University of Alberta
Department of Civil Engineering

Structural Engineering Report No. 90

Analysis and Design of Stub-Girders

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SUMMARY

This report presents the results of a major study on the behavior and strength of stub-girders. Developed by J.P. Colaco, the stub-girder floor system allows the integration of a composite (steel-concrete) girder and mechanical and electrical service systems that are needed in a building structure but with a minimum of structural disruption. This normally leads to savings in construction cost and time, through lower material weights, less expensive fabrication procedures, and shorter construction time.

The investigation was composed of a theoretical study and an experimental evaluation of the performance of the stub-girder system. The theoretical study focused on various methods of stub-girder analysis, utilizing different Vierendeel modeling schemes for the complex girder, as well as a non-prismatic beam analysis. The results were compared with the data obtained in the experimental phase of the project.

The physical testing was done in two stages: A series of stub-slab-chord assemblies were tested, utilizing different welding and stiffener details, with a view to improving the performance and economy of a crucial component of the girder. Based on the results of these tests, a full-size girder was designed, using the welding and stiffener layout indicated by the best-performing stub specimens. The full-size girder was tested in flexure, monitoring load-deflection behavior, characteristics of slab, chord, and stubs, as well as the ductility and stiffness at the various levels of load.

The comparison of the theoretical and experimental results indicated that the traditional Vierendeel approach to stub-girder modeling gives satisfactory predictions of internal stress resultants, but slightly lower deflections than as tested, in the service load range. Behavior in this range was essentially elastic, and the girder exhibited high stiffness and ductility. The differences may be reduced by further improvements in the modeling scheme.

Deflection to span ratio at service load was 1/670, and at ultimate load it was 1/95. The girder thus exhibited significant stiffness as well as ductility, without giving rise to local or overall premature failures.

The welding and stiffening details that were used for the full-size girder were substantially different from those that have been used in actual structures in the past. Thus, only partial perimeter welds were used between the stubs and the bottom chord of the girder, producing savings in the amount of welding of more than fifty percent. Similarly, partial end-plate stiffeners were used instead of fitted stiffeners for the exterior stubs, and no stiffeners were used for the interior stubs. Both of these choices represented significant departures from current practice, but did not produce undesirable behavior or strength characteristics.

It was found that transverse reinforcement of the concrete slab is an important factor for the overall behavior of the girder. Thus, minimum reinforcement in the form of a mere shrinkage mesh may not be sufficient. Similarly, the amount of logitudinal reinforcement is important to the overall ductility of the stub-girder, particularly for loads in excess of the service load.

The stub specimens failed in a combination mode of concrete slab shear and compression. The failure plane extended from the corners of the rib of the metal deck over the top of the stud shear connectors. Little or no distress was visible in the connectors. In the full-size girder, the ultimate failure was the same type of shear and compression mode that was observed in the stub specimens. However, because of the higher degree of redundancy, and thus ability of the stub-girder to redistribute the internal forces, a number of shear connectors on the exterior stubs also failed in a combination of shear and tension. There was ample warning of the impending failure, including extensive yielding of the exterior stubs. No web buckling or other local failure occurred in any of the stubs.

The final section of the report presents recommendations for analysis and design details. In addition, some topics that are in need of further study are briefly described.

ACKNOWLEDGEMENTS

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Extensive analyses were carried out by Alfred F.H. Wong during the initial phase of the project, and the authors are indebted to him for this work. Messrs. E.Y.L. Chien, M.I. Gilmor and J.K. Ritchie, all of the CSICC staff, provided invaluable input to the research work, especially during the early planning stages.

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TABLE OF CONTENTS

Page

LIST OF FIGURES

FIGURES		PAGE

FIGURE		PAGE
33	Load-Deformation Measurements for Stub Specimen I (No stiffeners)	131
34	Load-Deformation Measurements for Stub Specimen II (Full end-plate stiffeners at both ends of stub)	132
35	Load-Deformation Measurements for Stub Specimen III (Partial end-plate stiffeners at both ends of stub)	133
36	Load-Deformation Measurements for Stub Specimen IV (Fitted stiffener only in compression zone of stub)	134
37	Load-Deformation Measurements for Stub Specimen V (Fitted stiffeners at both ends of stub)	135
38	Cracking Pattern In Slab of Stub Specimen I	136
39	Cracking Pattern in Slab of Stub Specimen II	136
40	Cracking Pattern in Slab of Stub Specimen III	137
41	Cracking Pattern in Slab of Stub Specimen IV	137
42	Cracking Pattern in Slab of Stub Specimen V	138
43	Location of Typical Shear Failure Plane in Slab of Stub Specimens	139
44	Shear Failure Plane in One of the Test Specimens	140
45	Buckling of Steel Deck in a Stub Specimen	140
46	Shear Failure Plane at End of Slab in Stub Specimen III (Partial end-plate stiffeners)	141
47	Signs of Local Yielding, Evidenced by Whitewash Flaking in the Web of Stub Specimen IV (One fitted web stiffener)	141

FIGURE PAGE

CHAPTER 1
INTRODUCTION

1.1 The Stub-Girder System

The stub-girder floor system was developed as a direct result of the need for new, innovative designs, aimed at providing more economical structural steel buildings. The systems originator, J.P. Colaco, realized the monetary savings that could be achieved if the mechanical and electrical service ducts could be incorporated into the volume occupied by the floor framing system.[1,2,4] With this in mind, the composite stub-girder system subsequently came into being. Additional savings are realized from the system's ability to capitalize on the economic advantages of continuous, composite floor beams and the resultant savings afforded by a decrease in erection time.[3]

Over the last ten years the system has been used for a number of buildings in the United States and Mexico, ranging in height from three to fifty-six stories. Stub-girders are being used for the first time in Canada in the thirty-seven story new headquarters building for the Nova Corporation (formerly Alberta Gas Trunk Line Co.), currently under construction in Calgary, Alberta.

The stub-girder system consists of a built-up girder used as the principal framing member and standard wide-flange shapes used as transverse (secondary) members. The main girder can be visualized as a Vierendeel truss, utilizing a wide-flange column shape for its bottom chord and a concrete slab for its top chord. The vertical truss members which connect the top and bottom chords are provided by welding pieces of wide-flange beams, called stubs, to the top flange of the bottom chord. The axis of the stubs and the bottom chord are oriented in the same direction, with the stub centered on the chord flange. In cross section, the web of the stub lies in line with and directly above the web of the bottom chord as shown in Figs. 1 and 2.

The connection between the stubs and the top chord (concrete slab on a corrugated steel deck) is made by using stud shear connectors welded to the top flange of the stubs and embedded in the concrete slab. Fitted web stiffeners have traditionally been provided at both ends of each stub, and in some cases an additional stiffener has been placed at the middle of the stub. The continuous, transverse floor beams act compositely with the steel deck and concrete cover slab, and carry the floor loads to the stub-girder. Normally, the depth of the stubs are chosen the same as the depth of the floor beams in order that the steel deck can remain level across the stub-girder. Figures 3, 4 and 5 show completed stub-girder floors before they are concealed by the suspended ceilings.

1.2 Previous Studies

Following the introduction of the stub-girder system to industry, a number of articles appeared in the trade press.[3,4,5,6,7,8] Some studies have dealt with the analysis and design of the stub-girder.[9,10] The original article by J.P. Colaco[2] documented the results of a full scale test which was conducted at the test facilities of the Granco Steel Products Company in St. Louis, Missouri. In the following, this test will be referred to as the "Granco Test."

Two other full-scale, proprietary tests have also been reported.

H.H. Robertson Company in Ambridge, Pennsylvania, conducted one test for Ellisor and Tanner, Inc., Dallas, Texas[12], and the other also was done for Ellisor and Tanner, but performed by Inryco Research and Development Company in Milwaukee, Wisconsin.[13] During the preparation of this thesis, a full scale test was performed at the University of Alberta on a stub-girder specimen, typical of those being fabricated for use in the Nova Corporation building in Calgary. The test was performed for Nova Corporation and Dominion Bridge Company, Ltd., of Calgary.

As the previous tests were proprietary experiments, this study is believed to represent the first detailed and systematic research work dealing with the system to full scale. However, additional studies have now been commenced by at least two other organizations.[17-18]

CHAPTER 2
SCOPE OF INVESTIGATION

2.1 General Rationale

The economic benefits of using the stub-girder system has been documented in the construction of actual buildings. The main disadvantage of using the system is represented by the amount of time that must be spent on additional shop fabrication. Details such as stud shear connectors, web stiffeners and stub-to-chord welding significantly affect the economy of any job. Field connection details and slab reinforcement requirements also must be considered.

The investigation that is described in the following was undertaken for three basic reasons. Firstly, the system is relatively new and very little organized and intensive research has dealt with it. The fact that the system has performed very well in a variety of existing buildings inspires confidence, but does not necessarily mean that the most economical solutions have been found. As limit states design becomes the governing design philosophy, knowledge must be obtained about the behaviour of the system at loads up to and including ultimate. This can only be obtained in controlled laboratory tests.

Secondly, the accuracy of the internal stress resultants by the analytical models can be determined. Data obtained from strain gauge instrumentation can be converted to forces while the girder remains in its elastic response range.

Finally, the monetary savings that can be gained from simplifying and reducing the fabrication and construction details are very important. In the end, the success of any new development is inseparably tied to construction economy.

2.2 Scope of Study

Having ascertained the need for an investigation of this type, the topics that were deemed the most in need of detailed research can be itemized as follows:

1. Method of stub-girder modeling for analysis.
2. Design and method of stub stiffening.
3. Design of welded connection between stub and bottom chord.
4. Design of stud shear connectors.
5. Analysis of slab behaviour and strength.
6. Localized stress and strain effects, including "hard" points at end of stubs.
7. Cracking behaviour of slab.
8. Design of concrete slab reinforcement.

To ensure that the testing programme would cover all of these topics as fairly as possible, while observing the time and monetary constraints of the project, it was decided to run two different types of tests. Due to the cost and complexity of stub-girder experiments, preliminary testing was deemed necessary in order to investigate some of the crucial fabrication details that might influence the overall girder strength and behaviour. A variety of methods of stub-stiffening can be used, and the most efficient and economical required identification. This enabled the project to use only one full-size stub-girder, while deriving many of the benefits that otherwise only could have been gained by testing several.

The stub to bottom chord welding detail would also be studied in the preliminary tests. In traditional designs and in previous tests, the stubs were welded to the chord flange, using fillet welds around the entire stub perimeter. As will be seen in the following, the design of the stub-girder for this project specified only partial perimeter welding. This was based in part on the results of the preliminary tests, and in part on accurate analytical data.

It was therefore decided to fabricate a number of single stub-slab assemblies, each using the partial welds but different methods of web stiffening. These assemblies are referred to in the following as the stub specimems.

The study was divided into four major portions, as outlined below:

1. Analysis and design of stub specimens and full-size girder.
2. Fabrication and testing of stub specimens.
3. Fabrication and testing of full-size girder.
4. Analysis of test results and comparison with the analysis and design data.

This program was considered adequate for dealing with a significant portion of the problem areas. It is noted that certain topics were eliminated from the study, primarily because each of them would require complete research investigations of their own. For example, the dynamic characteristics of the stub-girder floor system are typically determined on the basis of the method that is given in Appendix G of the Canadian Standard for limit states design of steel structures.(15) Since the approach presented there was developed on the basis of simply supported solid slab and wide-flange shape composite beams, its applicability to the stub-girder system is not clear. Similarly, the method of shear stud design has been questioned, particularly in view of the possibility that the failure mode of closely spaced studs cannot be deduced from that of a single stud. Finally, the Vierendeel analysis and subsequent design make use of criteria that were developed for more "typical" members. The state-of-the-art of composite truss analysis has not been sufficiently advanced to allow a rational application to Vierendeel type members. Each of these problems should be studied in detail, but are of such a magnitude as to warrant separate investigations.

CHAPTER 3

STRUCTURAL ANALYSIS OF THE STUB-GIRDER

3.1 General Concepts

It was decided at an early stage to confine the analysis of the stub-girder system to two-dimensional approaches. Although a three-dimensional solution would be more accurate, such would make the analysis quite complex and not necessarily that much better. In an actual structure, the transverse floor beams would also be compositely designed, which would subject the concrete slab to a biaxial state of stress. However, as with other composite beam tests, this type of out-of-plane bending is difficult to simulate and is commonly not accounted for. Research in this area has been very limited. This simplification of analysis and testing gives a conservative representation of the behaviour of the system in an actual building, insofar as deflections and girder stresses and strains are concerned. With no adjacent girders or other parts of a real structure, the test girder could be expected to deform more, and thus subject its individual components to more severe conditions.

A two-dimensional finite element solution has been suggested by some researchers(9), but is not yet considered to be a convenient, design-office oriented technique. For this reason, it was decided to model the girder as a Vierendeel truss. This approach is the one that was used by Colaco in his original solution(2). A University of Alberta computer programme, entitled "Plane Frame and Truss Analysis" (PFT), was used to analyze a variety of different truss configurations that were used to model the stub-girder.

3.2 Preliminary Design

With input from the Project Analysis Division of the Canadian Steel Industries Construction Council, the basic member sizes for the stub-girder components were chosen. Sizes were picked on the basis of a preliminary design, a survey of existing building projects using this system, and a knowledge of the general availability of the materials needed. The following material and member sizes were selected for the stub-girder, a drawing of which is shown in Fig. 1:

Main girder (bottom chord of stub-girder)	: W310 x 86 (Imperial designation W12 x 58)
Transverse floor beams	: W410 x 39 (Imperial designation W16 x 26)
Stubs	: W410 x 39 (Imperial designation W16 x 26)
Steel deck	: 76 mm Westeel-Rosco T-30V - Wide rib profile - 1.22 mm thickness
Concrete slab	: 27.5 MPa semi-lightweight - 160 mm full thickness - 84 mm above top of steel deck ribs
Structural Steel grade	: CSA G40.21-M, Grade 300W
Slab reinforcement steel grade	: Grade 400

3.3 Vierendeel Analysis

3.3.1 General Criteria

Having chosen the basic member sizes for the test girder, a detailed analysis was carried out using the type of Vierendeel model which was first proposed by Colaco[2]. Figures 6 (a) and (b) show the model and the real girder which it represents. Because of symmetry, only one-half of the girder needed to be considered, with the boundary conditions adjusted appropriately. This model was used to determine internal forces for the detailed design. Following the full-size stub-girder test, strain gauge data were analyzed and compared with the predictions provided by the original model. Several other modeling schemes were also evaluated in an attempt to determine the best discretization pattern for the girder. These are described in detail later in this chapter.

3.3.2 Traditional Modeling of Stubs

The connections between all members in a Vierendeel truss are considered as rigid. Each stub, in this approach, therefore, was chosen to be represented by five vertical members, each with an area and moment of inertia equal to one-fifth that of the whole stub in the vertical direction. The number of members (five) used to represent each stub was an arbitrary choice, which sought only to divide the stub into a reasonable, finite number of segments.

One vertical member was placed at the location of each transverse beam. This member was pin-ended in the model, to reflect the low bending stiffness of the beam web about its longitudinal axis. The stiffness of the bottom chord was set equal to that of the W310 x 86, except for that section of the chord where the stubs were attached. Here the stiffness was increased, since the stub would tend to stiffen the bottom chord.

The section properties of the top chord (concrete slab and steel deck) were determined by transforming the slab into an equivalent steel member, using a modular ratio of 10. The effective width of the concrete slab was determined using the requirements given by the Canadian structural steel design standard(15). This appears to be an appropriate method for computing the effective width for this type of built-up, composite member. However, additional research is warranted, in view of the fact that the design rules were developed on the basis of the behaviour of quite different types of composite beams, using elastic methods of analysis. It is not known now whether this approach is equally acceptable under ultimate load conditions. The calculations that were done to determine the section properties of all members are given in Appendix 1.

Boundary conditions were assumed as shown in Figs. 6 (b) through (e). At the location of the connection between the bottom chord and the column at the left end, a roller support was placed. The actual connection was a typical double angle beam-to-column type. At the right end of the model, which is at midspan of the girder, supports which allowed only vertical displacements were used. Zero rotation was required here because of the girder symmetry.

Since the slab between the exterior stub and the end of the stub-girder was considered ineffective as a truss member, no member was placed there in the model. In the full-size test, the slab was not fixed to the column at either end and therefore could not transfer any load. However, it did rest on a channel section, which simulated the spandrel of a real structure, insofar as slab support was concerned.

In an actual structure, the slab would normally encase the column, leading to the possibility of tension cracks in this portion of the slab due to the unintentional horizontal restraint provided by the column. Since these cracks would reduce the shear capacity of the slab in the end panel, the conservative assumption was made that the bottom chord would carry the entire support reaction. Hence, this portion of the slab was omitted in the analysis. This simplification does not affect the equilibrium of the rest of the girder.

As in the actual test, concentrated loads were applied at the floor beam locations, as indicated in Fig. 6. The arbitrary designation of a unit load at the one-quarter span and one-half of a unit load at mid-span, allowed all member forces to be determined in terms of an arbitrary load, P.

The bending moments, axial forces and shear forces for certain members are shown in Fig. 7. A more complete listing of member forces is given by the output from the PFT analysis, contained in Appendix 1. (It should be noted that the PFT output is expressed in Imperial units, as the programme was not equipped to handle S.I. units).

3.3.3 Variations of Vierendeel Modeling

Two other models, derived from the first Vierendeel model, were also used to analyze the stub-girder. These are illustrated in Figs. 6 (b) and (c), and differ only in the way the stubs are modeled.

The first variation used three vertical members to represent a stub instead of five members. Their areas and moments of inertia were altered accordingly (i.e. the area of a member was one-third that of a stub web, instead of one-fifth,etc.). The second variation also utilized a three member stub, (Fig. 6 (d)) but instead of the center member being vertical, it was oriented diagonally, from the top corner of the stub at one end to the bottom corner at the other end. The orientation of the member was such that it would act in tension when the girder was subjected to load. Free rotation was permitted at both ends of this member so that it carried only axial tension and no moment. The area of this member was arbitrarily set equal to that of the two vertical members in the stub. Each of the two vertical members (one at each end) was given an area and moment of inertia equal to one-half that of the stub in the vertical direction. Appendix 2 contains the output for these two analyses.

3.4 Non-Prismatic Beam Analysis

In order to give another estimate of the deflected shape of the stub-girder, a non-prismatic beam model was also constructed and analyzed. In this case, the entire length of the beam was considered, rather than just one-half the span.

The girder was divided into nine segments, consisting of two alternating types of cross sections. The first cross section represented the concrete slab and steel deck, acting together with the bottom chord. The second cross section represented the slab, steel deck, stub and bottom chord. For each section, the slab properties were transformed into equivalent steel units and then combined with the section properties of the steel member(s) using the parallel axis theorem, to produce a single, equivalent cross section. The beam was simply supported and was loaded at the quarter points with three equal point loads, as indicated in Fig. 6 (e). The section property calculations are detailed in Appendix 2 and the associated values are given in Fig. 6 (e). The deflected shape of the non-prismatic beam is plotted in Fig. 57, along with the deflected shapes of the other three models. Chapter 7 compares the deflected shapes and analyzes the findings with regard to the modeling schemes.

CHAPTER 4

DESIGN OF THE STUB-GIRDER

4.1 Introductory Comments

There are no codified recommendations or guidelines that apply specifically to the design of the stub-girder. Therefore, by the current design procedures, the forces in each individual component of the stub-girder are determined through the use of an analytical model, and existing code requirements are then used to design these members. It must be pointed out that none of these requirements were developed specifically for unusual structural members, such as the stub-girder. Some uncertainty, therefore, still exists as to the suitability of all of the design requirements that have been used, but it is intended that many of these questions will be answered here.

Section 3.2 of Chapter 3 details the main member sizes which were chosen on the basis of the preliminary design. The member forces, shown in Fig. 7, were determined from the model shown in Fig. 6 (b). It remained to design the structural details and to determine the design ultimate load as accurately as possible. Appendix 3 contains these design data in their entirety.

4.2 Design of the Bottom Chord

The design assumed that the capacity of the stub-girder member would be governed by the capacity of its principal flexural component, the bottom chord. The other components of the stub-girder would then be designed to resist the forces imposed on them at the already determined ultimate load.

The design ultimate capacity of the stub-girder was based on the axial tension and bending requirements of CSA Standard S16.1-M78[15]. Several critical sections in the bottom chord thus had to be analyzed. Three potential critical sections were identified from data obtained in the detailed structural analysis. Their locations can be discerned from Fig. 7, as follows:

1. Section 1 - mid-span
2. Section 2 - exterior end of interior stub
3. Section 3 - exterior end of exterior stub

Each section was investigated on the assumption that no local failures in the stub-girder would prevent the bottom chord from reaching its ultimate capacity. This was assured through the choice of the W310 x 86 for the chord, which satisfies the plastic design compactness criteria of S16.1-M78. The design computations are given in Appendix 3.

The ultimate values of each of the concentrated loads for the three critical locations were found as follows:

1. Girder failing in combined tension and bending at mid-span (tension predominant):

   P = 228 kN
   Total load: 3P = 684 kN

   Note that these values include the dead load of the girder itself, estimated as 40 kN.

2. Girder failing in combined bending and tension at the exterior end of the interior stub (both stress resultants of about equal importance):

   P = 242 kN
   Total load: 3P = 723 kN

3. Girder failing in pure bending at the exterior end of the exterior stub:

   P = 260 kN
   Total load: 3P = 730 kN

The governing section was found to be at mid-span, with a corresponding ultimate load per load point of: P = 228 kN

It should be noted that this load is the design ultimate load, and not a prediction of the failure load of the test specimen.

