significance of ballast shoulders.

The results of these tests is contained in Tables D.1 and D.2. It was determined that: 1) pulls were made at a slow enough rate to exclude inertial effects; 2) ballast shoulders do supply resistance to sliding, but the results were not considered quantitatively conclusive; 3) the precast units and transition surfaces which would contact ballast would need to be artifically roughened and well bedded to mobilize sufficient shear resistance, 4) the value obtained for undisturbed cast-in-place units is not as high as normally assumed in the literature for highway design, but is higher than shown in the soil mechanics literature for friction on the interface. The value used for structural modeling is developed in the text, and is 175 $lb/in^2/in$.

TABLE D.1
SUMMARY OF FIELD
TESTS OF
BALLAST-SLAB SHEAR+
(5 ft. square x 18 inch deep slabs)

Test No.	Condition of Interface	Normal Force, lb	Maximum Pull Force, lb	Sliding Pull Force, lb	Maximum tan Ø	Sliding tan Ø	Remarks
1	As cast	17,800	23,500	11,500	1.32	0.65	Crane moved, restarted-slab rotated vertically about leading edge
2	Smooth	17,800	10,700	9,200	0.60	0.52	——
3	Smooth	34,800	12,600	11,550	0.36	0.33	Raining hard; attempted slow strain rate (avg. 0.124 in/sec)
4	Smooth	34,800	13,400	13,000	0.39	0.37	Raining; attempted moderate strain rate (avg. 0.147 in/sec)
5	Smooth	34,800	12,300	11,650	0.36	0.34	Raining; attempted high strain rate (avg. 0.235 in/sec)
6	As cast	34,800	>30,000	—	>1.0	—	Raining; load cell damaged, slab position slightly disturbed
7	As cast	34,800	(34,900)	(29,500)	(1.0)	(0.85)	Crane moved; these values are not valid for interface shear; slab position disturbed

+Kansas Test Track

TABLE D.1
(cont.)

Test No.	Condition of Interface	Normal Force, lb	Maximum Pull Force, lb	Sliding Pull Force, lb	Maximum tan Ø	Sliding tan Ø	Remarks
8	As cast	34,800	44,500	43,600*	1.28	1.26*	Slab moved about 4 inches
9	Rough	34,800	27,500	23,680	0.79	0.68	Slab moved about 14 inches
10	Rough (reseated)	17,800	14,300	12,400	0.80	0.74	Slab lifted and reseated on ballast; pulled 14 inches
11	Smooth	17,800	15,500	—	0.87	—	A tamped 1 ft wide ballast shoulder on 1:2 slope on pull side; pulled 6 inches
12	Smooth	17,800	14,400	—	0.81	—	A tamped 2 ft shoulder on 1:2 slope, pulled 6 inches
13	Smooth	17,800	12,000	—	0.67	—	A tamped 1 ft shoulder on each side of slab, parallel to pull; pulled 6 inches

+Tests 1 through 6 performed October 28 & 29, 1971. Tests 7 through 13 performed November 9, 1971.
( )These values are not valid measurements of interface action.
*Pull not long enough to get true measure of sliding force.

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TABLE D.2

BALLAST-SLAB

FRICTION VALUES

Condition	Peak μ	Sliding μ	Subgrade psi	Remarks
As cast	1.30	0.68	$F=\mu N$	Ballast friction angle
Rough	0.79	0.68	9.67	Heavy load only; sliding same as as cast
Smooth	0.60	0.50	4.94	*Concrete-ballast friction angle
Smooth	0.80+	—	4.94	With ballast shoulder against pull
Smooth	0.37	0.35	9.67	*Concrete-ballast friction angle

*Concrete-ballast friction angle does not follow $F=\mu N$ relationship. Approximate shearing stress for 4.94 psi normal pressure is 3.0 psi, while shearing stress for 9.67 psi normal pressure is 3.5 psi. The relation is probably non-linear. Thus, two points will not establish a general relationship but will serve to bound the working range of subgrade pressures between 5 psi and 10 psi. In addition, the general range is probably narrow.

+This value is not a true "friction factor" due to contribution of ballast shoulder.

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APPENDIX E

Field Plate Bearing tests

APPENDIX E

Field Plate Bearing Tests

As a portion of the design effort involved with the reinforced concrete track structures, plate bearing tests were performed on the three appropriate sections of embankment to establish values of vertical subgrade modulii for use in the computer analyses. These tests were performed as outlined in EM 1110-45-303, and were carried out between October 26, 1971 and October 29, 1971. The tests were positioned near the main instrument arrays at locations selected by reference to construction test data supplied by Shannon and Wilson (see Appendix F).

A 30 inch diameter bearing plate was used in conjunction with 24 inch and 18 inch plates to form the plate stack. Load was applied by means of a hydraulic jack reacting against a beam which was suspended under a flat-bed semi-trailer. The trailer was loaded with cast concrete blocks weighing about 2400 lbs each. Sand was used as a bedding medium to establish a uniform seating of the bearing plate on the ballast.

One test was performed on top of the in-place ballast in Sections 4, 5, and 7. One additional test was carried out on the lime stabilized layer in Section 5 to ascertain the influence of the ballast layer on the subgrade response to the plate loads. Also, the tests on Section 5 were performed using 5 psi increments instead of 10 psi increments, and two reload curves were obtained for the test made on the ballast layer in an effort to obtain some indication of performance under repeated loads. The results of these tests are shown on Figure E.1 while Table E.1 summarizes the subgrade modulus data calculated from the tests.

TABLE E.1

SUMMARY OF SUBGRADE MODULII

FROM 30" PLATE BEARING TESTS

STATION	BEARING SURFACE	UNADJUSTED MODULUS K'_v, lb/in^2/in	MODULUS ADJUSTED FOR PLATE BENDING K_v, lb/in^2/in	REMARKS
8551+25	Ballast	250	225	*Standard Procedure
8559+81	Ballast	227	210	Standard Procedure
		178	170	Full Curve
		500	390	Reload Curve #1
		556	430	Reload Curve #2
8559+87	Limed Clay	172	170	Standard Procedure
		152	150	Full Curve
8577+00	Ballast	333	280	Standard Procedure

*Standard Procedure as described in Corps of Engineers Manual EM 1110-45-303, ENGINEERING AND DESIGN: RIGID AIRFIELD PAVEMENTS

Kansas Test Track
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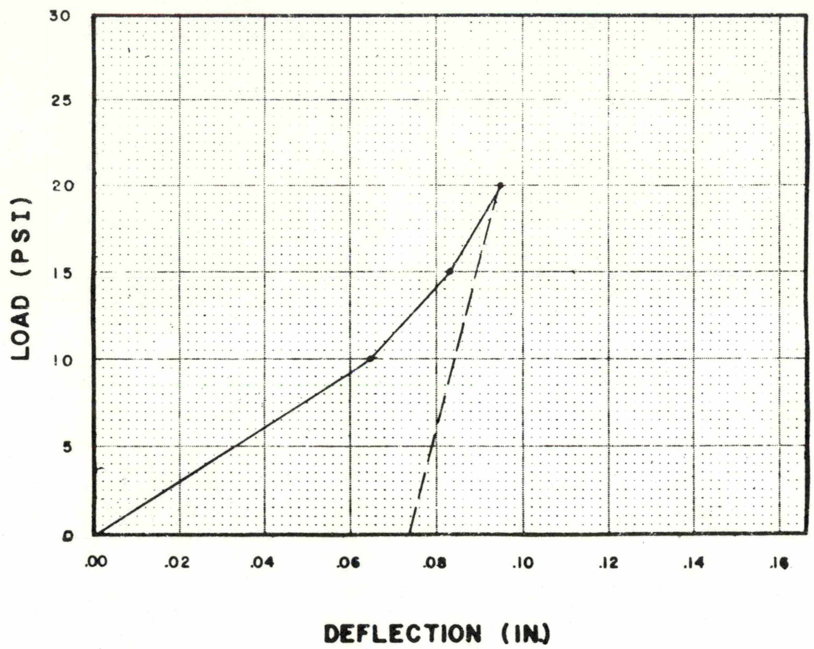
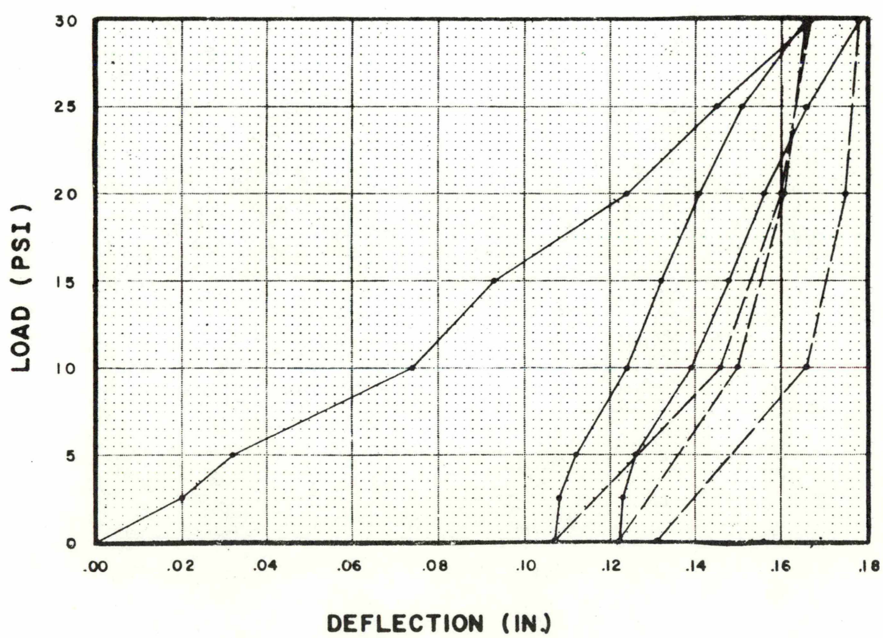