4.3 Stub Details

With the design ultimate load having been determined, actual values for the forces in each member of the stub-girder could be obtained from the analytical model. A free body diagram of each stub was drawn, in order that the appropriate design forces could be identified. The three details that required designing were:

1. Stud shear connectors
2. Stub to bottom chord welds
3. Web stiffeners and welds

Figure 8 shows a free body diagram of an exterior stub with the individual forces identified.

4.3.1 Design of Shear Connectors

The shear connectors were required to provide the shear transfer capacity at the interface of the slab and the stubs. This required consideration of two possible failures modes: a shear failure in the stubs and a failure in the concrete, allowing the studs to pull free from the slab.

A conservative approach was taken in which it was assumed that each stud provides either shear resistance or pull-out resistance, but not both. The shear resistance of the studs was calculated using the requirements of CSA S16.1-M78, Section 17.3.6 (a)[15]. The pull-out resistance was determined using a formula suggested by KSM Welding Systems Division[14]. Both resistances were then reduced to account for the overlapping of the concrete failure cones that develop when studs are closely spaced. Such a cone is shown in Fig. 10. This design approach appears to be sufficiently conservative. However, a detailed study of the resistance of single and multiple shear studs under combined loads is needed, particularly in view of the fact that the slab to stub shear connection may be critical to the overall behaviour of the girder. This is discussed in some detail in Chapter 7.

The shear connector design calculations, contained in Appendix 3, produced the following stud requirements:

1. Exterior stubs: 15 pairs 127 x 20 ϕ evenly distributed, along the stub length.
2. Interior stubs: 5 pairs 127 x 20 ϕ evenly distributed, along the stub length.

4.3.2 Stub To Bottom Chord Welds

A conservative approach was also taken with regard to the design of the welds. The free body diagram of the stub was again used to determine the forces that had to be resisted by the stub-to-chord welds.

The primary function of the weld is to provide shear transfer capability between the stub and the bottom chord. The length and size of the weld needed to resist the shear force were determined on the basis of the requirements of Section 13.13 of the CSA limit states design standard(15). For 300W grade steel and E480XX electrodes, the resistance of a fillet weld will be controlled by shear across the effective throat area of the weld metal. The resistance is expressed as (15):

$$V_r = 0.50\, \phi\, A_w\, X_u \qquad (1)$$

Where V_r = factored shear resistance.

ϕ = performance factor.

A_w = effective throat area of weld.

and X_u = ultimate strength as rated by the electrode classification number.

For the materials used in the stub girder, this expression gives a resistance of 1.22 kN/mm. The total shear forces to be transferred by the exterior and interior stubs are indicated in Figs. 8 and 9. Choosing an 8 mm fillet weld, the length of weld required were 1310 mm and 440 mm, respectively, which were placed along each side of the stub at either end. This is shown in Figs. 11 and 12. These figures also show the five vertical members that were used to model the stubs for the Vierendeel analysis, along with the shear forces that had to be carried by each, as determined by the PFT computer programme. The welds were distributed to the ends of the stubs according to the shear force distribution given by the analysis. As shown in the figures, the weld distribution for each end was chosen as follows:

Exterior stub:	54% at left end; 710 mm total, 355 mm each side
	46% at right end; 660 mm total, 330 mm each side
Interior stub:	63% at left end, 280 mm total, 140 mm each side
	37% at right end; 160 mm total, 80 mm each side

A length of weld was required across the end of each stub, to resist the tensile force that would be trying to separate the stub from the bottom chord. Equal in length to the width of the stub flange, a weld was therefore placed across the end of the stub to resist this force. If this length of weld were of insufficient capacity, additional lengths were placed on both sides of the stub flange.

It was assumed that because of the lower ductility of a fillet weld loaded in a direction perpendicular to its longitudinal axis,[16] the entire length of weld could not reach yield without the failure of the section of weld that yielded first.* Therefore, the stress could not exceed the yield stress anywhere. The weld design given in Appendix 3 shows the calculations for the design procedure, and should be referred to for clarification of the following description.

In the plan view of a stub, the tension weld at one end and a compression region under the stub at the other end were considered as "members" that had to resist the axial loads and bending moment that were applied to them. With an assumed effective compression (contact) area and a trial length of tension weld, the area and moment of inertia of the "member" could be calculated, as well as the location of the neutral axis. The forces acting on the stub were then reduced to one axial force (normal to a plan view of the stub) and one bending moment, acting at the neutral axis. The stress of the extreme fibre (tension weld across the end of the stub) was then computed using the relationship

$$\sigma = \frac{P}{A} \pm \frac{My}{I} \qquad (2)$$

where σ = normal stress
P = axial load
M = bending moment
A = cross-sectional area
I = moment of inertia
y = distance from neutral axis to fibre where stress is being calculated.

The required additional length of weld for the tension end of each stub was thus determined.

* It is noted that weld material does not "yield" in the normal sense of the word. Welding electrodes are made from alloys that normally do not exhibit the requisite yield plateau and their yield strength is based on an "offset method" (0.2% permanent strain) definition.

According to these calculations, each stub would require different weld lengths. To avoid possible confusion in the fabricating shop, it was deemed good practice not to specify different weld details at either end of an individual stub. The welding was therefore detailed as shown in Fig. 13, as follows:

Exterior stub: 140 mm of 8 mm fillet weld across each end

406 mm of 8 mm fillet weld along each side at each end

Interior stub: 140 mm of 8 mm fillet weld across each end

152 mm of 8 mm fillet weld along each side at each end

Current stub-girder fabrication practice utilizes fillet welds around the entire perimeter for exterior and interior stubs. The welds specified for the stub-girder in this study differ substantially from present usage. Weld reductions of 43% for the exterior stub and 73% for the interior stub thus were realized. As will be shown, the test specimen welds did not suffer any undue distress, despite the heavily reduced connection sizes.

4.3.3 <u>Design of Stub Stiffeners</u>

The need for bearing stiffeners for the stubs is not immediately apparent from a consideration of the results of the structural analysis. However, fitted stiffeners have traditionally been used in previous designs.

In the Granco test, on the other hand, the stubs were unstiffened. During testing, the web in one of the exterior stubs failed due to crippling at the exterior end at 85% of the ultimate load of the system.[2]

The five member stub Vierendeel representation showed that the exterior member in the exterior stub was the only one with an appreciable compressive axial load. Considering this and the data obtained from the stub specimen tests, the information necessary to decide on the location and type of stiffeners to be used for the full-size girder thus was provided. Partial end plate stiffeners (365 mm x 140 mm x 13 mm plate) were specified for the exterior stubs, with one stiffener at each end. The interior stubs were left unstiffened, as shown in Fig. 1.

In order to design the stiffeners, the free body diagram in Fig. 8 was used, and a compressive load of 342 kN was determined. A convenient size of plate was chosen for the stiffener (365 mm x 140 mm x 13 mm), which, when designed as a column, indicated a factored yield load of:

$$C_r = \phi A_s F_y = 490 \text{ kN} \qquad (3)$$

where C_r = column yield load

ϕ = performance factor (= 0.9)

A_s = area of steel

F_y = yield strength

The stiffener welds were proportioned to resist the compressive design load of 342 kN. A 6 mm fillet weld with 300 MPa base metal and E480XX electrodes has a capacity of 0.97 kN/mm. The total length of weld thus required was 352 mm. A 6 mm stitch weld was specified; 80 mm long at 130 mm on centers, as indicated in Fig. 13.

4.4 Concrete Slab

In an actual building, the design of the composite steel deck and concrete slab is controlled by their capacity in spanning between the floor beams in the regions adjacent to the stub-girder. The thickness of the steel deck, the thickness of the slab, and the size of the shrinkage and temperature mesh thereby can be determined. What remains for the stub-girder design is to check the capacity of the slab in acting as a portion of a Vierendeel truss, and to consider the need for any additional reinforcement.

The slab assembly used in this study utilized a wide-rib profile steel deck, produced by Westeel-Rosco, Ltd., designated as T-30V Hi-Bond Steel Floor. Figure 14 shows a section of this type of deck. The total slab thickness was 106 mm; it was comprised of 76 mm of steel deck and 84 mm of concrete cover above the top of the ribs. A semi-lightweight concrete with a 28 day strength of 27.5 MPa was specified. Shrinkage and temperature reinforcing steel was provided by a 150 x 150 - P9/P9 (old designation: 6x6 - 10/10) welded wire mesh.

The slab had to have sufficient strength and flexibility to ensure that a sudden (brittle) failure would not occur. The desired ductility of the system could only be achieved if the slab remained intact at least until some yielding had allowed a redistribution of the forces in the girder to take place. In order to improve the required characteristics of the slab, it was decided to place one 15M reinforcing bar in the bottom of each of the five centrally located flutes, running the entire length of the stub-girder. Three 15M bars also were placed near the top of the slab over the stubs. Figure 2 shows the position of this longitudinal reinforcement. This method of reinforcing is common in current stub-girder design practice.

In order to estimate the capacity of the slab in compression and bending, a simplified cross-sectional model was devised, using five rectangular shapes, as shown in Figure 15. It was assumed that five times the strength of one rectangular cross-section would give a conservative estimate of the capacity of the slab, since substantial areas of the concrete were ignored (See Fig. 15).

Figure 16 shows the details of the cross section that was analyzed. It was given the same depth (160 mm) as the real slab, and the width (b = 160 mm) was set equal to the minimum flute width. The capacity of the section was determined for positive as well as negative bending, each combined with an axial compressive load. The area of steel in the top of the slab was set equal to 3/5 of the area of one 15M bar. The bottom steel area was taken as the sum of one 15M bar and a 160 mm width of steel deck.

The theoretical interaction diagrams for uniaxial bending of the slab model were produced for both positive and negative bending using a single computer programme. The diagrams are shown in Fig. 17 . The forces and moments in the slab predicted by the Vierendeel analysis all fall within the design envelope, and the slab was thus regarded as having sufficient beam-column capacity.

Transverse reinforcement for the slab was chosen in accordance with the requirements of the design standard[15] for slabs without steel deck (i.e. 0.005 times the effective area of concrete in the longitudinal direction). This was achieved by specifying 15M bars every 400 mm. A standard shrinkage and temperature welded wire mesh was also specified (150 x 150 - P9/P9).

CHAPTER 5

TESTING PROGRAMME

5.1 General Testing Criteria

In order to obtain the most comprehensive results, it would have been preferable to test a series of full-size stub-girders. However, in view of the high cost of experimental research, economy dictated that some other avenue be sought. For this reason the testing programme was divided into two sections. The first section consisted of five tests, on five different slab-stub assemblies (later referred to as stub tests). The results of this preliminary testing phase were analyzed before the second portion of the program was finalized. In this way, information was obtained on the effects of the various fabrication and design details on the strength and behaviour of the floor system.

Having derived the necessary information from the stub tests, the welding, stiffening and reinforcing details that were to be used for the full-size stub-girder were selected. This girder test represented the second section of the testing programme.

A number of material properties tests were specified for the concrete and steel that would be used in the test specimen. Thus, concrete cylinders were to be tested at various ages to determine the properties of the slab, and tension specimens for testing of the steel were to be taken from the bottom chord of the full-size girder. The results are given in Chapter 6.1.

5.2 Stub Specimens

5.2.1 Test Set-up

The purpose of the stub specimens was to investigate the influence of various stub stiffening details on the strength and behaviour of the stub-slab assemblies, and ultimately on the stub-girder itself. Past experiments had shown that the interface between the external stub and the slab was one of the critical regions of the girder, as regards its ultimate strength and failure mode. The assembly would be loaded in such a way that it would be in the same state of stress as that predicted by the Vierendeel analysis. It was anticipated that the stub specimens would behave the same in the stub tests as they would in the full-scale test. As will be seen, this proved to be correct in certain respects.

Figure 18 illustrates the test set-up for the stub specimens, and Fig. 19 gives a cross-section of a typical sample. Note that no web stiffeners are indicated in either figure; these are shown in subsequent figures and are described in detail in the following.

The five specimens were similar in all respects, except for the stiffening detail. Each assembly was composed of a W410X39 stub, 1525 mm long, welded to a W310 x 179 column section along one flange. It is noted that the W310 x 179 column section is not the same as that used for the bottom chord of the full-size girder. This was done specifically to avoid stub test failures in the column, since the purpose of this test was to investigate stiffening details. Stud shear connectors were welded to the top flange of the stub, providing the connection to the concrete slab and steel deck.

The typical stub to column weld details are shown in Fig. 20. The stubs were welded across the flange and along both sides at either end, using 8 mm fillet welds with E480XX electrodes. Different lengths of weld were used along the sides of the stub at each end, due to the anticipated shear distribution along the length of the stub and the tensile force at one end. Weld lengths of 355 mm and 405 mm were used, as shown in Fig. 18.

The concrete slab and steel deck were attached to the top flange of the stub with 15 pairs of 20 mm diameter stud shear connectors, 127 mm long. The studs were evenly distributed along the length of the flange. Since the shear connectors had been welded to the stub prior to the placing of the steel deck, a long slotted hole was cut in the center flute of the deck to allow the shear studs to protrude into and above the flute as the concrete was placed.

Seven 15M reinforcing bars (Grade 400) were placed in the slab, as shown in Fig. 19. Three bars were located near the surface, over the three center deck flutes, and each of the other four were placed on chairs at the bottom of the exterior flutes.

A 150 x 150 - P9/P9 welded wire mesh was located just below the top three bars. The wire mesh constituted the only transverse reinforcement for the stub specimens.

The concrete was specified as 27.5 MPa, Type 10, semi-lightweight, with a maximum aggregate size of 20 mm. This is a commonly used floor slab mix. The average 28 day cylinder strength was 28.2 MPa. Figures 25 and 26 show several of the stub specimens just prior to the concrete being poured.

As is indicated in Fig. 18, the assemblies were tested in a vertical position, using the 6675 kN MTS Universal Testing Machine of the University of Alberta Structural Engineering Laboratory. The load was applied to the slab through a horizontal, W-shape, distributing beam. The distributing beam had a channel welded to its bottom flange, and it was grouted to the slab end to ensure uniform load transfer. Figure 27 shows the stub specimen set-up prior to testing.

The main vertical member, a W310 x 179, was purposely made heavier than the bottom chord of the full-size stub-girder. This was done since the column was required to take an axial compressive load because of the way the test was arranged, but more because the stub test should not fail in the column. As noted previously, the stub tests were intended to study the effects of using different stiffening details, and column behaviour problems might interfere with this purpose. It is also noted that the slab width in the stub test was reduced from 2680 mm, as required for the effective width of the stub-girder, to 1784 mm. This was done to allow placement of the assembly between the columns of the testing machine. It is believed that this change did not affect the relevant aspects of the behaviour of the stub.

Instrumentation for the test consisted of steel and concrete strain gauges, axial load and displacement readouts from the testing machine cross-head, and three linearly variable displacement transducers (LVDT's). Strain gauges were placed on both sides of the web of the stub, as shown in Fig. 28, in an effort to determine the stresses in potentially critical areas. The locations of these areas were predicted using a simple, two-dimensional finite element analysis. Four concrete strain gauges were placed on the face of the slab, as shown in Fig. 29. Figure 18 indicates the LVDT placement, under the bottom edge of the stub and slab to determine the relative movements between slab and stub, stub and column, and slab and column.

Each composite stub-slab specimen was, in turn, welded to the column, instrumented, run through a load-unload cycle, and then loaded to failure.

5.2.2 Description of Stub Specimens

5.2.2.1 Specimen I - No Stiffeners

This specimen is illustrated in Fig. 20. Web stiffeners were purposely omitted from this assembly, which was primarily intended as a control sample. The relative strength and stiffness of the other stubs could therefore be given relative to No. I. In addition, this test also would provide data on the behaviour of an unstiffened assembly, which might be possible to use as a lightly stressed interior stub.

However, the control aspect of this experiment was complicated by the fact that the test set-up was altered following the first test. It had originally been intended to test all specimens at a slight angle (3.7^{o}) from the vertical, in order to keep a proportional shear force present in the slab, together with the axial load, as the Vierendeel analysis indicated should be the case. The first stub test failed prematurely because of local bending in the slab, which was accentuated as the load increased, and the assembly deformed. For this reason the tilted set-up was not used for all the other tests. Although this made for slightly different force conditions in the slab, as compared to those of the full-size girder, subsequent evaluations showed that the vertical position did not interfere with the prime purpose of these tests.

5.2.2.2 Specimen II - Full End-Plate Stiffeners

This specimen is illustrated in Fig. 21. End-plate stiffeners extending over the full height of the stub were stitch-welded at both ends. This stiffening method results in faster fabrication at less cost. A disadvantage of these stiffeners would be the stress concentrations that might develop at the top, where the slab, stub and stiffener meet.

5.2.2.3 Specimen III - Partial End-Plate Stiffeners

This specimen is illustrated in Fig. 22. To avoid the stress concentration problem of the full height end-plate, end-plates that stopped 25 mm short of the top of the stub were used. Although this detail does not increase the stiffness of the top flange against local flange bending, it does retain the rest of the advantages of the full end-plate option. In particular, their primary objective, namely, to stiffen the web of the stub, is achieved just as well as with the full end-plates.

5.2.2.4 Specimen IV - One Standard Stiffener

This specimen is illustrated in Fig. 23. Intended as a variation of Specimen V, it was recognized that stiffeners are normally needed only in the regions of a web that are loaded in compression. The stiffener pair was placed at the end of the stub in which compression would prevail, and the tension region of the web was left unstiffened.

5.2.2.5 Specimen V - Two Standard Stiffeners

This specimen is illustrated in Fig. 24. A pair of fitted web stiffeners was placed at each end of the stub, as shown. These are typical of what has been used in actual stub-girders in building, although it has not been uncommon also to have a third pair of fitted stiffeners, located at the center of the stub. Note that the stiffener-to-web welds were intermittent (stitch) welds. The same welding detail was specified for the stiffeners in the other stub specimens. Common construction practice has typically required all-around fillet welds.

5.3 Full-Size Stub-Girder

Following an evaluation of the results from the five stub tests and the detailed analysis and design of the stub-girder, a full-size test specimen was fabricated, as shown by the drawings given in Figs. 2 and

13. All welding was done in the fabrication shop, including the welding of the stud shear connectors. The girder and floor beams were shipped to the laboratory, where the girder was fastened to the supporting columns, using double angle connections. The floor beams were then bolted into position on the girder, and the steel deck was installed, spanning between the floor beams. A channel shape was bolted to the column, to support the steel deck at both ends. As was done for the stub specimens, gaps were cut in the steel deck at the stub locations to allow for passage of the shear connectors. It is noted that although the shear connectors normally are welded to the girder after the deck has been placed, this could not be done because of insufficient electrical power in the testing laboratory. However, the strength and behavior of the connectors would not be affected by this procedure. The fact that the deck is placed before the connectors are welded merely represent a practical and economical construction procedure.

Slab reinforcement similar to that used in the stub tests was used for the girder, with two important differences. An additional, continuous 15M reinforcing bar was placed in the bottom of the center flute, running between the pairs of stiffeners. This was done to achieve better slab-to-stub force transfer. Additional transverse reinforcement was also used. 15M bars of length equal to the width of the slab were placed every 400 mm along the full length of the stub-girder, except over the exterior stubs, where they were placed every 300 mm.

The changes in the reinforcing system were made on the basis of the behaviour and failure modes of the stub specimens. The additional longitudinal and transverse reinforcement was intended to increase the shear transferring capacity of the slab. It should also be noted that for the same reason, a transverse reinforcing bar was positioned tightly against the pair of shear studs at each end of the exterior stubs. Figures 30 and 31 show the stub-girder prior to pouring of the concrete, including the slab reinforcement.

Partial end-plate stiffeners were specified for the two exterior stubs, whereas the two interior stubs were left unstiffened. The design calculations showed that no stiffeners would be required for the interior stubs. Stitch welds were used for the stiffeners, and partial perimeter welding was used for the stub-to-girder connection.

High early strength, semi-lightweight concrete was poured, vibrated into place and finished to a smooth, level surface, 84 mm above the steel deck, to give a total slab thickness of 160 mm. The high early strength concrete was specified since scheduling required testing to be completed within a certain time period. The ends of the floor beams were shored during construction. The slab was covered with a vapour barrier, and allowed to cure for ten days, at which time the concrete test cylinders indicated an average strength of 29.4 MPa.

A test frame was built around the girder, allowing hydraulic jacks of 535 kN capacity each to be placed at the quarter points of the stub-girder (floor beam locations). Spreader beams distributed the load from the jack head across the width of the slab. The hydraulic system provided for identical loads to be applied at the three load points.

The stub-girder was instrumented during the slab curing period. The instrumentation consisted of displacement transducers (LVDT's) and steel and concrete strain gauges. Five LVDT's were positioned to measure vertical deflections at the quarter points and at the supports; the latter being done to obtain reference data for the interior recordings. Ten concrete strain gauges were placed across the slab in two rows of five gauges each. Six locations on the bottom chord contained a total of 36 steel strain gauges, arranged in groups of six. Figure 32 shows the strain gauge arrangement for the steel, with two on the top flange, two at mid-height on the web, and two on the bottom flange of the bottom chord. The six sections, at each of which a group of 6 gauges was placed, are also shown in Fig. 32. Instrumentation readings were made automatically throughout the test, using the Nova 2/10 digital computer of the laboratory.

During the test, the load was increased in steps of 8.9 kN until it reached 97.8 kN per jack (the service load), unloaded, and then reloaded in the same steps to a load of 267 kN per jack. At this point the center hydraulic jack ran out of travel and the load had to be released to allow for jack resetting. At the same time, the mid-span LVDT, which had not been functional during the last five readings, also was reset. When these adjustments had been made, the system was reloaded in 44.5 kN intervals until failure.

The cycle during which the load was increased to the service load and then unloaded, allowed observation and recording of the girder behaviour while it was exhibiting elastic properties. The inelastic response was observed as the girder was loaded to failure.

CHAPTER 6

TEST RESULTS

6.1 Material Properties

Standard tests were performed in order to determine certain material properties of the steel and concrete used in the stub-girder tests.

Table 6.1 gives the average concrete strength at 28 days for the Type 10 concrete used in the single stub tests and at 10 days for the Type 30 concrete used in the full-sized girder tests.

Table 6.2 shows the static yield stress values determined for a number of tensile coupons cut from the girder and stubs of the full-size specimen. The weighted averages are also shown.

Concrete Type	Age (days)	Average Density (kg/m^3)	Average Ultimate Strength (f'_c) (MPa)
Type 10 semi-light weight	28	1960	28.2
Type 30 semi-light weight	10	1960	29.4

TABLE 6.1

Material Properties of Concrete.

Designation	Coupon No.	Location	F_y (MPa)
W16x26	1	web	350
	2	web	333
	3	flange	287
	4	flange	291
	5	flange	295
	6	flange	288
Weighted Average			306
W12x58	7	web	337
	8	web	335
	9	flange	305
	10	flange	297
	11	flange	301
	12	flange	310
Weighted Average			308

Table 6.2
Material Properties of Steel.

6.2 Results of Stub Specimen Tests

The load-deflection curves for the unstiffened stub (Specimen I) and the other stub tests are given in Figs. 33 through 37. Reference should be made to Fig. 18, which indicates the locations where displacements were measured. These measurements recorded the longitudinal (i.e. parallel to the slab and the stub) deformations of the assembly.

6.2.1 Stub Specimen I - No Stiffeners

Stub Specimen I was the first to be tested, and the tilted assembly therefore was used.

The load was applied in 22.2 kN increments. Cracking was first noticed in the slab at an applied load of 467 kN. As the loading continued, these cracks propagated along with the formation of new ones. The first cracks to develop were transverse cracks near the top of the slab directly above the end of the stub, as well as a longitudinal crack, running the length of the slab above one edge of the stub flange. The final crack patterns of this and the other stub tests are featured in Figs. 38 through 42.

Stub specimen I failed at a load of 875 kN, which was well below the axial slab load (1600 kN) used as an ultimate load, when designing the exterior stubs for the full-size girder. Failure occurred when the concrete crushed and the steel deck buckled at the top end of the stub. Along with the local crushing came a shear failure in the concrete, directly above the stub and along its entire length. The shear plane extended from the zone of crushed concrete, at the top of the stub, to the bottom of the assembly, where the slab and the stub both ended. In the other direction the plane followed a horizontal line across the top of the shear connectors (30 mm below the concrete surface), and then turned down on each side through the thin part of the slab to the steel deck. The diagram in Fig. 43 depicts a typical cross section of the shear plane. Figure 44 shows the shear plane as it could be seen where it surfaced at the end of the slab.

The white-wash that had been painted on the stub prior to testing flaked off in few locations, indicating that only localized yielding had taken place in the stub. The welds exhibited no signs of distress.

The specimen was dissembled after the test to allow a detailed study of the failure zones in the interior of the assembly. It revealed that the shear connectors had not undergone any noticeable permanent deformation. Careful chipping out of the concrete confirmed the existence of the shear failure plane.

6.2.2 Stub Specimen II - Full End-Plate Stiffener

For this and the subsequent tests, the tilted test set-up was changed to a vertical placement. The change had a significant effect on the failure load of the specimens, but the characteristic features of the failure modes were very similar.

The load was applied in 22.2 kN increments up to a load of 310 kN, and then unloaded. Reloading followed the same incremental procedure until the specimen failed. Cracking in the slab began at a load of 400 kN, and at the failure load of 1025 kN the slab had a crack pattern similar to that of the other specimens. This is shown in Fig. 39.

At a load of approximately 645 kN the whitewash began flaking off in the web of the stub at the top, near the slab. The flaking appears in Fig. 45 as dark, vertical lines on the web.

The specimen failed in the same shear and crushing mode as was described for Specimen I. The deck above the stub was visibly buckled, as shown in Fig. 45. Evidence of the final shear plane is seen in Fig. 39. The concrete area which spalled off the top surface of the slab exposed the heads of several shear connectors. This took place as the specimen reached its maximum load, and failed in the combined shear and compression mode.

6.2.3 Stub Specimen III - Partial End-Plate Stiffeners

Unfortunately, the slab of this specimen had been partially cracked while it was being moved into the laboratory. The crack pattern photograph in Fig. 40 shows the pre-testing crack identified as "0 K." Despite this flaw, Specimen III had the highest failure load of any of the stub specimens, and no discernible detrimental effect therefore could be traced to the existence of the handling crack.

The same loading procedure as before was used, including the load-unload cycle to 310 kN. The failure mode was the same shear-crushing type that was observed for the other specimens, and occurred at a load of 1250 kN. Yielding in the web around the ends of the stub was in evidence. Most of the yield lines appeared to originate close to the ends of the stitch welds at both ends of the stub. The stub and its column welds exhibited no other signs of distress. Figure 46 shows that the intersection of the shear plane with the end of the slab was very pronounced.

6.2.4 Stub Specimen IV - Single Standard Stiffener

The load-reload cycle to 310 kN, using increments of 22.2 kN was used for this specimen as well. At a load of 690 kN there were signs of yielding in the web of the stub at the stitch welds that fastened the single stiffener to the compression zone of the web (at the bottom of the stub, according to Fig. 18). At a load of 845 kN, yield lines also became evident in the flange of the stub in the vicinity of the stiffener location as may be observed in Fig. 24. A major crack developed suddenly in the slab when the load reached 1020 kN, although some cracking appeared at a load of 645 kN.

This specimen demonstrated considerable yielding throughout the stub, and far more than any of the other specimens. Yield lines extended approximately 400 mm upward on the web, from their origin around the stiffener stitch welds, as seen in Fig. 47. The typical shear and crushing of the concrete that was the failure mode of the other specimens also was the mode of this stub. It took place at a load of 1210 kN.

At the tension end of the web, where no stiffeners were used, the weld across the end of the stub flange developed a crack directly opposite the column and stub web. The stub had started to pull away from its support ("chord"), and noticeable flange bending in the stub (in a prying manner) as well as longitudinal weld yielding had taken place.

6.2.5 Stub Specimen V - Two Standard Stiffeners

The loading procedure for this specimen was the same as for the other ones. The load reached 1155 kN before the typical shear and crushing failure took place. Whitewash flaking in the stub was similar to that of Specimen IV, but not as severe. This is directly attributed to the presence of the second stiffener. After failure it was noticed that the welds that fastened the top (tension) stiffeners to the flange of the stub (column side) both had torn. This can be seen in Fig. 48.

Of special interest are the diagonal cracks which developed in the slab surface during loading (labeled by "240" kips in Fig. 42). These cracks were expected after considering the theory presented in Ref. 19.

6.2.6 Summary of Stub Test Results

The following observations concerning the results of the stub tests may be made:

1. All specimens failed at loads below the calculated design ultimate load levels.

2. The same failure mode was evident in all specimens; namely, a shear and crushing failure in the slab that caused the block of concrete enclosing the shear studs to lose its ability to transfer load.

3. The cracking patterns of the slabs were similar, each having longitudinal, transverse and diagonal cracking to various extents.

4. Abandoning the tilted set-up used for Specimen I generally increased the failure loads, but did not alter the mode of failure.

5. The top of the stub and the concrete slab exhibited very little differential displacement where it was measured. (See Figs. 33 to 37).

6. The shear connectors were largely undeformed.

7. Different amounts of yielding appears in the webs of the stubs. However, yielding was not severe in any case.

8. Specimen IV and V exhibited the greatest amounts of yielding in the stubs. Each of these specimens also had localized weld failures in the form of transverse weld cracks. The longitudinal column-to-stub welds also showed signs of yielding.

9. With the exception of Specimen II, all the load-displacement curves had slopes of approximately the same magnitude. The slope

of the curve of Specimen II was about half that of the other stub tests. There is no apparent reason why this should be so, and it has been attributed to faulty test read-out.

10. Specimen III attained the highest load, and generally showed the least distress.

11. Strain gauge data were of relatively little use in the evaluation of the response of the stubs.

6.3 Test Results For Full-Size Girder

6.3.1 Visual Observations

All relevant data regarding layout, materials, and so on, for the test girder, have already been given (Sections 5.3 and 6.1). The testing procedure has also been described. The results were obtained in the form of visual observations, strain gauge data, and deflection readings.

The girder was initially loaded in increments of 8.9 kN/load point, up to its service load of 98 kN. At this load level, no distress was detected anywhere in the girder. Following unloading, the same load increments were used as the girder was reloaded.

Very minor yield lines started appearing in the exterior stubs at an applied load of 133 kN per jack, but significant amounts did not appear until the jack forces had reached approximately 190 kN. By this time the design ultimate load of 188 kN had been exceeded. (The design ultimate load equals the ultimate load, as determined from the analysis and design, minus the dead load of the specimen, i.e. 228 - 40 = 188 kN).

At a load of approximately 133 kN/jack, a longitudinal crack, about 1 meter in length, developed in the slab over the top of the north exterior stub. A smaller crack (150 mm long) appeared in the same area at the south end of the girder. The south end crack continued to grow and was approximately 1 meter in length when the load had reached 160 kN. With the load in the 180 to 190 kN range, a second 300 mm long crack appeared, oriented parallel to and about 170 mm away from the first.

Extensive yielding had taken place in the south exterior stub by the time the applied loads had reached 210 kN, as the photograph in Fig. 49 shows. Figure 50 shows the yield line pattern in the north exterior stub for a jack load of about 230 kN (note that the black spot in the web of the bottom chord is electrical tape, covering strain gauges). At this location yielding was also evident in the flanges of the stub (see Fig. 50). At a load of approximately 260 kN per jack, diagonal yield lines appeared in the bottom chord under the ends of the interior stub, as shown in Fig. 51. Some yielding also had occurred at the toe of the interior stub, as can be seen in Fig. 51.

The center hydraulic jack ran out of stroke as the load reached 267 kN. The system was therefore unloaded and the jack reset. At the same time, two LVDT's that also had run out of stroke, were adjusted. By this time many more longitudinal and diagonal cracks had developed in the slab over both exterior stubs. Longitudinal cracks also had become visible over each of the interior stubs.

The system was reloaded, and upon reaching a load of 265 kN/jack, a shear stud failure occurred at the south end of the girder. Loud "popping" sounds could be heard as some of the shear connectors on the south exterior stub failed in combined shear and pull-out. At this point the test was discontinued.

At this stage almost all whitewash had disappeared from the exterior stubs; the interior stubs had separated from the slab (by a small amount over the interior ends); and the bottom flanges of the interior stubs had separated (3 mm gap) from the chord flanges over the unwelded (central) portion.

The interior stubs were largely unyielded; only localized whitewash flaking was visable around the stub ends.

An examination of the surface of the concrete slab revealed extensive longitudinal and diagonal cracking, particularly in the areas over the exterior stubs. Figures 52 to 54 show the cracking patterns, photographed after the completion of the test.

In Fig. 55, a photograph of the failed stub-girder slab shows the test frame, the three loading jacks and the distributing beams. The curvature of the slab should be noted. The cracks had not been highlighted at this point, and are therefore not easy to see.

When the test specimen subsequently was cut apart and dismantled, the concrete around the exterior stubs was carefully chipped away to expose the stud shear connectors. The south exterior stub shear connectors had all been subjected to about the same degree of plastic deformation; they were bent slightly in one direction. Exceptions to this were found in the four studs at the inside (north) end of the stub. They were severely bent in double curvature, and three of them had torn away from the stub flange. The tearing failure of the studs was likely the reason for the "popping" sounds that were heard. The shear studs on the exterior stub at the north end were not deformed as severely.

At the outside end of the south interior stub, the slab and steel deck were removed. Figure 56 shows a photograph of the cross section of the slab, directly over the stub. Notice the failure shear plane, and the similarity between this and the shear planes that were observed in the single stub tests.

6.3.2 LVDT and Strain Gauge Data

Appendix 4 contains the complete set of the data recorded by the Nova computer. Identification of the channels and the units associated with each are as follows:

Channel Number	Description	Units
1	load per jack	kN
2	north support LVDT	mm
3	north quarter point LVDT	mm
4	midspan LVDT	mm
5	south quarter point LVDT	mm
6	south support LVDT	mm
7 - 16	concrete strain gauges	mm/mm
17 - 52	steel strain gauges	mm/mm

The sections of the bottom chord where the steel strain gauges were located are shown in Fig. 32.

The method used to calculate the forces at each section of the bottom chord from the strain gauge data is given in the following. This method is only one of a number of methods possible, since there was a redundant pair of strain gauges at each section.

The axial force, T, and the bending moment, M, were calculated from the following relationships. Reference should be made to Fig. 58 for clarification of the variables used.

$$T = e_m EA \tag{4}$$

$$M = \frac{e_x EI}{y} \tag{5}$$

$$e_x = \frac{(e_t - e_m) + (e_m - e_b(y/y_1))}{2} \tag{6}$$

where T = axial force

M = bending moment

E = modulus of elasticity

A = cross-sectional area

I = moment of inertia

y = distance to extreme fibre

y_1= distance from neutral axis to the top of the bottom flange

e_m= average of two web strain gauges

e_t= average of two top strain gauges

e_b= average of two bottom strain gauges

e_x= average extreme fibre strain

Figures 59 through 70 are graphs which compare the bottom chord forces predicted by the three Vierendeel analyses with the forces computed from the strain gauge data. Since both the analyses and the strain gauge calculations imply elastic behaviour, the comparisons become invalid as the stub-girder begins exhibiting plastic characteristics (see load-deflection curve in Fig. 71).

Figure 71 shows the load-deflection curves for the full-size stub-girder test. For clarity, only the midspan and north 1/4 point curves have been shown. The apparent reversal in deflection, as the end of the second load cycle approached, which is indicated by the midspan deflection curve, was caused by the LVDT at that location running out of stroke. The dashed line indicates the path the curve would likely have followed had this not taken place.

CHAPTER 7

DISCUSSION OF TEST RESULTS

7.1 Single Stub Tests

The stub tests had originally been devised to evaluate the effects of partial perimeter welding on the stubs, and the effects of the various stiffening details. However, the unexpectedly low failure loads of all five of the specimens pointed to a problem area which had not been anticipated: severe slab cracking due to a lack of sufficient transverse reinforcement appeared to be the cause of the low strength.

The low failure load of the first specimen was thought to have been caused because a tilted test set-up was used. Although using a vertical set-up did increase the capacity of the remaining specimens, the same failure mode prevailed, and at loads lower than expected.

Two primary reasons are seen as the cause of the early cracking and deterioration of the slabs, rendering them incapable of transmitting load to the stubs. Two apparently important reinforcing details were missing. Thus, the only transverse reinforcing supplied was that of the 150 x 150 P9/P9 welded wire mesh, which is commonly provided as temperature and shrinkage reinforcement. The ratio of transverse reinforcement to longitudinal slab area was 0.0005, which is less than that required by the structural steel design standard[15] for slabs without steel deck (0.005). This small amount of reinforcement was not sufficient to maintain the integrity of the slab once longitudinal shear cracks had formed. The result was the formation of a shear plane over the top of the stud heads, just below the slab surface, and the crushing of the concrete in the center flute at the top of the slab.

The failure loads would undoubtedly have been higher had the second detail not also been omitted. As seen in Fig. 19, the only longitudinal reinforcing bar, provided near the shear studs, was located near the surface of the slab at about the same elevation as the heads of the studs, and very close to the shear failure plane. No reinforcing bars were provided in the centre flute. This block of concrete, containing the shear studs, and bounded by the shear plane, remained intact and could have used additional longitudinal reinforcement to help transfer the load to the rest of the slab.

The slab and the reinforcement away from the immediate vicinity of the centre flute did not appear to be fully effective parts of the system. The "handling" crack that was present in Specimen III did not prevent it from attaining the highest load of any of the specimens. The loss of approximately one third of the slab, along with its reinforcement, therefore attest to the lower effectiveness of the outer slab regions.

Because of the low failure loads, the stubs were not stressed to the level for which the various details were designed. It was therefore not possible to determine what the behaviour of the welding and stiffening details would be at the design ultimate load. However, valuable information concerning the details was obtained from observing and comparing the performances of the specimens in the ranges of loading to which they were subjected

The need for a stiffener that is welded to the chord at the tension end of the stub is apparent from the local weld failures that occurred in Specimens IV and V (see Fig. 48). In Specimen IV, the tension in the web at the top of the stub could only be transferred to a small length of the stub-to-chord weld, causing it to fail in tension. In this case the effective length of weld would be that shown in Fig. 72.

In Specimen V the tensile force in the web was transferred to the pair of standard stiffeners provided at that end. However, the tack welds which fastened the stiffeners to the bottom flange of the stub were not sufficient to transfer the force (see Fig. 48).

A comparison of Specimens II and III provides additional useful findings. The two assemblies were identical in all respects, except that the latter had its end-plate stiffeners end 25 mm below the top of the stub. This gave the partial end-plate specimen a 22 percent increase in ultimate load over that achieved by Specimen II. The overall ductility of the two specimens (in terms of total deformation capacity) was very similar but the partial end-plate feature seemed to allow for a significantly better redistribution of internal stress resultants.

The load-deformation data (Figs. 33 - 37) indicated very little difference in the displacements recorded for the slab and for the top of the stub. This attests to good bonding and shear transfer mechanism between the two components. The magnitudes of the deflections are on the large side, since for each test, no measurements were taken to determine the amount of rotation of the entire assembly (rotation about the base of the column) as the load was applied.

The shear studs which were exposed after the completion of the tests showed only minimal signs of permanent deformation. This was to be expected, since at no time did the applied loads reach or exceed the design ultimate loads.

On the basis of these observations and the test results, the following conclusions were made about the details to be used in the full-size stub-girder test.

1. The failure load for Specimen I (unstiffened web) was well above the calculated load that an interior stub would be expected to carry. Unstiffened stubs would therefore be used for the interior stubs in the full-size test.

2. Specimen III (partial end-plate stiffeners) attained the highest load and exhibited the best overall performance. Stubs with partial end-plate stiffeners therefore were chosen for the exterior stubs in the full-size test.

3. The stitch welds that were used for the stiffener-to-web connection behaved satisfactorily, and were therefore specified for use with the full-size girder.

4. The partial perimeter welds that were used between the stubs and the bottom chord exhibited no signs of distress. This type of welding therefore was specified for the full-size girder.

5. Additional transverse reinforcing would be required to maintain the integrity of the slab. The transverse reinforcement ratio recommended by the structural steel design standard for slabs without steel deck would be used throughout the length of the girder. A slightly higher ratio (0.006) would be used over the exterior stubs. This was the location where transverse reinforcement would be needed the most.

6. Reinforcement to transfer load from the block of concrete between

the stubs to the rest of the slab was needed. An additional 15M bar would therefore be placed in the centre flute on the full-size girder, running longitudinally between the shear studs.

7.2. Full-Size Stub-Girder Test

The full-size stub-girder demonstrated that this floor system, with its unconventional stiffening and welding details, possesses a high degree of stiffness as well as excellent ductility. Examination of the load-deflection curve in Fig. 71 shows that the girder behaved in a linearly elastic manner in its service load range. The slight non-linear response on the first load cycle was likely a result of a certain amount of initial "slack" in the system. Virtually all of the service load deflection was recovered upon unloading, and the first part of the reload cycle response curve was close to a straight line. The mid-span deflection at the approximate service load was 20 mm, which gives a deflection to span ratio of 1/670 (i.e. extremely stiff). As the bottom chord and stubs began yielding, the girder response became non-linear, as can be deduced from the load-deflection curves in Fig. 71. The ductile behaviour of the girder is evident.

The welding and stiffening details that had been used all performed their required functions without any undue problems. The unstiffened interior stubs showed very few signs of whitewash flaking and no signs of local web buckling. The exterior stub webs were able to sustain considerable shear deformations and massive yielding without any sign of buckling. However, widespread yielding was not apparent in any of the stubs until after the design ultimate load be been exceeded. No signs of distress were noticed in either the stiffener-to-stub stitch welds or the stub-to-chord partial perimeter welds. Their adequacy was thus confirmed.

Although the combined shear and pull-out failure of the shear connectors at the interior end of the south stub brought about the ultimate failure of the girder, all of the other studs on the girder were only slightly deformed, giving evidence that they had not been overloaded. In fact, the small amount of deformation indicates that the connectors would have been able to carry more load. In other words, they would appear to be overdesigned, as suspected at the outset of the project. The failure of the three studs on the south stub may have been initiated by the unloading and reloading which was done when the center loading jack was repositioned. These studs were severely bent in double curvature, indicating that their load could have been reversed at some point. Further research into the design of the shear connectors is clearly needed.

Slab cracks began developing when the girder load was approximately one half of its design ultimate load. The cracking patterns (Figs. 52 - 54) and the shear plane development (Fig. 56) are very similar to those observed in the single stub tests. However, the sudden crushing failure, characteristic of the stub specimens, never took place. This is a result of the increased longitudinal and transverse reinforcement that was used. The prevention of the sudden failure in the concrete allowed the attainment of loads high enough to yield the steel and thus allow the girder to exhibit its ductile characteristics.