STATION 8577 + 00



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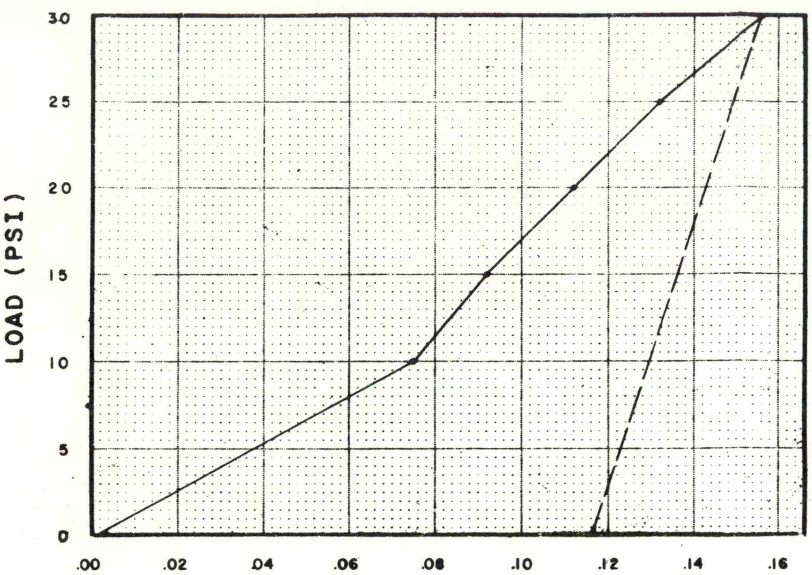
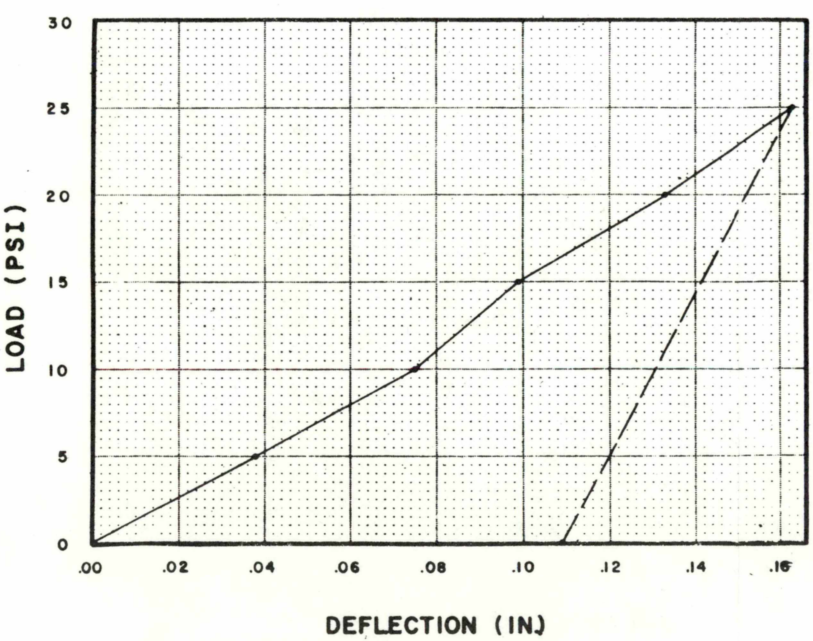