An examination of the moments, shears and axial forces predicted by each of the three Vierendeel analyses (contained in Appendices I and II) demonstrates that the degree of refinement of the method used to model the stubs does not affect the results significantly. The small differences in the bottom chord forces negated the need to compare the converted strain gauge data with each analysis independently. Rather, Fig. 59 to 70 compare the forces computed from the strain data to the forces predicted by the first Vierendeel analysis only (refer to Fig. 32 for identification of the locations of sections 1 through 6). Examination of the data shows that in most cases the predicted and the measured forces are very similar. The differences in the force and moment at section 1 are a result of the slab, which was ignored in the analysis, taking a portion of the end reaction. In addition, the boundary conditions are not quite the ideal pin and roller that were assumed for the analysis. Furthermore, once the girder starts exhibiting inelastic characteristics, the differences between the analytical (elastic) and the experimental results will be accentuated. This is the reason for the differences in the curve shapes at higher load levels in Figs. 59 through 70.

Each of the four modeling techniques (including the non-prismatic beam analysis) underestimated the elastic deflection of the stub-girder. The Vierendeel analyses were the most accurate with an error of approximately 13 percent.

The full-size stub-girder specimen was able to carry approximately 40 percent more load than the value the analysis and design had determined was the design ultimate. Two major reasons are seen for this difference. Firstly, as the girder undergoes inelastic deformation, a certain amount of load redistribution will take place. Secondly, the design ultimate was chosen on the basis of the capacity of the bottom chord in bending and tension at mid-span. In actual fact, the development of a plastic hinge at this location will not result in the overall failure of the girder. Further studies should be directed at analyzing the stub-girder for a number of additional failure modes.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the investigation presented in this thesis, the following conclusions and recommendations can be made:

1. The Vierendeel approach to stub-girder modeling gives satisfactory results. Changes in the number of individual members, orientation of such members (vertical, horizontal or diagonal) give rise only to minor differences in the internal stress resultants throughout the girder. Further studies of the methods of analysis should be aimed at including second order effects, inelastic behaviour, and refined methods of stub discretization (finite element techniques). In particular, a shear element representation of the stub appears promising.

2. Deflection computations based on elastic theory are somewhat unconservative in the service load range. However, improved modeling schemes will likely lead to more accurate results. The current practice of some designers to compute ultimate load deflections should be discontinued. This is primarily because such deflections have no real purpose in the first place; only service load data are needed. Secondly, some such computations have been based on elastic solution schemes, and indicate deflections at ultimate load that are significantly less than those observed at test ultimate. Such calculations are therefore not only philosophically and numerically incorrect, but may lead to entirely false conclusions.

3. Service load deflection to span ratio for the full-size stub-girder was 1/670, which is significantly less than the normal building requirement of 1/360. Analysis and test data alike showed that the girder behaved fully elastically in the service load range.

4. The stub-girder possesses excellent strength as well as ductility. The deflection to span ratio at ultimate load was 1/95, and in spite of this very significant deflection, the girder was capable of carrying load well in excess of the service _after_ it had formally failed.

5. Design and test load ratios were found as:

   (i) $\frac{\text{Design Ultimate Load}}{\text{Design Service Load}} = 2.30$

   (ii) $\frac{\text{Actual Ultimate Load}}{\text{Design Service Load}} = 3.08$

   (iii) $\frac{\text{Actual Ultimate Load}}{\text{Design Ultimate Load}} = 1.34$

   These ratios compare favorably with those found in other investigations. However, the test girder exhibited none of the tendencies to sudden failure that had been observed in the other tests. Rather, its ductile failure demonstrated a significant improvement over past practice. This can be attributed to the changes that were made in the design and detailing of the stub-girder, and is amplified in the following.

6. Improvements in the modeling of the stub-girder are likely to lead to a smaller difference between the actual and the design ultimate loads.

7. The application of the principles of composite design, as they are currently formulated, to the stub-girder appears to give satisfactory results. However, improvements should be sought in the method of finding the effective width of the concrete slab. It is believed that the current approach is too conservative when applied to the stub-girder.

8. The method of shear connector design appears to be conservative, but this is open for further study. The number of shear connectors that was used on the exterior stubs seemed excessive, even though the final failure was related to the studs. It is possible that if the studs had been placed farther apart the failure load would have been higher. Although somewhat speculative at this stage, it is believed that failure of closely spaced studs may occur prematurely.

9. The use of partial end-plate stiffeners instead of traditional fitted stiffeners is recommended. The former give at least as much strength and stiffness, and cost less.

10. The interior stubs can be left unstiffened in many cases. The test girder span was substantial; yet, the interior stubs showed very little signs of distress. Using interior stubs that are shorter than the exterior stubs could also be considered.

11. The current practice of prescribing an "all-around" stub-to-bottom chord weld is overly conservative and costly. The stub-girder utilized only a fraction of this, and still no welds were the cause of any failure. 42 percent of weld material was thus eliminated from each exterior stub; 73 percent for each interior stub.

12. Using stitch welds for the stiffener to web connection can further decrease the amount of weld material required.

13. The amount and method of slab reinforcement are important. Longitudinal reinforcement should be concentrated in the vicinity of the shear connectors. Transverse reinforcement should be concentrated in the slab area over the stubs, particularly for the critical exterior stubs. This could be the subject of continued stub-girder research. For the time being, however, it is recommended that a transverse reinforcement ratio not less than that prescribed by the structural steel design standard, CSA S16. 1-M78, for slabs without steel deck, should be used.

14. The current practice of over-designing the stub-girder details (i.e. stiffeners on all stubs, "all-around" welds) and lack of limitations on the size of the bottom chord member, will produce systems with excess load capacity, but not as much ductility as would be desirable. Further research into this aspect of the stub-girder system is needed.

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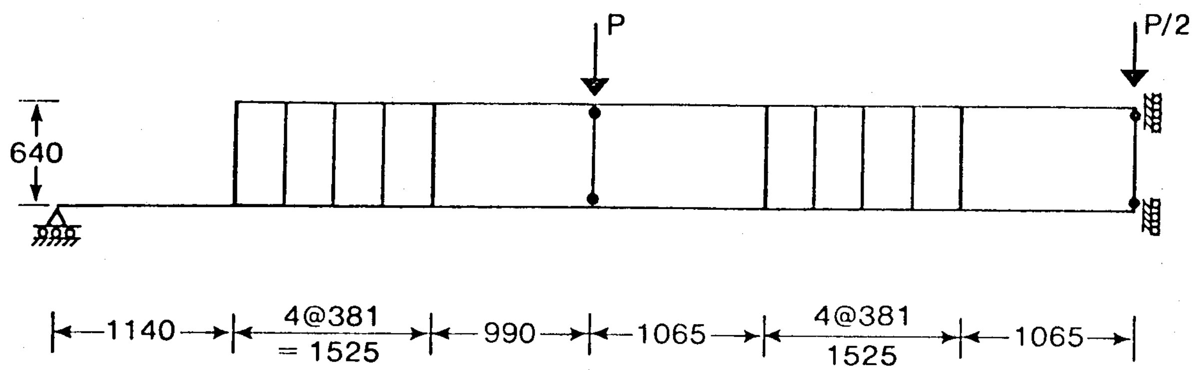
APPENDIX 1

VIERENDEEL ANALYSIS

APPENDIX 1 VIERENDEEL ANALYSIS

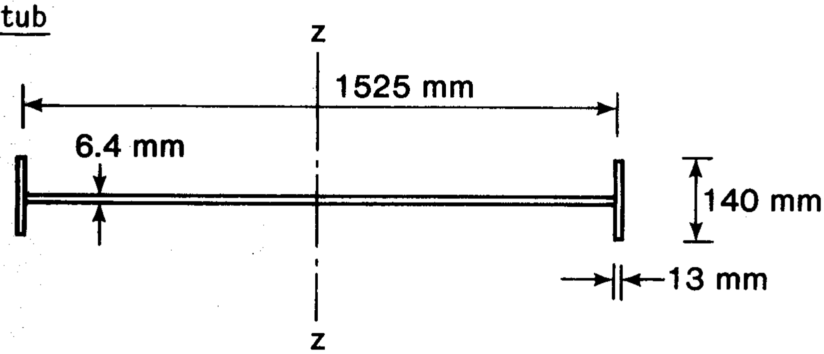
1.1 Vierendeel Model



Vierendeel Model

1.2 Section Properties

Exterior Stub



$$I_{zz} = \frac{0.25(60)^3}{12} + 2(5.5)(0.5)(30)^2 = 9450 \text{ in.}^4 \quad (3.933 \times 10^9 \text{mm}^4)$$

-use 3 vertical members to model stub

$$I_{member} = \frac{9450}{5} = 1890 \text{ in.}^4 (7.867 \times 10^8 \text{mm}4)$$

$$A = 60(0.25) + 2(5.5)(.5) = 20.5 \text{ in.}^2 \quad (1.323 \times 10^4 \text{mm}^2)$$

$$A_{member} = \frac{20.5}{5} = 4.10 \text{ in.}^2 (2.65 \times 10^3 \text{mm}^2)$$

Interior Stub (no stiffener)

$$I_{zz} = \frac{0.25(60)^3}{12} = 4500 \text{ in.}^4 \quad (1.873 \times 10^9 \text{mm}^4)$$

$$I_{member} = 900 \text{ in.}^4 \quad (3.746 \times 10^8 \text{ mm}^4)$$

$$A = 60(.25) = 15 \text{ in.}^2 \quad (9.68 \times 10^3 \text{ mm}^2)$$

$$A_{member} = 3.00 \text{ in.}^2 \quad (1.94 \times 10^3 \text{ mm}^2)$$

Bottom Chord Under Stubs

Bottom Chord: W12 x 58 (W310 x 86)

$I_x = 476 \text{ in}^4$ $(199 \times 10^6 \text{ mm}^4)$

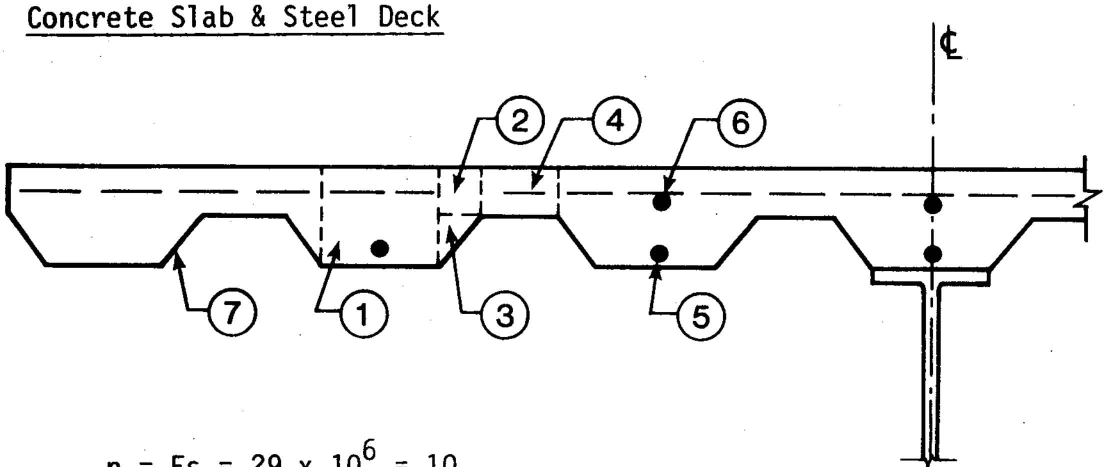
-add to I_x the I of the bottom flange of the stub about the x axis of the bottom chord.

$$I = 476 + (.345)(5.5)(12.19/2)^2$$
$$I = 546 \text{ in.}^4 \quad (227 \times 10^6 \text{ mm}^4)$$

$A = 17.1 + (.345)(5.5)$

$A = 19.0 \text{ in}^2 \quad (1.23 \times 10^4 \text{ mm}^2)$

Concrete Slab & Steel Deck



$n = \frac{Es}{Ec} = \frac{29 \times 10^6}{2.9 \times 10^6} = 10$

$Ec = 33w^{1.5}\sqrt{F'_c}$ $\qquad$ w = 125 lb/cu.ft.

Element No.	No. of Elements	Total Tranformed (in^2)(a)	$\bar{x}$ (in)	$A\bar{x}$	$A\bar{x}^2$	I_{local} (in^4)
1	7	27.56	3.13	86.3	270.0	89.7
2	14	3.87	4.63	17.9	82.9	3.4
3	14	1.79	2.00	3.6	7.2	0.9
4	6	15.60	4.63	72.2	334.4	13.7
5	5	1.55	1.00	1.6	1.6	-
6	3	0.93	1.63	1.5	2.5	-
7	8.8ft.	6.70	1.65	11.1	18.2	10.7
		A = 58.0		194.2	716.8	118.4

$$\bar{x} = \frac{194.2}{58.0} = 3.35$$

$$A\bar{x}^2 = 58.0(3.35)^2 = 650.2$$

$$I = 118.4 - 650.2 + 716.8 = 185.0 \text{ in}^4 \qquad (7.70 \times 10^7 \text{ mm}^4)$$

1.3 Loading

-arbitrary load P at joint no. 7 and P/2 at joint no. 13.

-assigned load value: P = 100 kips

1.4 PFT Output

-output is for an arbitrary load per load point (P) equal to 100 kips

-output units are as follows:

moments - ft. kips

forces - kips

deflections - inches

JOINT COORDINATES

JOINT	IRR	IVR	IHR	X-COORDINATE	Y-COORDINATE
1	1	0	0	0.0	2.100
2	0	0	0	3.250	2.100
3	0	0	0	4.500	2.100
4	0	0	0	5.750	2.100
5	0	0	0	7.000	2.100
6	0	0	0	8.250	2.100
7	0	0	0	11.000	2.100
8	0	0	0	14.000	2.100
9	0	0	0	15.250	2.100
10	0	0	0	16.500	2.100
11	0	0	0	17.750	2.100
12	0	0	0	19.000	2.100
13	1	0	1	22.000	2.100
14	1	1	0	0.0	0.0
15	0	0	0	3.250	0.0
16	0	0	0	4.500	0.0
17	0	0	0	5.750	0.0
18	0	0	0	7.000	0.0
19	0	0	0	8.250	0.0
20	0	0	0	11.000	0.0
21	0	0	0	14.000	0.0
22	0	0	0	15.250	0.0
23	0	0	0	16.500	0.0
24	0	0	0	17.750	0.0
25	0	0	0	19.000	0.0
26	1	0	1	22.000	0.0

MEMBER DATA

MEM NO	J	K	INERTIA	AREA	MODULUS	LENGTH	TYPE
1	1	2	185.000	58.000	29000.000	3.250	1
2	2	3	185.000	58.000	29000.000	1.250	0
3	3	4	185.000	58.000	29000.000	1.250	0
4	4	5	185.000	58.000	29000.000	1.250	0
5	5	6	185.000	58.000	29000.000	1.250	0
6	6	7	185.000	58.000	29000.000	2.750	0
7	7	8	185.000	58.000	29000.000	3.000	0
8	8	9	185.000	58.000	29000.000	1.250	0
9	9	10	185.000	58.000	29000.000	1.250	0
10	10	11	185.000	58.000	29000.000	1.250	0
11	11	12	185.000	58.000	29000.000	1.250	0
12	12	13	185.000	58.000	29000.000	3.000	0
13	15	2	1890.000	4.100	29000.000	2.100	0
14	16	3	1890.000	4.100	29000.000	2.100	0
15	17	4	1890.000	4.100	29000.000	2.100	0
16	18	5	1890.000	4.100	29000.000	2.100	0
17	19	6	1890.000	4.100	29000.000	2.100	0
18	20	7	10.000	26.400	29000.000	2.100	3
19	21	8	900.000	3.000	29000.000	2.100	0
20	22	9	900.000	3.000	29000.000	2.100	0
21	23	10	900.000	3.000	29000.000	2.100	0
22	24	11	900.000	3.000	29000.000	2.100	0
23	25	12	900.000	3.000	29000.000	2.100	0
24	26	13	10.000	13.200	29000.000	2.100	3
25	14	15	476.000	17.100	29000.000	3.250	1
26	15	16	546.000	19.000	29000.000	1.250	0
27	16	17	546.000	19.000	29000.000	1.250	0
28	17	18	546.000	19.000	29000.000	1.250	0
29	18	19	546.000	19.000	29000.000	1.250	0

30	19	20	476.000	17.100	29000.000	2.750	0
31	20	21	476.000	17.100	29000.000	3.000	0
32	21	22	546.000	19.000	29000.000	1.250	0
33	22	23	546.000	19.000	29000.000	1.250	0
34	23	24	546.000	19.000	29000.000	1.250	0
35	24	25	546.000	19.000	29000.000	1.250	0
36	25	26	476.000	17.100	29000.000	3.000	0

JOINT INPUT LOADS FOR LOADING 1

J	MOMENT	VERT FORCE	HOR FORCE
7	0.0	-100.00	0.0
13	0.0	-50.00	0.0

JOINT DEFLECTIONS AND ROTATIONS FOR LOADING 1

J	ROTATION	VERT DEFL	HORZ DEFL
1	0.0	-0.2725907	0.0924648
2	-0.0151599	-0.8638261	0.0924648
3	-0.0151548	-1.0939574	0.0903490
4	-0.0147301	-1.3212077	0.0873497
5	-0.0141210	-1.5407331	0.0835874
6	-0.0133857	-1.7501306	0.0790243
7	-0.0144405	-2.2366076	0.0652502
8	-0.0083979	-2.6583158	0.0502238
9	-0.0072241	-2.7766685	0.0431051
10	-0.0060436	-2.8773889	0.0356735
11	-0.0048030	-2.9599176	0.0279828
12	-0.0035231	-3.0235022	0.0200280
13	0.0	-3.1005873	0.0
14	0.0	0.0	-0.2988227
15	-0.0163528	-0.8526212	-0.2988227
16	-0.0154106	-1.0925723	-0.2923641
17	-0.0148445	-1.3210948	-0.2832081
18	-0.0142516	-1.5409151	-0.2717232
19	-0.0137136	-1.7522689	-0.2577940
20	-0.0141811	-2.2344462	-0.2110745
21	-0.0086851	-2.6559952	-0.1601078
22	-0.0073215	-2.7765307	-0.1383770
23	-0.0060873	-2.8775742	-0.1156911
24	-0.0048565	-2.9601412	-0.0922142
25	-0.0036353	-3.0243319	-0.0679314
26	0.0	-3.0985375	0.0

BEAM MOMENTS AND SHEARS LOADING 1

MEMBER	J MOMENT	K MOMENT	J SHEAR	K SHEAR	J AXIAL	K AXIAL
NO. 1	0.0	0.000	0.0	0.0	-0.000	0.000
NO. 2	32.890	33.194	52.867	-52.867	237.246	-237.246
NO. 3	24.468	49.785	59.402	-59.402	336.328	-336.328
NO. 4	19.307	55.612	59.935	-59.935	421.878	-421.878
NO. 5	15.005	58.840	59.076	-59.076	511.669	-511.669
NO. 6	81.648	53.067	48.987	-48.987	702.066	-702.066
NO. 7	-53.067	97.019	14.651	-14.651	702.066	-702.066
NO. 8	-20.822	49.150	22.662	-22.662	798.245	-798.245
NO. 9	-20.724	49.647	23.138	-23.138	833.330	-833.330
NO. 10	-22.914	51.038	22.499	-22.499	862.383	-862.383
NO. 11	-24.569	51.728	21.727	-21.727	891.989	-891.989
NO. 12	-15.459	72.047	18.862	-18.862	935.755	-935.755
NO. 13	-465.327	-32.890	-237.246	237.246	52.867	-52.867
NO. 14	-150.411	-57.663	-99.083	99.083	6.535	-6.535
NO. 15	-110.563	-69.092	-85.550	85.550	0.533	-0.533
NO. 16	-117.943	-70.617	-89.791	89.791	-0.859	0.859
NO. 17	-259.346	-140.488	-190.397	190.397	-10.089	10.089
NO. 18	0.0	0.0	0.0	0.0	65.663	-65.663
NO. 19	-125.779	-76.197	-96.179	96.179	8.012	-8.012
NO. 20	-45.252	-28.426	-35.085	35.085	0.476	-0.476
NO. 21	-34.280	-26.732	-29.053	29.053	-0.640	0.640
NO. 22	-35.704	-26.468	-29.606	29.606	-0.772	0.772
NO. 23	-55.640	-36.269	-43.766	43.766	-2.864	2.864
NO. 24	0.0	0.0	0.0	0.0	31.138	-31.138
NO. 25	0.0	487.500	150.000	-150.000	0.000	-0.000
NO. 26	-22.173	143.589	97.133	-97.133	-237.246	237.246
NO. 27	6.822	106.426	90.598	-90.598	-336.328	336.328
NO. 28	4.137	108.444	90.065	-90.065	-421.878	421.878
NO. 29	9.499	104.155	90.924	-90.924	-511.669	511.669
NO. 30	155.191	122.594	101.013	-101.013	-702.066	702.066
NO. 31	-122.594	228.642	35.349	-35.349	-702.066	702.066
NO. 32	-102.863	137.035	27.338	-27.338	-798.245	798.245
NO. 33	-91.784	125.361	26.862	-26.862	-833.330	833.330
NO. 34	-91.081	125.458	27.501	-27.501	-862.383	862.383
NO. 35	-89.754	125.095	28.273	-28.273	-891.989	891.989
NO. 36	-69.455	162.868	31.138	-31.138	-935.755	935.755

APPENDIX 2

THREE ALTERNATE MODELING SCHEMES

APPENDIX 2 THREE ALTERNATE MODELING SCHEMES

2.1 Vierendeel Model - 3 member stub

-model numbering scheme the same as in Appendix 1

-stub members 14, 16, 20 and 22 assigned section properties with very small magnitudes to effectively delete the member

-remaining stub members section properties revised as follows:

exterior stub (members 13, 15, and 17)

$$A = \frac{20.5}{3} = 6.83 \text{ in.}^2 \qquad (4.41 \times 10^3 \text{ mm}^2)$$

$$I = \frac{9450}{3} = 3150 \text{ in.}^4 \qquad (1.311 \times 10^9 \text{ mm}^4)$$

interior stub (members 19, 21, and 23)

$$A = \frac{15.0}{3} = 5.0 \text{ in.}^2 \qquad (3.23 \times 10^3 \text{ mm}^2)$$

$$I = \frac{4500}{3} = 1500 \text{ in.}^4 \qquad (0.624 \times 10^9 \text{ mm}^4)$$

-remaining section properties left the same as in Appendix 1

-loading as in Appendix 1

-the computer progam output, showing member forces and nodal displacements, is given on the following pages.

PLANE FRAME AND TRUSS ANALYSIS
STRUCTURE DATA FOR JOB NO. 1
M= 36 NJ= 26 NL= 1 E= 29000.
JOINT COORDINATES

JOINT	IRR	IVR	IHR	X-COORDINATE	Y-COORDINATE
1	1	0	0	0.0	2.100
2	0	0	0	3.250	2.100
3	0	0	0	4.500	2.100
4	0	0	0	5.750	2.100
5	0	0	0	7.000	2.100
6	0	0	0	8.250	2.100
7	0	0	0	11.000	2.100
8	0	0	0	14.000	2.100
9	0	0	0	15.250	2.100
10	0	0	0	16.500	2.100
11	0	0	0	17.750	2.100
12	0	0	0	19.000	2.100
13	1	0	1	22.000	2.100
14	1	1	0	0.0	0.0
15	0	0	0	3.250	0.0
16	0	0	0	4.500	0.0
17	0	0	0	5.750	0.0
18	0	0	0	7.000	0.0
19	0	0	0	8.250	0.0
20	0	0	0	11.000	0.0
21	0	0	0	14.000	0.0
22	0	0	0	15.250	0.0
23	0	0	0	16.500	0.0
24	0	0	0	17.750	0.0
25	0	0	0	19.000	0.0
26	1	0	1	22.000	0.0

MEMBER DATA

MEM NO	J	K	INERTIA	AREA	MODULUS	LENGTH	TYPE
1	1	2	185.000	58.000	29000.000	3.250	1
2	2	3	185.000	58.000	29000.000	1.250	0
3	3	4	185.000	58.000	29000.000	1.250	0
4	4	5	185.000	58.000	29000.000	1.250	0
5	5	6	185.000	58.000	29000.000	1.250	0
6	6	7	185.000	58.000	29000.000	2.750	0
7	7	8	185.000	58.000	29000.000	3.000	0
8	8	9	185.000	58.000	29000.000	1.250	0
9	9	10	185.000	58.000	29000.000	1.250	0
10	10	11	185.000	58.000	29000.000	1.250	0
11	11	12	185.000	58.000	29000.000	1.250	0
12	12	13	185.000	58.000	29000.000	3.000	0
13	15	2	3150.000	6.830	29000.000	2.100	0
14	16	3	1.000	0.010	29000.000	2.100	0
15	17	4	3150.000	6.830	29000.000	2.100	0
16	18	5	1.000	0.010	29000.000	2.100	0
17	19	6	3150.000	6.830	29000.000	2.100	0
18	20	7	10.000	26.400	29000.000	2.100	3
19	21	8	1500.000	5.000	29000.000	2.100	0
20	22	9	1.000	0.010	29000.000	2.100	0
21	23	10	1500.000	5.000	29000.000	2.100	0
22	24	11	1.000	0.010	29000.000	2.100	0
23	25	12	1500.000	5.000	29000.000	2.100	0
24	26	13	10.000	13.200	29000.000	2.100	3
25	14	15	476.000	17.100	29000.000	3.250	1
26	15	16	546.000	19.000	29000.000	1.250	0
27	16	17	546.000	19.000	29000.000	1.250	0
28	17	18	546.000	19.000	29000.000	1.250	0
29	18	19	546.000	19.000	29000.000	1.250	0

30	19	20	476.000	17.100	29000.000	2.750	0
31	20	21	476.000	17.100	29000.000	3.000	0
32	21	22	546.000	19.000	29000.000	1.250	0
33	22	23	546.000	19.000	29000.000	1.250	0
34	23	24	546.000	19.000	29000.000	1.250	0
35	24	25	546.000	19.000	29000.000	1.250	0
36	25	26	476.000	17.100	29000.000	3.000	0

JOINT INPUT LOADS FOR LOADING 1

J	MOMENT	VERT FORCE	HOR FORCE
7	0.0	-100.00	0.0
13	0.0	-50.00	0.0

JOINT DEFLECTIONS AND ROTATIONS FOR LOADING 1

J	ROTATION	VERT DEFL	HORZ DEFL
1	0.0	-0.2537429	0.0925346
2	-0.0152955	-0.8502692	0.0925346
3	-0.0161012	-1.0885289	0.0899354
4	-0.0146988	-1.3223134	0.0873312
5	-0.0150918	-1.5484188	0.0831930
6	-0.0133633	-1.7645079	0.0790500
7	-0.0143254	-2.2480399	0.0652772
8	-0.0083659	-2.6667989	0.0502522
9	-0.0076115	-2.7876816	0.0429576
10	-0.0060233	-2.8909946	0.0356609
11	-0.0051563	-2.9758340	0.0278483
12	-0.0035029	-3.0417708	0.0200338
13	0.0	-3.1181247	0.0
14	0.0	0.0	-0.2990369
15	-0.0161192	-0.8435122	-0.2990369
16	-0.0161966	-1.0876017	-0.2911024
17	-0.0149040	-1.3225764	-0.2831526
18	-0.0149777	-1.5484463	-0.2705205
19	-0.0136518	-1.7649247	-0.2578734
20	-0.0141513	-2.2458524	-0.2111584
21	-0.0086186	-2.6657837	-0.1601966
22	-0.0075839	-2.7878333	-0.1379287
23	-0.0061266	-2.8911935	-0.1156546
24	-0.0050956	-2.9759118	-0.0918056
25	-0.0036254	-3.0418709	-0.0679510
26	0.0	-3.1160414	0.0

BEAM MOMENTS AND SHEARS LOADING 1

MEMBER	J MOMENT	K MOMENT	J SHEAR	K SHEAR	J AXIAL	K AXIAL
NO. 1	0.0	0.000	0.0	0.0	-0.000	0.000
NO. 2	57.206	9.181	53.110	-53.110	291.459	-291.459
NO. 3	-8.599	74.999	53.120	-53.120	292.022	-292.022
NO. 4	43.621	20.196	51.053	-51.053	464.022	-464.022
NO. 5	-19.610	83.426	51.053	-51.053	464.569	-464.569
NO. 6	78.728	52.658	47.777	-47.777	701.999	-701.999
NO. 7	-52.658	95.363	14.235	-14.235	701.999	-701.999
NO. 8	-9.938	35.034	20.077	-20.077	817.973	-817.973
NO. 9	-34.790	59.884	20.075	-20.075	818.203	-818.203
NO. 10	-14.009	37.672	18.931	-18.931	876.052	-876.052
NO. 11	-37.449	61.111	18.930	-18.930	876.259	-876.259
NO. 12	-15.972	71.034	18.354	-18.354	936.025	-936.025
NO. 13	-554.857	-57.206	-291.459	291.459	53.110	-53.110
NO. 14	-0.601	-0.583	-0.564	0.564	0.011	-0.011
NO. 15	-242.580	-118.620	-172.000	172.000	-2.067	2.067
NO. 16	-0.564	-0.585	-0.547	0.547	-0.000	0.000
NO. 17	-336.448	-162.154	-237.430	237.430	-3.276	3.276
NO. 18	0.0	0.0	0.0	0.0	66.458	-66.458
NO. 19	-158.120	-85.425	-115.974	115.974	5.842	-5.842
NO. 20	-0.239	-0.244	-0.230	0.230	-0.002	0.002
NO. 21	-75.609	-45.875	-57.849	57.849	-1.144	1.144
NO. 22	-0.212	-0.223	-0.207	0.207	-0.001	0.001
NO. 23	-80.369	-45.139	-59.766	59.766	-0.576	0.576
NO. 24	0.0	0.0	0.0	0.0	31.646	-31.646
NO. 25	0.0	487.500	150.000	-150.000	0.000	-0.000
NO. 26	67.357	53.756	96.890	-96.890	-291.459	291.459
NO. 27	-53.155	174.254	96.880	-96.880	-292.022	292.022
NO. 28	68.326	55.358	98.947	-98.947	-464.022	464.022
NO. 29	-54.794	178.478	98.947	-98.947	-464.569	464.569
NO. 30	157.970	123.144	102.223	-102.223	-701.999	701.999
NO. 31	-123.144	230.439	35.765	-35.765	-701.999	701.999
NO. 32	-72.319	109.723	29.923	-29.923	-817.973	817.973
NO. 33	-109.484	146.890	29.925	-29.925	-818.203	818.203
NO. 34	-71.281	110.118	31.069	-31.069	-876.052	876.052
NO. 35	-109.907	148.745	31.070	-31.070	-876.259	876.259
NO. 36	-68.376	163.314	31.646	-31.646	-936.025	936.025

2.2 Viernedeel Model - 3 member stub; diagonal member

-model numbering scheme the same as in Appendix 1, with the following exceptions:

member 15 - j end at pt. 19

- k end at pt. 2

member 21 - j end at pt. 21

- k end at pt. 12

-both ends of members 15 and 21 are pinned

-stub members 14, 16, 20 and 22 assigned section properties with very small magnitudes to effectively delete the member

-remaining stub members section properties revised as follows:

exterior stub (members 13, 15, and 17)

$$A = \frac{20.5}{2} = 10.25 \quad (6.61 \times 10^3 \ mm^2)$$

$$I = \frac{9450}{2} = 4725 \ in.^4 (1.967 \times 10^9 \ mm^4)$$

interior stub (members 19, 21, and 23)

$$A = \frac{15.0}{2} = 75. \ in.^2 \quad (4.84 \times 10^3 \ mm^2)$$

$$I = \frac{4500}{2} = 2250 \ in.^4 \quad (0.937 \times 10^9 \ mm^4)$$

-remaining section properties left the same as Appendix 1.

-loading as in Appendix 1.

-The computer program output, showing member forces and nodal displacement is given on the following pages

PLANE FRAME AND TRUSS ANALYSIS
STRUCTURE DATA FOR JOB NO. 1
M= 36 NJ= 26 NL= 1 E= 29000.
JOINT COORDINATES

JOINT	IRR	IVR	IHR	X-COORDINATE	Y-COORDINATE
1	1	0	0	0.0	2.100
2	0	0	0	3.250	2.100
3	0	0	0	4.500	2.100
4	0	0	0	5.750	2.100
5	0	0	0	7.000	2.100
6	0	0	0	8.250	2.100
7	0	0	0	11.000	2.100
8	0	0	0	14.000	2.100
9	0	0	0	15.250	2.100
10	0	0	0	16.500	2.100
11	0	0	0	17.750	2.100
12	0	0	0	19.000	2.100
13	1	0	1	22.000	2.100
14	1	1	0	0.0	0.0
15	0	0	0	3.250	0.0
16	0	0	0	4.500	0.0
17	0	0	0	5.750	0.0
18	0	0	0	7.000	0.0
19	0	0	0	8.250	0.0
20	0	0	0	11.000	0.0
21	0	0	0	14.000	0.0
22	0	0	0	15.250	0.0
23	0	0	0	16.500	0.0
24	0	0	0	17.750	0.0
25	0	0	0	19.000	0.0
26	1	0	1	22.000	0.0

MEMBER DATA

MEM NO	J	K	INERTIA	AREA	MODULUS	LENGTH	TYPE
1	1	2	185.000	58.000	29000.000	3.250	1
2	2	3	185.000	58.000	29000.000	1.250	0
3	3	4	185.000	58.000	29000.000	1.250	0
4	4	5	185.000	58.000	29000.000	1.250	0
5	5	6	185.000	58.000	29000.000	1.250	0
6	6	7	185.000	58.000	29000.000	2.750	0
7	7	8	185.000	58.000	29000.000	3.000	0
8	8	9	185.000	58.000	29000.000	1.250	0
9	9	10	185.000	58.000	29000.000	1.250	0
10	10	11	185.000	58.000	29000.000	1.250	0
11	11	12	185.000	58.000	29000.000	1.250	0
12	12	13	185.000	58.000	29000.000	3.000	0
13	15	2	4725.000	10.250	29000.000	2.100	0
14	16	3	1.000	0.010	29000.000	2.100	0
15	19	2	4725.000	10.250	29000.000	5.423	3
16	18	5	1.000	0.010	29000.000	2.100	0
17	19	6	4725.000	10.250	29000.000	2.100	0
18	20	7	10.000	26.400	29000.000	2.100	3
19	21	8	2250.000	7.500	29000.000	2.100	0
20	22	9	1.000	0.010	29000.000	2.100	0
21	21	12	2250.000	7.500	29000.000	5.423	3
22	24	11	1.000	0.010	29000.000	2.100	0
23	25	12	2250.000	7.500	29000.000	2.100	0
24	26	13	10.000	13.200	29000.000	2.100	3
25	14	15	476.000	17.100	29000.000	3.250	1
26	15	16	546.000	19.000	29000.000	1.250	0
27	16	17	546.000	19.000	29000.000	1.250	0
28	17	18	546.000	19.000	29000.000	1.250	0
29	18	19	546.000	19.000	29000.000	1.250	0

30	19	20	476.000	17.100	29000.000	2.750	0
31	20	21	476.000	17.100	29000.000	3.000	0
32	21	22	546.000	19.000	29000.000	1.250	0
33	22	23	546.000	19.000	29000.000	1.250	0
34	23	24	546.000	19.000	29000.000	1.250	0
35	24	25	546.000	19.000	29000.000	1.250	0
36	25	26	476.000	17.100	29000.000	3.000	0

JOINT INPUT LOADS FOR LOADING 1

J	MOMENT	VERT FORCE	HOR FORCE
7	0.0	-100.00	0.0
13	0.0	-50.00	0.0

JOINT DEFLECTIONS AND ROTATIONS FOR LOADING 1

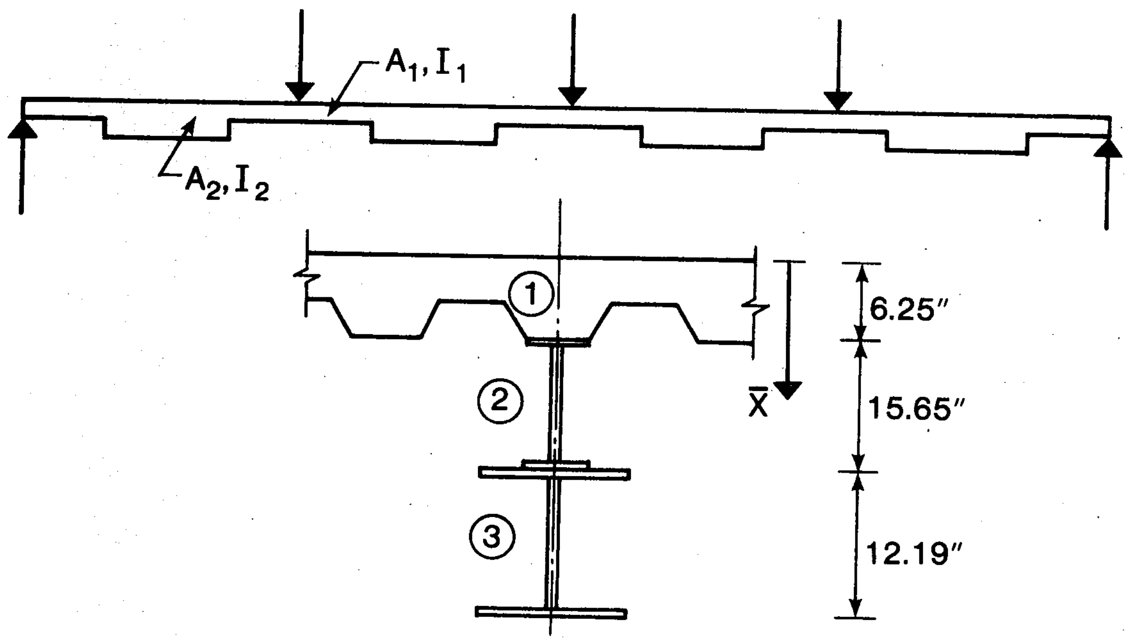
J	ROTATION	VERT DEFL	HORZ DEFL
1	0.0	-0.2456685	0.0958110
2	-0.0151334	-0.8358717	0.0958110
3	-0.0162602	-1.0726612	0.0917137
4	-0.0163454	-1.3185424	0.0876089
5	-0.0153597	-1.5576698	0.0835041
6	-0.0133315	-1.7741944	0.0793920
7	-0.0142134	-2.2540800	0.0656102
8	-0.0083199	-2.6705925	0.0505755
9	-0.0082012	-2.7954276	0.0429436
10	-0.0073623	-2.9130795	0.0353065
11	-0.0057827	-3.0125923	0.0276695
12	-0.0034825	-3.0830079	0.0200272
13	0.0	-3.1589122	0.0
14	0.0	0.0	-0.2900552
15	-0.0157243	-0.8281084	-0.2900552
16	-0.0164175	-1.0702092	-0.2818297
17	-0.0162907	-1.3165576	-0.2735815
18	-0.0153341	-1.5547806	-0.2653333
19	-0.0135577	-1.7725053	-0.2570627
20	-0.0141158	-2.2517902	-0.2103174
21	-0.0085874	-2.6702890	-0.1593225
22	-0.0081889	-2.7968127	-0.1364900
23	-0.0072362	-2.9132032	-0.1136414
24	-0.0057221	-3.0110928	-0.0907929
25	-0.0036536	-3.0821121	-0.0679286
26	0.0	-3.1568215	0.0

BEAM MOMENTS AND SHEARS LOADING 1

MEMBER	J MOMENT	K MOMENT	J SHEAR	K SHEAR	J AXIAL	K AXIAL
NO. 1	0.0	0.0	-0.000	0.000	-0.000	0.000
NO. 2	49.527	-17.643	25.508	-25.508	459.452	-459.452
NO. 3	18.500	13.420	25.536	-25.536	460.282	-460.282
NO. 4	-13.420	45.340	25.536	-25.536	460.282	-460.282
NO. 5	-44.472	76.433	25.569	-25.569	461.107	-461.107
NO. 6	74.501	50.605	45.493	-45.493	702.454	-702.454
NO. 7	-50.605	95.778	15.058	-15.058	702.454	-702.454
NO. 8	7.510	14.586	17.677	-17.677	855.783	-855.783
NO. 9	-13.967	36.043	17.661	-17.661	856.372	-856.372
NO. 10	-36.043	58.119	17.661	-17.661	856.372	-856.372
NO. 11	-57.508	79.606	17.678	-17.678	856.948	-856.948
NO. 12	-15.888	70.611	18.241	-18.241	935.717	-935.717
NO. 13	-584.994	-49.527	-302.153	302.153	91.573	-91.573
NO. 14	-0.887	-0.857	-0.831	0.831	0.028	-0.028
NO. 15	0.0	0.0	0.0	0.0	-170.610	170.610
NO. 16	-0.863	-0.868	-0.824	0.824	0.033	-0.033
NO. 17	-355.896	-150.934	-241.348	241.348	19.924	-19.924
NO. 18	0.0	0.0	0.0	0.0	69.565	-69.565
NO. 19	-218.702	-103.288	-153.328	153.328	2.619	-2.619
NO. 20	-0.617	-0.620	-0.589	0.589	-0.016	0.016
NO. 21	0.0	0.0	0.0	0.0	-18.513	18.513
NO. 22	-0.599	-0.611	-0.576	0.576	0.017	-0.017
NO. 23	-137.540	-63.719	-95.837	95.837	7.731	-7.731
NO. 24	0.0	0.0	0.0	0.0	31.759	-31.759
NO. 25	0.0	487.500	150.000	-150.000	0.000	-0.000
NO. 26	97.494	-24.460	58.427	-58.427	-302.153	302.153
NO. 27	25.347	47.651	58.399	-58.399	-302.983	302.983
NO. 28	-47.651	120.649	58.399	-58.399	-302.983	302.983
NO. 29	-119.786	192.743	58.365	-58.365	-303.808	303.808
NO. 30	163.153	124.241	104.507	-104.507	-702.454	702.454
NO. 31	-124.241	229.068	34.942	-34.942	-702.454	702.454
NO. 32	-10.366	59.730	39.492	-39.492	-838.715	838.715
NO. 33	-59.113	108.498	39.508	-39.508	-839.303	839.303
NO. 34	-108.498	157.883	39.508	-39.508	-839.303	839.303
NO. 35	-157.283	206.646	39.490	-39.490	-839.880	839.880
NO. 36	-69.107	164.384	31.759	-31.759	-935.717	935.717

2.3 Non-Prismatic Beam Model

Section Properties (Transformed Section)



$A = 58.0\ in^2$ $\quad \bar{y} = 2.9$ in $\quad$ transformed units
$I = 185.0\ in^4$

$A = 7.67\ in^2$
$I = 300.0\ in^4$

$A = 17.1\ in^2$
$I = 476.0\ in^4$

Element No.	Transformed Area	$\bar{x}$	$A\bar{x}$	$A\bar{x}^2$	I_{local}
1	58.0	2.90	168.2	487.8	185.0
2	7.67	14.08	108.0	1520.5	300.0
3	17.1	28.00	478.8	13406.4	476.0
Totals 1,2 & 3	82.8		755.0	15,141.7	961.0
Totals 1 & 3 only	75.1		647.0	13,894.2	661.0

$A_1 = 75.1 \qquad \bar{x}_1 = \frac{647.0}{75.1} = 8.62 \text{ in}$

$A\bar{x}_1^2 = 5574.0$

$I_1 = 661.0 - 5574.0 + 13{,}894.2$

$I_1 = 8981. \text{ in}^4$

$A_2 = 82.8 \qquad \bar{x}_2 = \frac{755.0}{82.8} = 9.12$

$A_2\bar{x}_2^2 = 6884.4$

$I_2 = 961 - 6884.4 + 15{,}414.7$

$I_2 = 9491 \text{ in}^4$

The computer program output, showing member forces and nodal displacement, is given on the following pages.