STATION 8551 + 75



STATION 8559 + 87

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY
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U.S. DEPARTMENT OF TRANSPORTATION
KANSAS TEST TRACK

NON - CONVENTIONAL TRACK STRUCTURES

RESULTS OF PLATE BEARING TESTS

WESTENHOFF AND NOVICK, INC. CONSULTING ENGINEERS	W1169	JUNE 1972

FIGURE E.1

APPENDIX F

Statistical Analysis of Construction Test Data

APPENDIX F

Construction Test Data

Due to reasons beyond the control of the Santa Fe, the Construction Report for the embankment was not available at a time necessary to select the location of the plate bearing tests. Westenhoff and Novick was then authorized to perform simple statistical analyses of data supplied by Shannon and Wilson. These data are summarized in Figure F.1 and Tables F.1 through F.3.

TABLE F.1

RESULTS OF STATISTICAL ANALYSIS

OF CONSTRUCTION TEST DATA* FOR

SECTIONS 4, 5 AND 7

Material	Item	Sample	Range	Average	Standard Deviation	Variance
Limed Clay	Dry density (pcf)	23	82.2-93.7	89.2	3.1	9.8
	Water content (%)	23	26.6-30.8	28.6	1.2	1.5
Clay	Unconfined Compressive Strength (tsf)	22	1.2-7.6	3.2	1.4	2.0
	Dry density (pcf)	147	73.9-111.9	98.1	9.6	92.3
	Water content (%)	165	16.6-33.3	23.7	2.7	7.5

*Includes data from: N-, T-, and nuclear density tests; laboratory unconfined tests; and plate bearing tests

TABLE F.2

SUMMARY OF SUBGRADE MODULII

FROM CONSTRUCTION TEST DATA

(Limed Layer)

Station	Initial Tangent Modulus ($lb/in^2/in$)	Reload Modulus ($lb/in^2/in$)	Reload Modulus / Initial Tangent Modulus
8543+43	876.77	1702.04	1.941
8553+53	1076.31	1939.53	1.802
8558+88	610.76	1939.53	3.176
8568+27	1081.53	2690.32	2.488
8575+09	641.97	2382.86	3.712
AVERAGE	857.47	2130.86	2.624
VARIANCE	51,443.10	158,400.11	0.662
STANDARD DEVIATION	226.81	398.00	0.814
RANGE	MIN. 610.76 MAX. 1081.53	MIN. 1702.04 MAX. 2690.32	1.802 3.712

Kansas Test Track
Santa Fe/DOT

TABLE F.3

SUMMARY OF SUBGRADE MODULII

(Except Limed Layer)

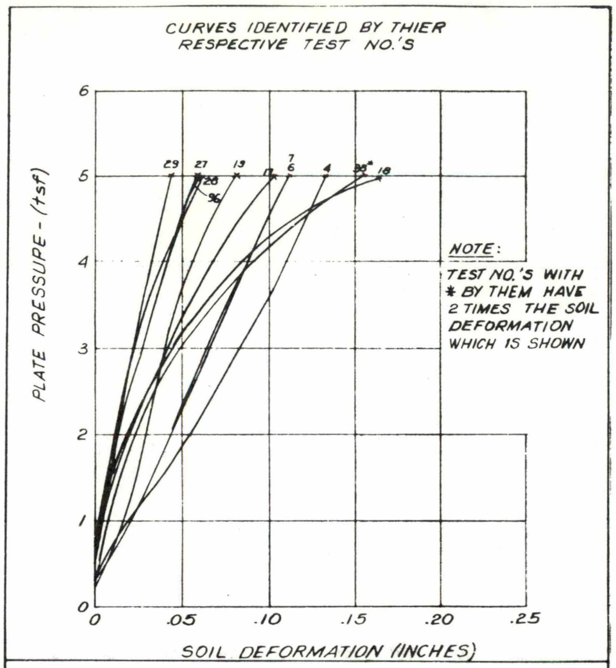
Station	Initial Tangent Modulus ($lb/in^2/in$)	Reload Modulus ($lb/in^2/in$)	Reload Modulus / Initial Tangent Modulus
8547+01	1893.88	1939.53	1.024
8547+54	1949.56	2316.67	1.188
8547+80	726.88	1191.43	1.639
8550+06	785.35	1004.82	1.279
8550+36	626.05	1104.64	1.764
8551+34	534.40	916.48	1.715
8551+94	1481.79	1619.42	1.093
8551+99	586.66	926.67	1.580
8553+53	1262.58	1544.44	1.223
8554+85	2051.72	2885.81	1.407
8557+60	974.76	969.77	0.995
8557+93	368.35	370.67	1.006
8558+88	593.07	1437.93	2.425
(2nd cycle)		829.85	1.399
8559+30	383.24	538.06	1.404
8559+35	573.25	842.42	1.470
8560+48	534.05	758.18	1.420
8561+56	847.17	1273.28	1.503
8569+95	437.85	1544.44	3.527
8570+11	1000.43	937.08	0.937
8570+37	550.17	432.12	0.785

Table F.3
(cont.)

Station	Initial Tangent Modulus ($lb/in^2/in$)	Reload Modulus ($lb/in^2/in$)	Reload Modulus / Initial Tangent Modulus
8570+42	1813.78	2452.94	1.352
8572+82	2046.39	2059.26	1.006
(2nd cycle)		3336.00	1.630
8572+91	562.85	1774.47	3.153
8574+45	826.84	1476.11	1.785
8575+09	3212.44	4508.11	1.403
8576+00	516.93	1813.04	3.507
8576+02	962.31	1413.56	1.469
8577+09	1481.74	2138.46	1.443
8578.66	1668.00	2254.05	1.351
AVERAGE	1077.67	1568.06	1.577
VARIANCE	473,350.76	801,514.06	0.469
STANDARD DEVIATION	688.00	895.27	0.685
NUMBER	29	31	31
RANGE MIN.	368.35	370.67	.785
MAX.	3212.44	4508.11	3.527