PLANE FRAME AND TRUSS ANALYSIS
STRUCTURE DATA FOR JOB NO. 1
M= 10 NJ= 11 NL= 1 E= 29000.

JOINT COORDINATES

JOINT	IRR	IVR	IHR	X-COORDINATE	Y-COORDINATE
1	1	1	1	0.0	0.0
2	0	0	0	3.250	0.0
3	0	0	0	8.250	0.0
4	0	0	0	14.000	0.0
5	0	0	0	19.000	0.0
6	0	0	0	22.000	0.0
7	0	0	0	25.000	0.0
8	0	0	0	30.000	0.0
9	0	0	0	35.750	0,0
10	0	0	0	40.750	0.0
11	1	1	0	44.000	0.0

MEMBER DATA

MEM NO	J	K	INERTIA	AREA	MODULUS	LENGTH	TYPE
1	1	2	8981.000	75.100	29000.000	3.250	1
2	2	3	9491.000	82.800	29000.000	5.000	0
3	3	4	8981.000	75.100	29000.000	5.750	0
4	4	5	9491.000	82.800	29000.000	5.000	0
5	5	6	8981.000	75.100	29000.000	3.000	0
6	6	7	8981.000	75.100	29000.000	3.000	0
7	7	8	9491.000	82.800	29000.000	5.000	0
8	8	9	8981.000	75.100	29000.000	5.750	0
9	9	10	9491.000	82.800	29000.000	5.000	0
10	10	11	8981.000	75.100	29000.000	3.250	2

INPUT DATA FOR LOADING NUMBER 1

MEMBER	LOAD	X BEGIN	X END	ALPHA
3	100.000	2.750	2.750	0.0
8	100.000	3.000	3.000	0.0

JOINT INPUT LOADS FOR LOADING 1

J	MOMENT	VERT FORCE	HOR FORCE
6	0.0	-100.00	0.0

JOINT DEFLECTIONS AND ROTATIONS FOR LOADING 1

J	ROTATION	VERT DEFL	HORZ DEFL
1	0.0	0.0	0.0
2	-0.0158729	-0.6304306	0.0
3	-0.0136167	-1.5249271	0.0
4	-0.0085603	-2.3002012	0.0
5	-0.0035247	-2.6660207	0.0
6	-0.0000000	-2.7302114	0.0
7	0.0035247	-2.6660207	0.0
8	0.0085603	-2.3002012	0.0
9	0.0136167	-1.5249271	0.0
10	0.0158729	-0.6304306	0.0
11	0.0	0.0	0.0

BEAM MOMENTS AND SHEARS LOADING 1

MEMBER	J MOMENT	K MOMENT	J SHEAR	K SHEAR	J AXIAL	K AXIAL
NO. 1	0.0	487.500	150.000	-150.000	0.0	0.0
NO. 2	-487.500	1237.500	150.000	-150.000	0.0	0.0
NO. 3	-1237.500	1800.000	150.000	-50.000	0.0	0.0
NO. 4	-1800.000	2050.000	50.000	-50.000	0.0	0.0
NO. 5	-2050.000	2200.000	50.000	-50.000	0.0	0.0
NO. 6	-2200.000	2050.000	-50.000	50.000	0.0	0.0
NO. 7	-2050.000	1800.000	-50.000	50.000	0.0	0.0
NO. 8	-1800.000	1237.500	-50.000	150.000	0.0	0.0
NO. 9	-1237.500	487.500	-150.000	150.000	0.0	0.0
NO. 10	-487.500	0.0	-150.000	150.000	0.0	0.0

APPENDIX 3

DETAILED DESIGN OF STUB-GIRDER

APPENDIX 3 DETAILED DESIGN OF STUB-GIRDER

3.1 Determine Design Ultimate Load

It is assumed that the stresses in the bottom chord (due to combined bending and tension) will govern. The design follows the requirements of CSA Standard S16.1-M78.

Section 6 - Midspan

$M_f = 0.495\ P\ \text{kN}\cdot\text{m}$

$T_f = 9.358\ P\ \text{kN}$

where M_f = factored moment

T_f = factored tensile load

and P = load per load point in kN

for a W310 x 36 :

$M_r = \phi\, Z_x F_y = 387\ \text{kN}\cdot\text{m}$

$T_r = \phi\, A_s F_y = 3012\ \text{kN}\cdot\text{m}$

where M_r = factored moment resistance

T_r = factored tensile resistance

ϕ = performance factor = 0.9

As = area of steel

Z_x = plastic section modulus

F_y = yield strength of steel

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1.0$$

$$\left[\frac{9.358}{3012} + \frac{0.495}{387}\right] P \leq 1.0$$

P = 228 kN

Section 4 - exterior end of interior stub

$$M_f = (0.697)\ P\ \text{kN·m}$$

$$T_f = (7.021)\ P\ \text{kN}$$

$$\left[\frac{7.021}{3012} + \frac{0.697}{367}\right] P \leq 1.0$$

$P \leq 242$ kN

Section 1 - exterior end of exterior stub

$$M_f = (1.486)\ P\ \text{kN·m}$$

$$T_f = 0.0$$

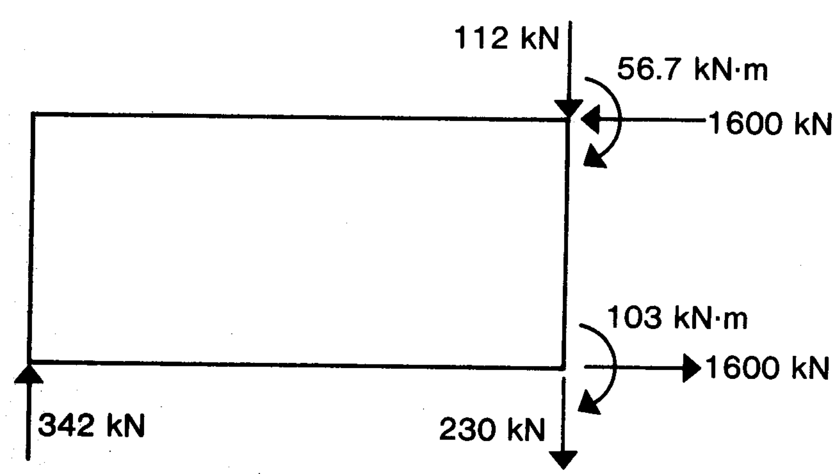
$$\left[\frac{1.486}{387}\right] P \leq 1.0$$

$P \leq 260$ kN

3.2 Design Shear Connectors

3.2.1 Exterior Stub

The free body diagram of the stub is shown below:



Design loads: shear = 1600 kN

tension = 342 kN

Capacity of studs (observe that the formulas are given in Imperial units)

-assume that 20 mm diameter, 127 mm long studs are used.

shear - $q_r = 0.5\ A_{sc} \sqrt{F'_c E_c} \leq 65\ A_{sc}$

where q_r = factored resistance of shear connector

A_{sc} = area of shear connector

F'_c = specified compressive strength of concrete

E_c = elastic modulus of concrete

$q_r = 0.5\ (.4418\ \text{in}^2)\ \sqrt{(4\ \text{ksi})(2900\ \text{ksi})}$

= 23.8 kips/stud

= 106 kN/stud

tension - $P_{uc} = \phi\ C\ K\ A_{fc} \sqrt{F'_c}$ (Ref.16)

where P_{uc} = ultimate tensile capacity of concrete cone

ϕ = 0.85 (reduction factor)

C = constant for concrete type

(= 0.85 for semi-lightweight)

K = 4.0

A_{fc} = area of full conical surface

-from Table 5 of reference

P_{uc} = 19.5 kips/stud

= 87 kN/stud

Number of studs required;

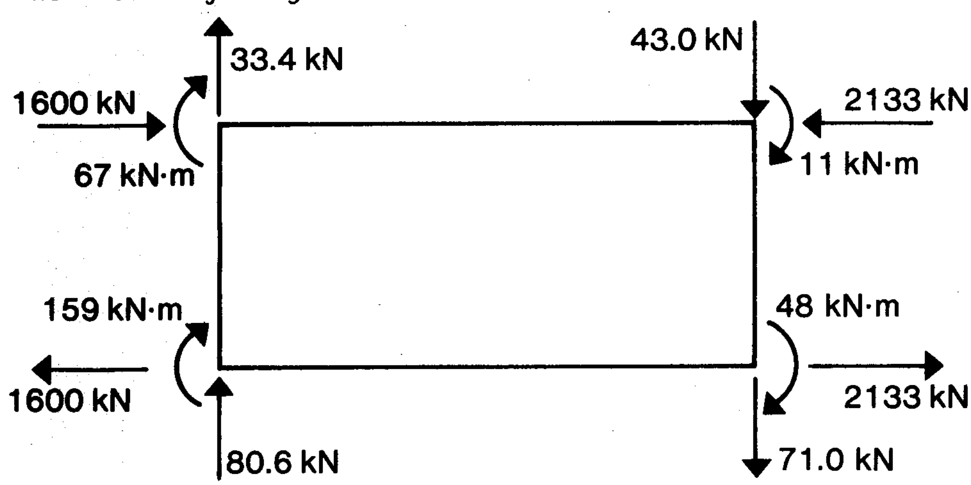
It is assumed that the shear stud capacity in shear and tension is reduced by 50 percent, due to the close spacing of the studs.

Number required: $\left[\frac{1600}{106} + \frac{342}{87}\right] 1.5 = 28.5$ studs

Use 15 pairs of 20 mm diameter studs, 127 mm long

32.2 Interior Stub

The free body diagram of the stub is shown below



Design loads: shear = 533 kN

tension = 114 kN

No. of studs required

No. required $= \left[\frac{533}{106} + \frac{114}{87}\right] 1.5 = 9.5$ studs

Use 5 pairs of 20 mm diameter studs, 127 mm long.

Note: Since so few studs are required for the interior stub, the 50% decrease in shear and tension capacity should not have been applied. However, the design as given is conservative.

3.3 Design stiffeners on exterior stub

Design load = 342 kN

Try 365 mm x 140 mm x 13 mm plate

$C_r = \phi A_x F_y$ = compressive resistance of short column

$C_r = (0.9)(1820)(300) = 490{,}000$ N

$= 490$ kN < 342 kN

therefore O.K.

-design stiffener welds

$V_r = 0.66 \phi A_m F_y$

or

$V_r = 0.5 \phi A_w X_u$

where V_r = factored resistance of weld

A_m = area of fusion face

A_w = effective throat area of weld

X_u = ultimate strength as rated by the electrode classification number.

Try a 6 mm fillet weld - E480 XX electrodes

$V_r = 0.66 (0.9)(6)(300) = 1070$ N

or $V_r = 0.5(0.9)(.7071)(6)(480) = 916$ N

i.e. weld capacity = 0.916 kN/mm

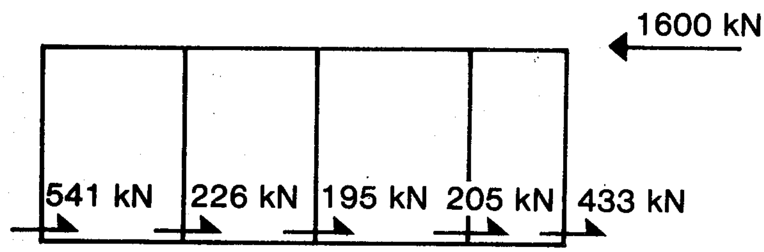
therefore the required length $= \dfrac{342}{.916} = 373$ mm

Use a 6 mm intermittent fillet weld, 80 mm long, at 130 mm center to center on both sides of stiffener.

3.4 Design of Welds - Stub to bottom chord connection

3.4.1 Exterior Stub Welds

Weld length required for shear



Try 8 mm fillet weld, E480XX electrodes

$$V_r = 0.5\ \phi\ A_w X_u$$

$$= 0.5(0.9)(.7071)(8)(480) = 1220\text{N}$$

$$V_r = 1.22 \text{ kN/mm}$$

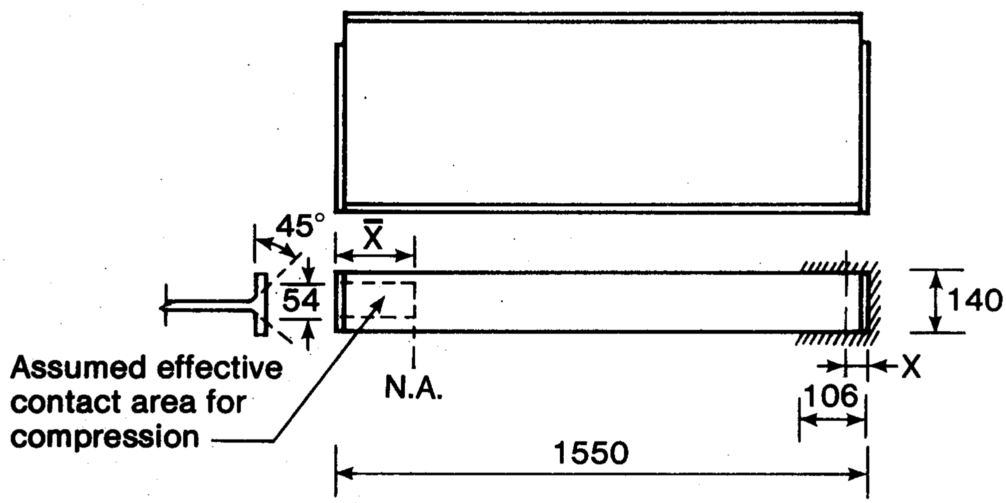
weld length required $= \dfrac{1600}{1.22} = 1310$ mm

weld length at left end $= \dfrac{(541 + 226 + 195/2)}{1600}\ (1310)$

$= 710$ mm

weld length at right end $= (1310 - 710) = 600$ mm

Additional weld length required at right end for tension :



-try 106 mm of 8 mm fillet weld on each side and 140 mm across end

A_w = 352 (.7071)(8) = 1990 mm^2
x = 40 mm

A_{comp} = 140(13) + 54($\bar{x}$) where A_{comp} = contact area

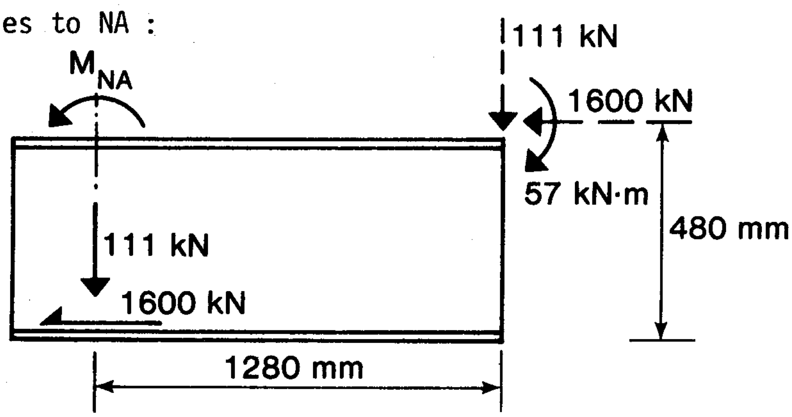
Locate NA:

$$\frac{54(\bar{x})^2}{2} + 140(13)\bar{x} - 1990(1510 - \bar{x}) = 0$$

$\bar{x}$ = 270 mm

A_{comp} = 16400 mm^2

Transfer forces to NA :



$$M_{na} = -57 - 111(1.280) + 1600(.480)$$
$$= 580 \text{ kN m}$$
$$P_{na} = 111 \text{ kN}$$
$$A_{total} = A_w + A_{comp} = 1990 + 16400 = 18390 \text{ mm}^w$$

$$I_{na} = 140(13)(270)^2 + \frac{54(270)^3}{12} + 54(270)\frac{(270)}{2}$$

$$+ \frac{2(106)^3(.7071)(8)}{12} + (106)(2)(.7071)(8)(1227)^2$$

$$+ 140(0.7071)(8)(1280)^2$$

$$I_{na} = 3.19 \times 10^9 \text{ mm}^2$$

extreme fibre stress: $\sigma = \dfrac{P_{na}}{A_{total}} - \dfrac{M_{na}\, y}{I_{na}}$

$$\sigma = \frac{111000}{18390} - \frac{570 \times 10^6}{3.19 \times 10^9}(1280) = 223 \text{ MPa}$$

therefore no yielding will occur.

Total lengths of weld required: (on sides of flange)

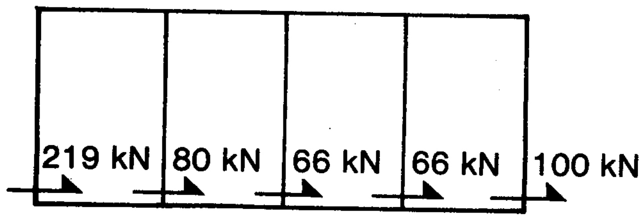
left end - 1310 mm

right end - 600 mm + 212 = 812 mm

For simplicity use same weld length at each end. Use 406 mm of 8 mm fillet weld, both sides, at each end. Use an 8 mm fillet weld across each end of the stub.

3.4.2 Interior Stub Welds

Weld length required for shear.



Weld length at left end = $\frac{332}{1.22} = 272$ mm

Weld length at right end = $\frac{199}{1.22} = 163$ mm

Use same weld length at each end.

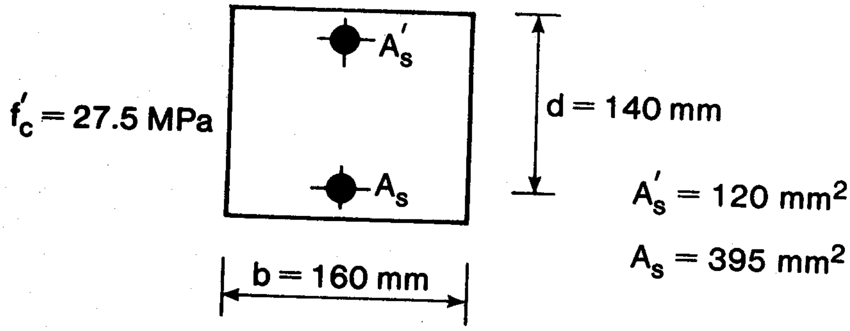
left end - 152 mm of 8 mm weld on each side

right end - 152 mm of 8 mm weld on each side

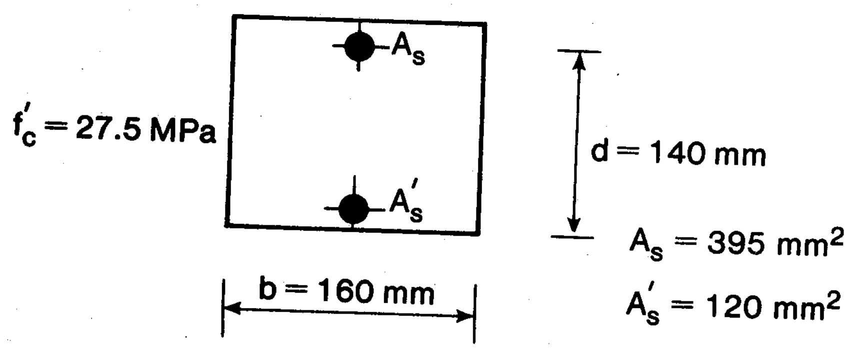
Use 8 mm fillet weld across each end (more than sufficient for tension force at right end).

3.5 Check Slab Capacity

- assume slab capacity is five times the capacity of this member for positive bending:



and this member for negative bending:



The following pages contain the output for the interaction diagrams for positive and negative bending for an individual member model. The interaction diagrams for five times the capacity of one member, are shown in Fig. 17.

Theoretical Interaction Diagram

B	=	6.30 Inches
H	=	6.30 Inches
Distance to Tension Steel	=	5.51 Inches
Column Length	=	35.91 Inches
Slenderness Ratio (L/H)	=	5.70
Steel Ratio	=	2.011
Area of Steel	=	0.798 sq in
Concrete Compresive Strength	=	4000.00 psi
Concrete Modulus of Rupture	=	475.00 psi
Concrete Modulus of Elasticity	=	0.2900E+07 psi
Steel Yield Strength	=	60000.00 psi
Steel Modulus of Elasticity	=	0.2900E+08 psi
Maximum Allowable Strain	=	0.00300
Pure Axial Load	=	203.4 Kips
Pure Bending Moment	=	178.1 Kip-in
Balance Load	=	29.9 Kips
Balance Bending Moment	=	219.0 Kip-in

Bar Number	Steel Area (sq in)	Distance (in)
1	0.19	0.8
2	0.61	5.5

Vertical Load (Kips)	Cross-Sectional Moment (Kip-in)
4.1	186.0
10.2	196.4
20.3	210.6
30.5	217.7
40.7	206.2
50.9	193.1
61.0	182.6
71.2	169.9
81.4	157.9
91.6	144.8
101.7	130.7
111.9	115.2
122.1	97.6
132.2	78.8
142.4	59.9
152.6	40.9
162.8	22.0
172.9	2.5
183.1	-17.1
193.3	-37.2

Theoretical Interaction Diagram

B	=	6.30 Inches
H	=	6.30 Inches
Distance to Tension Steel	=	5.51 Inches
Column Length	=	35.91 Inches
Slenderness Ratio (L/H)	=	5.70
Steel Ratio	=	2.011
Area of Steel	=	0.798 sq in
Concrete Compresive Strength	=	4000.00 psi
Concrete Modulus of Rupture	=	475.00 psi
Concrete Modulus of Elasticity	=	0.2900E+07 psi
Steel Yield Strength	=	60000.00 psi
Steel Modulus of Elasticity	=	0.2900E+08 psi
Maximum Allowable Strain	=	0.00300
Pure Axial Load	=	203.4 Kips
Pure Bending Moment	=	24.4 Kip-in
Balance Load	=	79.4 Kips
Balance Bending Moment	=	216.1 Kip-in

Bar Number	Steel Area (sq in)	Distance (in)
1	0.61	0.8
2	0.19	5.5

Vertical Load (Kips)	Cross-Sectional Moment (Kip-in)
4.1	29.5
10.2	37.2
20.3	108.3
30.5	132.2
40.7	155.1
50.9	176.4
61.0	195.1
71.2	209.1
81.4	215.3
91.6	211.4
101.7	206.4
111.9	199.7
122.1	190.7
132.2	178.7
142.4	163.1
152.6	145.4
162.8	128.2
172.9	110.8
183.1	93.0
193.3	75.1

APPENDIX 4

STUB-GIRDER TEST DATA

	1	2	3	4	5	6
0	0.0	0.0	0.0	0.0	0.0	0.0
1	8.895	0.2093E-01	0.9676	1.349	0.9374	0.3232E-01
2	17.79	0.1046	2.086	3.009	2.028	0.1069
3	26.68	0.1962	3.322	4.868	3.249	0.1765
4	35.58	0.2825	4.506	6.622	4.414	0.2511
5	44.47	0.3766	5.741	8.462	5.645	0.3555
6	53.37	0.4917	7.112	10.53	7.028	0.5096
7	62.26	0.6172	8.471	12.60	8.379	0.6886
8	71.16	0.7532	9.898	14.78	9.814	0.8726
9	80.05	0.8787	11.33	16.93	11.27	1.025
10	88.95	1.023	12.82	19.11	12.76	1.186
11	97.84	1.161	14.32	21.25	14.26	1.360
12	88.95	1.193	13.32	19.92	13.36	1.373
13	71.16	1.036	11.02	16.50	11.07	1.224
14	53.37	0.8447	8.642	12.85	8.677	1.055
15	35.58	0.6277	6.318	9.293	6.355	0.8664
16	17.79	0.4577	3.705	5.390	3.743	0.6675
17	0.0	0.2615	0.9663	1.341	1.042	0.4798
18	17.79	0.3844	3.196	4.582	3.220	0.5979
19	35.58	0.5780	5.714	8.344	5.689	0.7682
20	53.37	0.7427	7.992	11.82	7.979	0.9422
21	71.16	0.9310	10.61	15.85	10.60	1.142
22	88.95	1.109	13.14	19.57	13.14	1.340
23	97.84	1.203	14.53	21.56	14.52	1.469
24	106.7	1.326	15.94	23.59	15.95	1.605
25	115.6	1.454	17.48	25.80	17.47	1.777
26	124.5	1.580	18.98	28.06	18.98	1.927
27	133.4	1.723	20.56	30.46	20.58	2.116
28	142.3	1.873	22.27	33.10	22.34	2.338
29	151.2	2.019	23.95	35.62	24.07	2.513
30	160.1	2.176	25.65	38.14	25.83	2.643
31	169.0	2.333	27.57	41.05	27.74	2.820
32	177.9	2.498	29.60	44.27	29.84	2.996
33	186.8	2.678	31.85	47.83	32.11	3.189
34	195.7	2.872	34.21	51.52	34.52	3.387
35	204.6	3.079	36.71	55.45	37.19	3.596
36	213.5	3.316	39.77	60.49	40.57	3.854
37	217.9	3.436	41.45	63.20	42.41	3.985
38	222.4	3.559	43.22	66.07	44.42	4.114
39	226.8	3.697	45.16	69.15	46.62	4.272
40	231.3	3.850	47.31	72.52	49.07	4.521
41	235.7	3.996	49.53	76.16	51.62	4.766
42	240.2	4.173	52.15	80.59	54.79	5.080
43	244.6	4.367	55.54	86.02	58.84	5.387
44	249.1	4.586	59.03	91.79	63.00	5.656
45	253.5	4.832	63.66	99.18	68.57	6.078
46	255.7	4.974	66.42	101.0	72.21	6.302
47	258.0	5.104	69.11	101.3	75.63	6.565
48	260.2	5.293	72.83	100.8	78.31	6.877
49	262.4	5.465	76.16	99.85	78.31	7.117
50	264.6	5.759	81.40	97.64	78.31	7.491
51	266.8	5.963	85.23	96.21	78.31	7.738
52	222.4	5.757	80.95	97.97	78.32	7.460
53	177.9	5.325	74.41	100.3	78.33	6.933
54	133.4	4.834	67.99	101.3	78.27	6.416
55	88.95	4.314	60.84	97.05	70.98	5.751
56	44.47	3.702	52.57	84.35	62.78	5.029
57	0.0	3.138	44.67	72.29	54.70	4.413
58	0.0	3.107	44.65	72.25	54.69	4.363
59	44.47	3.605	51.08	81.70	61.23	4.895
60	88.95	4.110	58.01	92.13	68.20	5.526
61	133.4	4.587	65.00	102.5	75.34	6.133
62	177.9	5.059	72.23	113.5	82.57	6.715
63	222.4	5.618	79.53	124.6	90.12	7.308
64	244.6	5.846	83.53	130.5	94.07	7.621
65	255.7	5.997	85.76	134.0	96.35	7.808
66	262.4	6.114	87.49	136.8	98.25	7.958

	7	8	9	10	11	12
0	0.0	0.0	0.0	0.0	0.0	0.0
1	-0.5877E-05	-0.1843E-05	0.1096E-05	0.1843E-05	-0.2042E-05	-0.1437E-04
2	-0.1071E-04	-0.1892E-05	0.2142E-05	0.3486E-06	-0.5279E-05	-0.2715E-04
3	-0.1848E-04	-0.6873E-05	0.2490E-06	0.2490E-05	-0.1753E-04	-0.4451E-04
4	-0.2525E-04	-0.9213E-05	0.1494E-05	0.1345E-05	-0.1454E-04	-0.5968E-04
5	-0.3108E-04	-0.1011E-04	0.2291E-05	0.1843E-05	-0.1907E-04	-0.7445E-04
6	-0.3964E-04	-0.1340E-04	0.2042E-05	0.8466E-06	-0.2550E-04	-0.9341E-04
7	-0.4841E-04	-0.1992E-04	0.2440E-05	0.2191E-05	-0.3133E-04	-0.1112E-03
8	-0.5468E-04	-0.2435E-04	0.2938E-05	-0.7968E-06	-0.3685E-04	-0.1303E-03
9	-0.5911E-04	-0.2709E-04	0.5528E-05	0.1494E-06	-0.4248E-04	-0.1467E-03
10	-0.6748E-04	-0.3177E-04	0.2490E-06	-0.5478E-06	-0.4646E-04	-0.1663E-03
11	-0.7899E-04	-0.3750E-04	0.1145E-05	-0.5379E-05	-0.5189E-04	-0.1848E-03
12	-0.7620E-04	-0.3680E-04	-0.3735E-05	-0.2291E-05	-0.4841E-04	-0.1752E-03
13	-0.6375E-04	-0.3262E-04	-0.5976E-05	0.3486E-06	-0.4223E-04	-0.1425E-03
14	-0.5269E-04	-0.2744E-04	-0.9114E-05	0.3735E-05	-0.3396E-04	-0.1118E-03
15	-0.4139E-04	-0.2326E-04	-0.1081E-04	-0.5478E-06	-0.2465E-04	-0.7804E-04
16	-0.2475E-04	-0.1454E-04	-0.6524E-05	0.1643E-05	-0.1280E-04	-0.4471E-04
17	-0.7470E-05	-0.8267E-05	0.2092E-05	0.2241E-05	0.8466E-06	-0.5589E-05
18	-0.2012E-04	-0.9412E-05	0.6973E-06	0.1145E-05	-0.8566E-05	-0.3493E-04
19	-0.3476E-04	-0.1529E-04	-0.3984E-06	0.9960E-07	-0.1873E-04	-0.6906E-04
20	-0.4771E-04	-0.2156E-04	0.4482E-06	-0.1643E-05	-0.2968E-04	-0.9880E-04
21	-0.6215E-04	-0.2998E-04	-0.1843E-05	0.3436E-05	-0.3944E-04	-0.1337E-03
22	-0.7575E-04	-0.3636E-04	-0.9960E-06	0.4482E-06	-0.4632E-04	-0.1659E-03
23	-0.8382E-04	-0.3860E-04	-0.2241E-05	0.1843E-05	-0.5249E-04	-0.1872E-03
24	-0.9094E-04	-0.4427E-04	-0.3337E-05	0.6972E-06	-0.5672E-04	-0.2026E-03
25	-0.1012E-03	-0.4995E-04	-0.4432E-05	-0.2739E-05	-0.6270E-04	-0.2198E-03
26	-0.1127E-03	-0.5354E-04	-0.5677E-05	-0.1992E-06	-0.6793E-04	-0.2407E-03
27	-0.1201E-03	-0.5787E-04	-0.7719E-05	-0.1494E-06	-0.7460E-04	-0.2579E-03
28	-0.1355E-03	-0.6922E-04	-0.8914E-05	-0.1046E-05	-0.7794E-04	-0.2786E-03
29	-0.1431E-03	-0.7884E-04	-0.1355E-04	-0.1444E-05	-0.8237E-04	-0.2970E-03
30	-0.1504E-03	-0.8651E-04	-0.1459E-04	-0.1345E-05	-0.8795E-04	-0.3162E-03
31	-0.1553E-03	-0.1022E-03	-0.2231E-04	-0.1992E-05	-0.9522E-04	-0.3411E-03
32	-0.1615E-03	-0.1106E-03	-0.2933E-04	-0.3486E-06	-0.1015E-03	-0.3661E-03
33	-0.1732E-03	-0.1205E-03	-0.4009E-04	0.1992E-06	-0.1077E-03	-0.3922E-03
34	-0.1801E-03	-0.1299E-03	-0.4522E-04	0.1992E-06	-0.1134E-03	-0.4176E-03
35	-0.1882E-03	-0.1395E-03	-0.5164E-04	0.7968E-06	-0.1186E-03	-0.4457E-03
36	-0.1963E-03	-0.1490E-03	-0.5822E-04	-0.9462E-06	-0.1245E-03	-0.4802E-03
37	-0.2010E-03	-0.1545E-03	-0.6823E-04	-0.5976E-05	-0.1281E-03	-0.4976E-03
38	-0.2033E-03	-0.1600E-03	-0.6987E-04	0.4482E-06	-0.1302E-03	-0.5140E-03
39	-0.2091E-03	-0.1655E-03	-0.8058E-04	-0.4482E-06	-0.1340E-03	-0.5307E-03
40	-0.2157E-03	-0.1744E-03	-0.8880E-04	-0.1245E-05	-0.1384E-03	-0.5513E-03
41	-0.2212E-03	-0.1814E-03	-0.9253E-04	-0.1444E-05	-0.1416E-03	-0.5719E-03
42	-0.2274E-03	-0.1867E-03	-0.9856E-04	-0.8964E-06	-0.1461E-03	-0.5966E-03
43	-0.2336E-03	-0.1935E-03	-0.1073E-03	-0.1295E-05	-0.1523E-03	-0.6200E-03
44	-0.2395E-03	-0.2005E-03	-0.1178E-03	-0.2191E-05	-0.1583E-03	-0.6483E-03
45	-0.2496E-03	-0.2089E-03	-0.1266E-03	-0.1245E-05	-0.1686E-03	-0.6796E-03
46	-0.2574E-03	-0.2129E-03	-0.1316E-03	-0.2789E-05	-0.1731E-03	-0.6946E-03
47	-0.2634E-03	-0.2202E-03	-0.1354E-03	-0.1992E-05	-0.1773E-03	-0.7112E-03
48	-0.2774E-03	-0.2319E-03	-0.1443E-03	-0.2341E-05	-0.1890E-03	-0.7309E-03
49	-0.2846E-03	-0.2379E-03	-0.1466E-03	-0.3187E-05	-0.1964E-03	-0.7475E-03
50	-0.2974E-03	-0.2464E-03	-0.1552E-03	-0.4532E-05	-0.2060E-03	-0.7731E-03
51	-0.3066E-03	-0.2511E-03	-0.1603E-03	-0.2639E-05	-0.2135E-03	-0.7928E-03
52	-0.2814E-03	-0.2302E-03	-0.1749E-03	-0.3287E-05	-0.1918E-03	-0.7158E-03
53	-0.2465E-03	-0.2019E-03	-0.1715E-03	-0.2938E-05	-0.1647E-03	-0.6275E-03
54	-0.2115E-03	-0.1709E-03	-0.1577E-03	-0.4980E-06	-0.1374E-03	-0.5415E-03
55	-0.1740E-03	-0.1388E-03	-0.1328E-03	-0.2291E-05	-0.1050E-03	-0.4447E-03
56	-0.1278E-03	-0.9383E-04	-0.8531E-04	0.8466E-06	-0.6942E-04	-0.3331E-03
57	-0.8063E-04	-0.5573E-04	-0.3989E-04	-0.4482E-06	-0.3909E-04	-0.2244E-03
58	-0.7694E-04	-0.5015E-04	-0.3242E-04	-0.1544E-05	-0.3436E-04	-0.2062E-03
59	-0.1135E-03	-0.7097E-04	-0.4890E-04	-0.2490E-06	-0.5877E-04	-0.2918E-03
60	-0.1524E-03	-0.1055E-03	-0.7724E-04	0.2839E-05	-0.9153E-04	-0.3854E-03
61	-0.1923E-03	-0.1411E-03	-0.1030E-03	0.7968E-06	-0.1204E-03	-0.4800E-03
62	-0.2308E-03	-0.1804E-03	-0.1268E-03	0.5478E-06	-0.1539E-03	-0.5810E-03
63	-0.2718E-03	-0.2151E-03	-0.1517E-03	0.2590E-05	-0.1823E-03	-0.6780E-03
64	-0.2928E-03	-0.2359E-03	-0.1667E-03	0.1394E-05	-0.1987E-03	-0.7289E-03
65	-0.3028E-03	-0.2443E-03	-0.1782E-03	0.8964E-06	-0.2071E-03	-0.7571E-03
66	-0.3112E-03	-0.2499E-03	-0.1864E-03	0.5478E-06	-0.2121E-03	-0.7752E-03

	13	14	15	16	17	18
0	0.0	0.0	0.0	0.0	0.0	0.0
1	-0.1597E-04	-0.1006E-04	-0.1190E-04	-0.1200E-04	0.3041E-04	0.3239E-04
2	-0.2915E-04	-0.1927E-04	-0.2515E-04	-0.2814E-04	0.5800E-04	0.6149E-04
3	-0.4632E-04	-0.2993E-04	-0.3944E-04	-0.4193E-04	0.8288E-04	0.8816E-04
4	-0.6189E-04	-0.4084E-04	-0.5204E-04	-0.5677E-04	0.1056E-03	0.1130E-03
5	-0.7667E-04	-0.5224E-04	-0.6604E-04	-0.7137E-04	0.1303E-03	0.1388E-03
6	-0.9504E-04	-0.6395E-04	-0.8162E-04	-0.8730E-04	0.1547E-03	0.1653E-03
7	-0.1126E-03	-0.7585E-04	-0.9666E-04	-0.1023E-03	0.1801E-03	0.1930E-03
8	-0.1306E-03	-0.8790E-04	-0.1114E-03	-0.1173E-03	0.2052E-03	0.2207E-03
9	-0.1450E-03	-0.9866E-04	-0.1207E-03	-0.1306E-03	0.2300E-03	0.2482E-03
10	-0.1653E-03	-0.1114E-03	-0.1384E-03	-0.1474E-03	0.2536E-03	0.2741E-03
11	-0.1857E-03	-0.1252E-03	-0.1559E-03	-0.1663E-03	0.2823E-03	0.3045E-03
12	-0.1717E-03	-0.1132E-03	-0.1464E-03	-0.1543E-03	0.2583E-03	0.2793E-03
13	-0.1406E-03	-0.8929E-04	-0.1189E-03	-0.1265E-03	0.2097E-03	0.2271E-03
14	-0.1102E-03	-0.6504E-04	-0.9019E-04	-0.9637E-04	0.1590E-03	0.1723E-03
15	-0.7867E-04	-0.4383E-04	-0.6245E-04	-0.6803E-04	0.9883E-04	0.1078E-03
16	-0.4472E-04	-0.2007E-04	-0.3317E-04	-0.3939E-04	0.5291E-04	0.5863E-04
17	-0.8386E-05	0.3835E-05	-0.9960E-07	-0.8516E-05	0.4704E-05	0.8147E-05
18	-0.3834E-04	-0.1648E-04	-0.2634E-04	-0.3182E-04	0.6193E-04	0.6843E-04
19	-0.7388E-04	-0.3974E-04	-0.5508E-04	-0.6534E-04	0.1158E-03	0.1267E-03
20	-0.1030E-03	-0.6136E-04	-0.8262E-04	-0.9139E-04	0.1620E-03	0.1753E-03
21	-0.1358E-03	-0.8402E-04	-0.1124E-03	-0.1218E-03	0.2102E-03	0.2273E-03
22	-0.1677E-03	-0.1074E-03	-0.1393E-03	-0.1506E-03	0.2555E-03	0.2761E-03
23	-0.1877E-03	-0.1221E-03	-0.1562E-03	-0.1654E-03	0.2831E-03	0.3047E-03
24	-0.2049E-03	-0.1341E-03	-0.1679E-03	-0.1796E-03	0.3091E-03	0.3334E-03
25	-0.2220E-03	-0.1464E-03	-0.1849E-03	-0.1991E-03	0.3366E-03	0.3624E-03
26	-0.2408E-03	-0.1590E-03	-0.2005E-03	-0.2151E-03	0.3647E-03	0.3932E-03
27	-0.2604E-03	-0.1714E-03	-0.2131E-03	-0.2296E-03	0.3938E-03	0.4239E-03
28	-0.2803E-03	-0.1853E-03	-0.2325E-03	-0.2494E-03	0.4234E-03	0.4544E-03
29	-0.2987E-03	-0.1959E-03	-0.2472E-03	-0.2671E-03	0.4476E-03	0.4801E-03
30	-0.3163E-03	-0.2075E-03	-0.2620E-03	-0.2838E-03	0.4782E-03	0.5120E-03
31	-0.3370E-03	-0.2211E-03	-0.2809E-03	-0.3073E-03	0.5079E-03	0.5418E-03
32	-0.3650E-03	-0.2358E-03	-0.2961E-03	-0.3291E-03	0.5384E-03	0.5739E-03
33	-0.3893E-03	-0.2517E-03	-0.3212E-03	-0.3513E-03	0.5668E-03	0.6020E-03
34	-0.4133E-03	-0.2679E-03	-0.3398E-03	-0.3741E-03	0.6022E-03	0.6360E-03
35	-0.4389E-03	-0.2846E-03	-0.3606E-03	-0.3994E-03	0.6291E-03	0.6618E-03
36	-0.4648E-03	-0.3037E-03	-0.3879E-03	-0.4294E-03	0.6619E-03	0.6923E-03
37	-0.4808E-03	-0.3112E-03	-0.3986E-03	-0.4410E-03	0.6671E-03	0.6982E-03
38	-0.4948E-03	-0.3193E-03	-0.4106E-03	-0.4572E-03	0.6962E-03	0.7276E-03
39	-0.5079E-03	-0.3292E-03	-0.4235E-03	-0.4743E-03	0.7106E-03	0.7410E-03
40	-0.5255E-03	-0.3412E-03	-0.4388E-03	-0.4904E-03	0.7272E-03	0.7572E-03
41	-0.5439E-03	-0.3500E-03	-0.4536E-03	-0.5080E-03	0.7459E-03	0.7762E-03
42	-0.5650E-03	-0.3617E-03	-0.4727E-03	-0.5277E-03	0.7663E-03	0.7964E-03
43	-0.5862E-03	-0.3771E-03	-0.4930E-03	-0.5526E-03	0.7827E-03	0.8135E-03
44	-0.6094E-03	-0.3909E-03	-0.5133E-03	-0.5718E-03	0.8025E-03	0.8327E-03
45	-0.6389E-03	-0.4081E-03	-0.5364E-03	-0.5968E-03	0.8151E-03	0.8434E-03
46	-0.6545E-03	-0.4167E-03	-0.5519E-03	-0.6136E-03	0.8229E-03	0.8502E-03
47	-0.6709E-03	-0.4259E-03	-0.5644E-03	-0.6274E-03	0.8252E-03	0.8507E-03
48	-0.6880E-03	-0.4357E-03	-0.5803E-03	-0.6450E-03	0.8360E-03	0.8586E-03
49	-0.7060E-03	-0.4444E-03	-0.5947E-03	-0.6610E-03	0.8410E-03	0.8611E-03
50	-0.7320E-03	-0.4580E-03	-0.6174E-03	-0.6857E-03	0.8487E-03	0.8653E-03
51	-0.7515E-03	-0.4690E-03	-0.6364E-03	-0.7043E-03	0.8539E-03	0.8687E-03
52	-0.6765E-03	-0.4076E-03	-0.5702E-03	-0.6342E-03	0.7090E-03	0.7113E-03
53	-0.5894E-03	-0.3413E-03	-0.4934E-03	-0.5533E-03	0.5645E-03	0.5544E-03
54	-0.5047E-03	-0.2782E-03	-0.4158E-03	-0.4762E-03	0.4296E-03	0.4092E-03
55	-0.4089E-03	-0.2083E-03	-0.3479E-03	-0.3889E-03	0.2868E-03	0.2575E-03
56	-0.3019E-03	-0.1367E-03	-0.2545E-03	-0.2898E-03	0.1456E-03	0.1091E-03
57	-0.1981E-03	-0.6838E-04	-0.1642E-03	-0.1965E-03	0.1212E-04	-0.2944E-04
58	-0.1837E-03	-0.5647E-04	-0.1473E-03	-0.1852E-03	0.1203E-04	-0.2890E-04
59	-0.2683E-03	-0.1142E-03	-0.2196E-03	-0.2619E-03	0.1498E-03	0.1165E-03
60	-0.3582E-03	-0.1776E-03	-0.2942E-03	-0.3480E-03	0.2927E-03	0.2678E-03
61	-0.4520E-03	-0.2425E-03	-0.3696E-03	-0.4321E-03	0.4356E-03	0.4206E-03
62	-0.5487E-03	-0.3119E-03	-0.4517E-03	-0.5179E-03	0.5787E-03	0.5743E-03
63	-0.6469E-03	-0.3832E-03	-0.5370E-03	-0.6068E-03	0.7178E-03	0.7230E-03
64	-0.6988E-03	-0.4218E-03	-0.5797E-03	-0.6573E-03	0.7770E-03	0.7866E-03
65	-0.7256E-03	-0.4409E-03	-0.6035E-03	-0.6798E-03	0.8224E-03	0.8336E-03
66	-0.7463E-03	-0.4547E-03	-0.6201E-03	-0.6964E-03	0.8435E-03	0.8551E-03