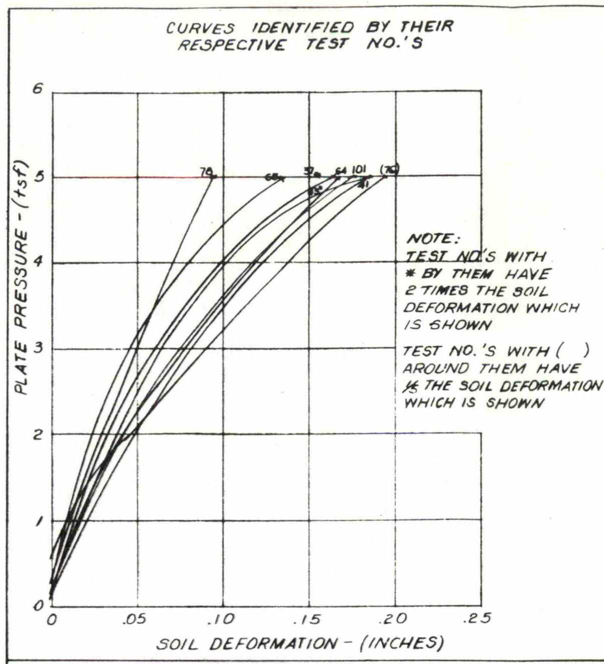
Kansas Test Track
Santa Fe/DOT

SECTION 7

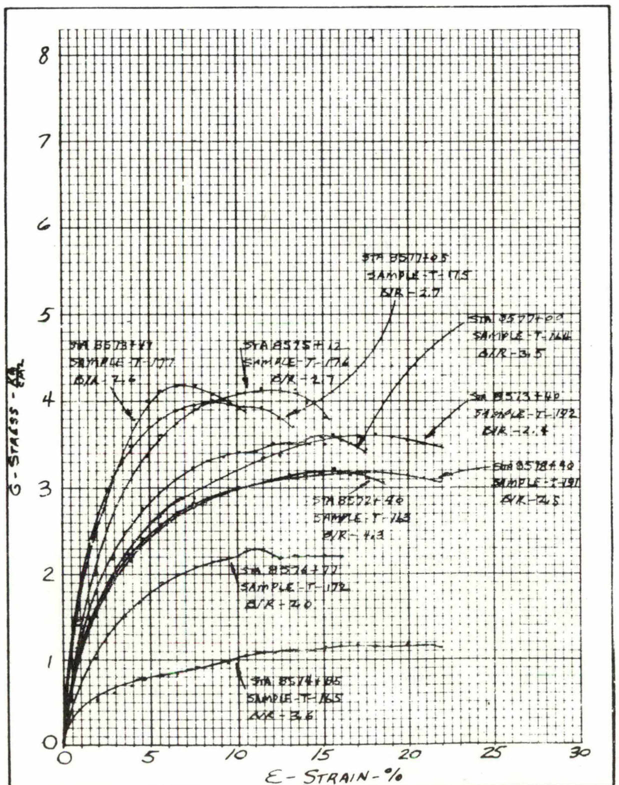


SUMMARY OF PLATE BEARING TEST PLOTS

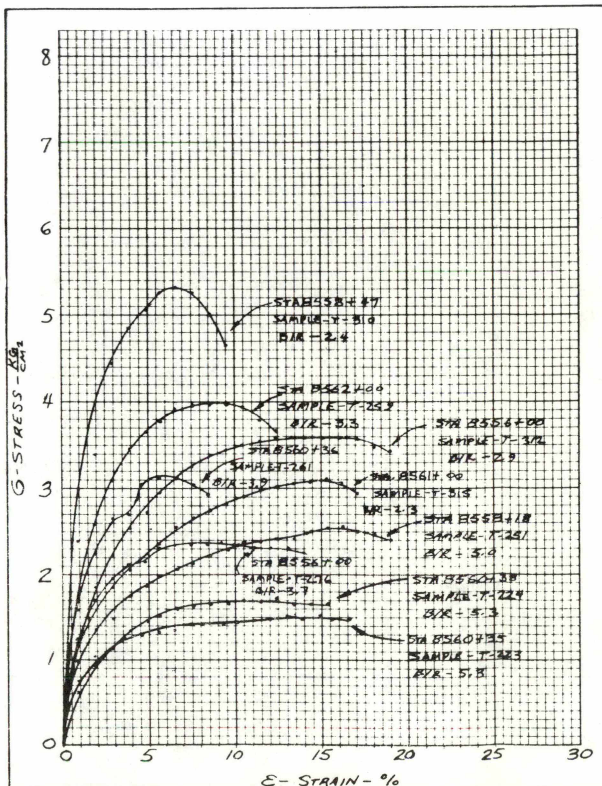
SECTION 5



SUMMARY OF PLATE BEARING TEST PLOTS

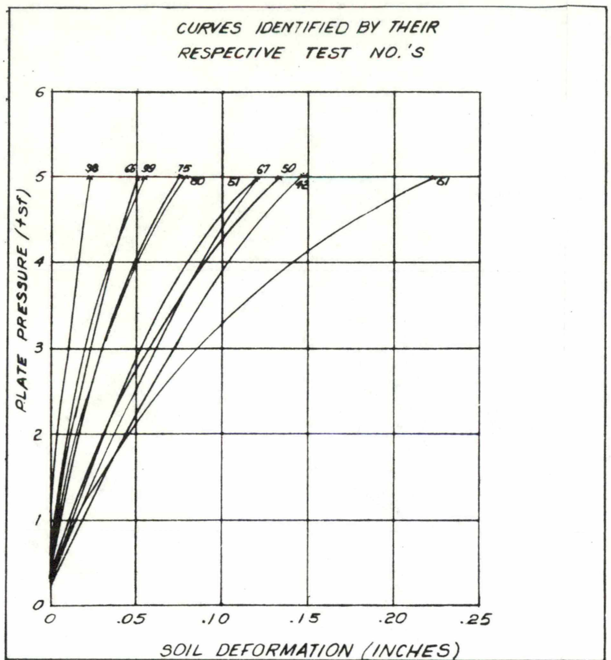


SUMMARY OF UNCONFINED COMPRESSIVE TESTS

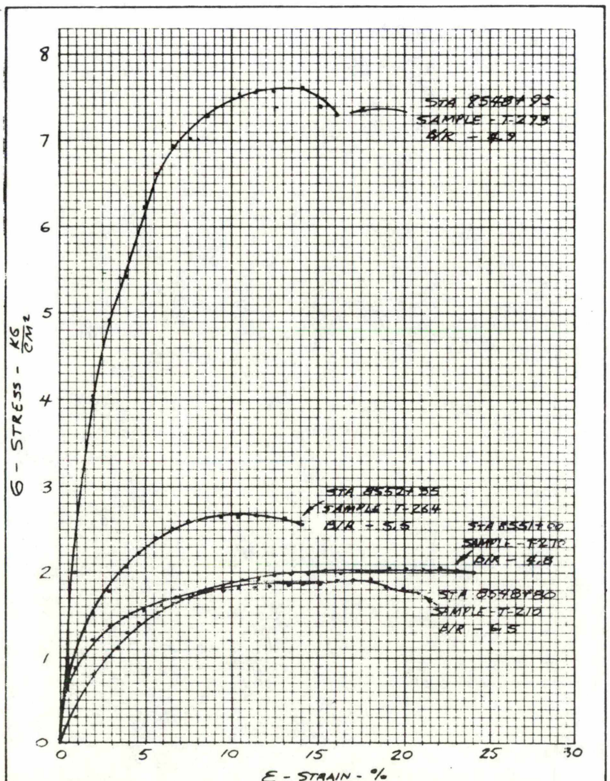


SUMMARY OF UNCONFINED COMPRESSIVE TESTS

SECTION
4



SUMMARY OF PLATE BEARING TEST PLOTS



SUMMARY OF UNCONFINED COMPRESSIVE TESTS

THE ATCHISON, TOPEKA AND SANTA FE RAILWAY COMPANY EASTERN LINES TOPEKA KANSAS		
U.S. DEPARTMENT OF TRANSPORTATION KANSAS TEST TRACK		
NON - CONVENTIONAL TRACK STRUCTURES **PLOTS OF CONSTRUCTION TEST DATA**		
WESTENHOFF AND NOVICK, INC. **CONSULTING ENGINEERS**	W 1169	JUNE 1972

FIGURE F.1

APPENDIX G

Selected Bibliography

SELECTED BIBLIOGRAPHY

American Concrete Institute, Torsion of Structural Concrete, Publication SP-18, 1968

American Concrete Institute, Building Code Requirements for Reinforced Concrete, Publication 318-71

American Railway Engineering Association, Manual for Railway Engineering

American Railway Engineering Association, Various Reports, Proceedings 1930, 1931, 1932, 1937

Barkan, D. D., Dynamics of Bases and Foundations, McGraw-Hill, 1962

Birmann, F., "German Federal Railway Experiments with Concrete Track Beds," The Railway Gazette, April 18, 1969

Birmann, F., "Schwellen - und bettungsloser Oberbau", ETR, August 1969

Bruggeling, A.S.G., "Limitation of Crack Formation in Long Reinforced Concrete Slabs", CEMENT, Vol XXXI, No. 2, 1969, translated, International Civil Engineering, April 1969

Cheung, Y. K. and O. C. Zienkiewicz, "Plates and Tanks on Elastic Foundations - An Application of the Finite Element Method," International Journal of Solids, Vol. 1, 1965

Clough, R. W. and J. L. Tocher, "Finite Element Stiffness Matrices for the Analysis of Plate Bending," Proc, Conference on Matrix Methods in Structural Mechanics, Wright Patterson AFB, 1965

Curro, J. R., Jr., "Vibroseismic Survey, Railroad Test Track Embankment," Report from U.S.C.E. Waterways Experiment Station to U.S. Department of Transportation, 1971

Desai, C. S. and J. F. Abel, Introduction to the Finite Element Method, Van Nostrand-Reinhold, 1972

Dietrich, R. J. and J. R. Salley, "Embankment Support for a Railroad Test Track, Design Studies," Report from Shannon and Wilson, Inc. to U. S. Department of Transportation, 1971

Dietrich, R. J. and J. R. Salley, "Embankment Support for a Railroad Test Track, Embankment Construction and Instrumentation, Volume I," Report from Shannon and Wilson, Inc. to U. S. Department of Transportation, 1971

SELECTED BIBLIOGRAPHY
(Continued)

Eisenmann, J., "Stress Distribution in the Permanent Way," AREA Bulletin #622, 1969

Engineering/Analysis Corporation, EAC/EASE (a user information manual), Control Data Corporation, April 1971

Friberg, B. F., "Frictional Resistance under Concrete Pavements and Restraint Stresses in Long Reinforced Slabs," Proceedings, Highway Research Board, Vol. 33, 1954

Hetenyi, M., Beams on Elastic Foundation, University of Michigan Press, 1946

Lampert, P., "Torsion and Bending in Reinforced and Prestressed Concrete Members," Proc, Institution of Civil Engineers, Vol. 68, No. 12

Lyse, I. and B. G. Johnson, "Structural Beams in Torsion," Transactions, ASCE, Vol. 101, 1936

Matsubara, K., "Rail Tracks for Japan's 130-mph Trains," Civil Engineering, December 1969

Meachem, H. C., et al, "Study of New Track Structure Designs, Phases I and II," Research Reports from Battelle Memorial Institute to U. S. Department of Transportation, 1968 and 1969

Meachem, H. C. et al, "Studies for Rail Vehicle Track Structures," Research Report from Battelle Memorial Institute to U. S. Department of Transportation, 1970

Murayama, H., "Manpower Saving in Track Maintenance - Further Development of Concrete Slab Tracks," Japanese Railway Engineering, 4th Quarter, 1971

NASA, The NASTRAN Theoretical Manual, NASA SP-221, COSMIC, 1970

NASA, The NASTRAN User's Manual, NASA SP-222, COSMIC, 1970

NASA, The NASTRAN Programmer's Manual, NASA SP-223, COSMIC, 1970

Persson, B.O.E., and B.F. Friberg, "Concrete Pavements With Continuous Reinforcement and Elastic Joints," HRR No. 291, Highway Research Board, 1969

Portland Cement Association, Concrete Supported Railway Track, April 1941

SELECTED BIBLIOGRAPHY (Continued)

Richart, F. E. et al, _Vibrations of Soils and Foundations_, Prentice - Hall, 1970

Special Committee No. 2: Railway Track and Structures, "Nonconventional Track Structures," _Railway Roadmasters_ - _Bridge and Building Association_, September 1971

Timoshenko, S. P., _Strength of Materials_, Parts I and II, Van Nostrand, 1956

Timoshenko, S. P., "Method of Analysis of Statical and Dynamical Stresses in Rail," _The Collected Papers of Stephen P. Timoshenko_, McGraw-Hill, 1953

Timoshenko, S. P., and J. N. Goodier, _Theory of Elasticity_, McGraw-Hill, 1970

Timoshenko, S. P., and S. Woinowsky - Krieger, _Theory of Plates and Shells_, McGraw-Hill, 1959

Tokahara, K., "JNR Experiments with Concrete Track Bed," _The Railway Gazette_, April 4, 1969

U. S. Corps of Engineers, "Engineering and Design: Rigid Airfield Pavements," EM 1110-45-303

U. S. Corps of Engineers, "Engineering and Design: Foundation Subject to Vibratory Loads," EM 1110-45-310

United States Steel Corporation, _Steel Design Manual_, 1968

Walters, J. M., "Research into Track Bed Conditions," _Rail Engineering International_, June 1971

Zienkiewicz, O. C., _The Finite Element Method in Engineering Science_, McGraw-Hill, 1971

Zienkiewicz, O. C. and Y. K. Cheung, _The Finite Element Method in Structural and Continum Mechanics_, McGraw-Hill, 1967

The Kansas Test Track Non-Conventional Track Structures: Design Report, 1972
FRA, Office of High Speed Ground Transportation