	19	20	21	22	23	24
0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.2279E-05	0.1794E-05	-0.1377E-04	-0.1416E-04	-0.2701E-04	-0.2856E-04
2	0.1018E-05	0.3395E-06	-0.3104E-04	-0.3210E-04	-0.5630E-04	-0.5979E-04
3	-0.3783E-05	-0.4364E-05	-0.5257E-04	-0.5388E-04	-0.8952E-04	-0.9505E-04
4	-0.8535E-05	-0.9359E-05	-0.7366E-04	-0.7507E-04	-0.1218E-03	-0.1280E-03
5	-0.1411E-04	-0.1426E-04	-0.9490E-04	-0.9679E-04	-0.1543E-03	-0.1635E-03
6	-0.2003E-04	-0.2012E-04	-0.1180E-03	-0.1204E-03	-0.1796E-03	-0.1947E-03
7	-0.2590E-04	-0.2599E-04	-0.1421E-03	-0.1449E-03	-0.2047E-03	-0.2242E-03
8	-0.3181E-04	-0.3147E-04	-0.1659E-03	-0.1690E-03	-0.2293E-03	-0.2527E-03
9	-0.3787E-04	-0.3744E-04	-0.1904E-03	-0.1938E-03	-0.2554E-03	-0.2815E-03
10	-0.4297E-04	-0.4238E-04	-0.2129E-03	-0.2169E-03	-0.2771E-03	-0.3065E-03
11	-0.4864E-04	-0.4782E-04	-0.2396E-03	-0.2434E-03	-0.3037E-03	-0.3374E-03
12	-0.4685E-04	-0.4617E-04	-0.2253E-03	-0.2286E-03	-0.2714E-03	-0.3043E-03
13	-0.4069E-04	-0.3981E-04	-0.1886E-03	-0.1917E-03	-0.2093E-03	-0.2383E-03
14	-0.3424E-04	-0.3332E-04	-0.1498E-03	-0.1522E-03	-0.1451E-03	-0.1715E-03
15	-0.3123E-04	-0.3007E-04	-0.1110E-03	-0.1127E-03	-0.8346E-04	-0.1068E-03
16	-0.1731E-04	-0.1634E-04	-0.6299E-04	-0.6265E-04	-0.1581E-04	-0.3453E-04
17	-0.7274E-06	-0.7759E-06	-0.1072E-04	-0.9214E-05	0.5543E-04	0.4020E-04
18	-0.3686E-05	-0.3734E-05	-0.4709E-04	-0.4655E-04	-0.3782E-05	-0.2168E-04
19	-0.1246E-04	-0.1251E-04	-0.9102E-04	-0.9175E-04	-0.7298E-04	-0.9529E-04
20	-0.2177E-04	-0.2163E-04	-0.1308E-03	-0.1320E-03	-0.1326E-03	-0.1578E-03
21	-0.3361E-04	-0.3317E-04	-0.1766E-03	-0.1786E-03	-0.2013E-03	-0.2314E-03
22	-0.4364E-04	-0.4282E-04	-0.2176E-03	-0.2204E-03	-0.2630E-03	-0.2960E-03
23	-0.4874E-04	-0.4791E-04	-0.2411E-03	-0.2442E-03	-0.2941E-03	-0.3293E-03
24	-0.5538E-04	-0.5402E-04	-0.2686E-03	-0.2715E-03	-0.3276E-03	-0.3655E-03
25	-0.5989E-04	-0.5863E-04	-0.2942E-03	-0.2976E-03	-0.3475E-03	-0.3890E-03
26	-0.6479E-04	-0.6319E-04	-0.3220E-03	-0.3258E-03	-0.3707E-03	-0.4150E-03
27	-0.6964E-04	-0.6736E-04	-0.3501E-03	-0.3546E-03	-0.3912E-03	-0.4391E-03
28	-0.7293E-04	-0.7032E-04	-0.3776E-03	-0.3825E-03	-0.4090E-03	-0.4584E-03
29	-0.7662E-04	-0.7327E-04	-0.4033E-03	-0.4085E-03	-0.4284E-03	-0.4775E-03
30	-0.8021E-04	-0.7580E-04	-0.4331E-03	-0.4393E-03	-0.4518E-03	-0.5010E-03
31	-0.8273E-04	-0.7686E-04	-0.4611E-03	-0.4682E-03	-0.4725E-03	-0.5213E-03
32	-0.8559E-04	-0.7749E-04	-0.4785E-03	-0.4888E-03	-0.4879E-03	-0.5383E-03
33	-0.8710E-04	-0.7938E-04	-0.5203E-03	-0.5312E-03	-0.5200E-03	-0.5643E-03
34	-0.8816E-04	-0.8084E-04	-0.5547E-03	-0.5670E-03	-0.5528E-03	-0.5917E-03
35	-0.8719E-04	-0.7972E-04	-0.5837E-03	-0.5969E-03	-0.5781E-03	-0.6048E-03
36	-0.8128E-04	-0.7555E-04	-0.6129E-03	-0.6289E-03	-0.6019E-03	-0.6304E-03
37	-0.7638E-04	-0.7255E-04	-0.6216E-03	-0.6379E-03	-0.6120E-03	-0.6639E-03
38	-0.6939E-04	-0.6906E-04	-0.6477E-03	-0.6648E-03	-0.6549E-03	-0.9085E-03
39	-0.6571E-04	-0.6673E-04	-0.6635E-03	-0.6814E-03	-0.6879E-03	-0.1143E-02
40	-0.6479E-04	-0.6881E-04	-0.6793E-03	-0.6979E-03	-0.7307E-03	-0.1475E-02
41	-0.6702E-04	-0.7250E-04	-0.6995E-03	-0.7183E-03	-0.7770E-03	-0.2265E-02
42	-0.8288E-04	-0.8957E-04	-0.7193E-03	-0.7370E-03	-0.9261E-03	-0.2782E-02
43	-0.1076E-03	-0.1165E-03	-0.7360E-03	-0.7525E-03	-0.1078E-02	-0.3154E-02
44	-0.1396E-03	-0.1477E-03	-0.7566E-03	-0.7718E-03	-0.1254E-02	-0.3390E-02
45	-0.1653E-03	-0.1727E-03	-0.7777E-03	-0.7912E-03	-0.1483E-02	-0.3565E-02
46	-0.1811E-03	-0.1880E-03	-0.7906E-03	-0.8035E-03	-0.1622E-02	-0.3635E-02
47	-0.1959E-03	-0.2022E-03	-0.7984E-03	-0.8115E-03	-0.1742E-02	-0.3674E-02
48	-0.2118E-03	-0.2184E-03	-0.8185E-03	-0.8301E-03	-0.1883E-02	-0.3725E-02
49	-0.2235E-03	-0.2285E-03	-0.8293E-03	-0.8389E-03	-0.2000E-02	-0.3754E-02
50	-0.2408E-03	-0.2437E-03	-0.8469E-03	-0.8520E-03	-0.2197E-02	-0.3803E-02
51	-0.2538E-03	-0.2557E-03	-0.8584E-03	-0.8601E-03	-0.2282E-02	-0.3818E-02
52	-0.2481E-03	-0.2498E-03	-0.7490E-03	-0.7459E-03	-0.2148E-02	-0.3642E-02
53	-0.2248E-03	-0.2270E-03	-0.6263E-03	-0.6209E-03	-0.1973E-02	-0.3456E-02
54	-0.1981E-03	-0.2015E-03	-0.5022E-03	-0.4940E-03	-0.1804E-02	-0.3273E-02
55	-0.1662E-03	-0.1701E-03	-0.3647E-03	-0.3522E-03	-0.1617E-02	-0.3066E-02
56	-0.1303E-03	-0.1345E-03	-0.2170E-03	-0.2007E-03	-0.1411E-02	-0.2837E-02
57	-0.9616E-04	-0.1004E-03	-0.7555E-04	-0.5509E-04	-0.1214E-02	-0.2620E-02
58	-0.9709E-04	-0.1012E-03	-0.7560E-04	-0.5533E-04	-0.1214E-02	-0.2620E-02
59	-0.1276E-03	-0.1312E-03	-0.2035E-03	-0.1875E-03	-0.1387E-02	-0.2803E-02
60	-0.1550E-03	-0.1584E-03	-0.3304E-03	-0.3180E-03	-0.1552E-02	-0.2973E-02
61	-0.1812E-03	-0.1836E-03	-0.4555E-03	-0.4447E-03	-0.1728E-02	-0.3164E-02
62	-0.2115E-03	-0.2132E-03	-0.5907E-03	-0.5825E-03	-0.1921E-02	-0.3378E-02
63	-0.2401E-03	-0.2410E-03	-0.7232E-03	-0.7181E-03	-0.2108E-02	-0.3582E-02
64	-0.2511E-03	-0.2514E-03	-0.7811E-03	-0.7784E-03	-0.2180E-02	-0.3645E-02
65	-0.2616E-03	-0.2616E-03	-0.8274E-03	-0.8251E-03	-0.2265E-02	-0.3750E-02
66	-0.2657E-03	-0.2662E-03	-0.8493E-03	-0.8471E-03	-0.2303E-02	-0.3789E-02

	25	26	27	28	29	30
0	0.0	0.0	0.0	0.0	0.0	0.0
1	-0.1047E-04	-0.1111E-04	-0.3977E-05	-0.3686E-05	-0.1178E-04	-0.1397E-04
2	-0.2444E-04	-0.2546E-04	-0.1208E-04	-0.1033E-04	-0.2493E-04	-0.3007E-04
3	-0.4141E-04	-0.4301E-04	-0.2396E-04	-0.1857E-04	-0.4195E-04	-0.5102E-04
4	-0.5819E-04	-0.5979E-04	-0.3550E-04	-0.2774E-04	-0.6062E-04	-0.7240E-04
5	-0.7555E-04	-0.7745E-04	-0.4685E-04	-0.3690E-04	-0.7997E-04	-0.9393E-04
6	-0.9277E-04	-0.9553E-04	-0.5810E-04	-0.4660E-04	-0.9660E-04	-0.1140E-03
7	-0.1116E-03	-0.1149E-03	-0.7017E-04	-0.5504E-04	-0.1136E-03	-0.1348E-03
8	-0.1308E-03	-0.1337E-03	-0.8147E-04	-0.6430E-04	-0.1304E-03	-0.1561E-03
9	-0.1500E-03	-0.1534E-03	-0.9490E-04	-0.7264E-04	-0.1495E-03	-0.1799E-03
10	-0.1672E-03	-0.1705E-03	-0.1065E-03	-0.8157E-04	-0.1650E-03	-0.2005E-03
11	-0.1890E-03	-0.1911E-03	-0.1183E-03	-0.9000E-04	-0.1849E-03	-0.2269E-03
12	-0.1766E-03	-0.1788E-03	-0.1118E-03	-0.8365E-04	-0.1664E-03	-0.2059E-03
13	-0.1473E-03	-0.1487E-03	-0.9379E-04	-0.6954E-04	-0.1304E-03	-0.1626E-03
14	-0.1169E-03	-0.1177E-03	-0.7584E-04	-0.5451E-04	-0.9587E-04	-0.1216E-03
15	-0.8986E-04	-0.9030E-04	-0.6101E-04	-0.4335E-04	-0.6697E-04	-0.8457E-04
16	-0.5373E-04	-0.5325E-04	-0.3365E-04	-0.2153E-04	-0.3312E-04	-0.4088E-04
17	-0.1465E-04	-0.1304E-04	-0.4122E-05	0.1649E-05	0.4849E-05	0.4025E-05
18	-0.4170E-04	-0.4248E-04	-0.1949E-04	-0.1178E-04	-0.2124E-04	-0.2662E-04
19	-0.7575E-04	-0.7643E-04	-0.4141E-04	-0.2861E-04	-0.5523E-04	-0.6983E-04
20	-0.1061E-03	-0.1075E-03	-0.6105E-04	-0.4466E-04	-0.8414E-04	-0.1076E-03
21	-0.1409E-03	-0.1425E-03	-0.8496E-04	-0.6265E-04	-0.1230E-03	-0.1540E-03
22	-0.1716E-03	-0.1741E-03	-0.1068E-03	-0.7963E-04	-0.1600E-03	-0.1982E-03
23	-0.1890E-03	-0.1918E-03	-0.1178E-03	-0.8768E-04	-0.1778E-03	-0.2206E-03
24	-0.2103E-03	-0.2132E-03	-0.1314E-03	-0.9767E-04	-0.2013E-03	-0.2506E-03
25	-0.2299E-03	-0.2328E-03	-0.1429E-03	-0.1059E-03	-0.2181E-03	-0.2753E-03
26	-0.2514E-03	-0.2535E-03	-0.1560E-03	-0.1143E-03	-0.2366E-03	-0.3013E-03
27	-0.2718E-03	-0.2740E-03	-0.1684E-03	-0.1233E-03	-0.2546E-03	-0.3304E-03
28	-0.2916E-03	-0.2943E-03	-0.1782E-03	-0.1300E-03	-0.2696E-03	-0.3578E-03
29	-0.3119E-03	-0.3142E-03	-0.1892E-03	-0.1380E-03	-0.2870E-03	-0.3851E-03
30	-0.3351E-03	-0.3370E-03	-0.2010E-03	-0.1458E-03	-0.3077E-03	-0.4177E-03
31	-0.3576E-03	-0.3602E-03	-0.2094E-03	-0.1523E-03	-0.3267E-03	-0.4509E-03
32	-0.3788E-03	-0.3825E-03	-0.2155E-03	-0.1585E-03	-0.3475E-03	-0.4850E-03
33	-0.4068E-03	-0.4107E-03	-0.2221E-03	-0.1642E-03	-0.3782E-03	-0.5302E-03
34	-0.4368E-03	-0.4409E-03	-0.2284E-03	-0.1698E-03	-0.4154E-03	-0.5831E-03
35	-0.4660E-03	-0.4702E-03	-0.2321E-03	-0.1725E-03	-0.4495E-03	-0.6294E-03
36	-0.5001E-03	-0.5033E-03	-0.2319E-03	-0.1717E-03	-0.4893E-03	-0.6836E-03
37	-0.5120E-03	-0.5154E-03	-0.2297E-03	-0.1695E-03	-0.5050E-03	-0.7041E-03
38	-0.5398E-03	-0.5434E-03	-0.2329E-03	-0.1720E-03	-0.5329E-03	-0.7407E-03
39	-0.5638E-03	-0.5662E-03	-0.2326E-03	-0.1717E-03	-0.5584E-03	-0.7699E-03
40	-0.5897E-03	-0.5920E-03	-0.2305E-03	-0.1698E-03	-0.5821E-03	-0.7967E-03
41	-0.6224E-03	-0.6249E-03	-0.2304E-03	-0.1704E-03	-0.6099E-03	-0.8255E-03
42	-0.6576E-03	-0.6613E-03	-0.2280E-03	-0.1689E-03	-0.6367E-03	-0.8523E-03
43	-0.6929E-03	-0.6987E-03	-0.2242E-03	-0.1664E-03	-0.6561E-03	-0.8725E-03
44	-0.7302E-03	-0.7395E-03	-0.2228E-03	-0.1646E-03	-0.6771E-03	-0.8947E-03
45	-0.7696E-03	-0.7822E-03	-0.2241E-03	-0.1642E-03	-0.6955E-03	-0.9149E-03
46	-0.7893E-03	-0.8046E-03	-0.2252E-03	-0.1666E-03	-0.7045E-03	-0.9251E-03
47	-0.8063E-03	-0.8222E-03	-0.2281E-03	-0.1681E-03	-0.7106E-03	-0.9308E-03
48	-0.8283E-03	-0.8461E-03	-0.2348E-03	-0.1736E-03	-0.7144E-03	-0.9355E-03
49	-0.8434E-03	-0.8626E-03	-0.2370E-03	-0.1741E-03	-0.7195E-03	-0.9424E-03
50	-0.8664E-03	-0.8883E-03	-0.2441E-03	-0.1771E-03	-0.7259E-03	-0.9519E-03
51	-0.8796E-03	-0.9017E-03	-0.2493E-03	-0.1807E-03	-0.7273E-03	-0.9537E-03
52	-0.8060E-03	-0.8283E-03	-0.2057E-03	-0.1456E-03	-0.6119E-03	-0.8210E-03
53	-0.7243E-03	-0.7444E-03	-0.1564E-03	-0.1061E-03	-0.5005E-03	-0.6815E-03
54	-0.6439E-03	-0.6627E-03	-0.1061E-03	-0.6770E-04	-0.4000E-03	-0.5472E-03
55	-0.5550E-03	-0.5722E-03	-0.4912E-04	-0.2391E-04	-0.2922E-03	-0.3990E-03
56	-0.4572E-03	-0.4726E-03	0.1178E-04	0.2143E-04	-0.1975E-03	-0.2524E-03
57	-0.3608E-03	-0.3740E-03	0.7410E-04	0.6910E-04	-0.1139E-03	-0.1279E-03
58	-0.3624E-03	-0.3749E-03	0.7293E-04	0.6799E-04	-0.1175E-03	-0.1307E-03
59	-0.4479E-03	-0.4627E-03	0.1440E-04	0.1571E-04	-0.1990E-03	-0.2458E-03
60	-0.5266E-03	-0.5441E-03	-0.3695E-04	-0.1770E-04	-0.2777E-03	-0.3775E-03
61	-0.6081E-03	-0.6269E-03	-0.8448E-04	-0.5334E-04	-0.3735E-03	-0.5126E-03
62	-0.6968E-03	-0.7175E-03	-0.1393E-03	-0.9490E-04	-0.4843E-03	-0.6582E-03
63	-0.7846E-03	-0.8070E-03	-0.1915E-03	-0.1353E-03	-0.5978E-03	-0.7974E-03
64	-0.8194E-03	-0.8426E-03	-0.2169E-03	-0.1543E-03	-0.6527E-03	-0.8619E-03
65	-0.8557E-03	-0.8782E-03	-0.2323E-03	-0.1657E-03	-0.6947E-03	-0.9117E-03
66	-0.8712E-03	-0.8950E-03	-0.2392E-03	-0.1717E-03	-0.7124E-03	-0.9321E-03

	31	32	33	34	35	36
0	0.0	0.0	0.0	0.0	0.0	0.0
1	-0.1896E-04	-0.1988E-04	-0.2255E-04	-0.2318E-04	0.3152E-05	0.4849E-05
2	-0.4064E-04	-0.4335E-04	-0.4903E-04	-0.5102E-04	0.4461E-05	0.8292E-05
3	-0.6639E-04	-0.7138E-04	-0.7977E-04	-0.8399E-04	0.4122E-05	0.2570E-05
4	-0.9194E-04	-0.9869E-04	-0.1103E-03	-0.1158E-03	0.2425E-05	0.8244E-05
5	-0.1178E-03	-0.1267E-03	-0.1421E-03	-0.1491E-03	0.1940E-05	0.8583E-05
6	-0.1435E-03	-0.1544E-03	-0.1727E-03	-0.1822E-03	0.2716E-05	0.1004E-04
7	-0.1707E-03	-0.1834E-03	-0.2068E-03	-0.2178E-03	0.3492E-05	0.1256E-04
8	-0.1974E-03	-0.2119E-03	-0.2389E-03	-0.2518E-03	0.4558E-05	0.1411E-04
9	-0.2259E-03	-0.2411E-03	-0.2740E-03	-0.2871E-03	0.4849E-05	0.1557E-04
10	-0.2501E-03	-0.2674E-03	-0.3031E-03	-0.3184E-03	0.5819E-05	0.1692E-04
11	-0.2810E-03	-0.2985E-03	-0.3411E-03	-0.3565E-03	0.6983E-05	0.2017E-04
12	-0.2602E-03	-0.2748E-03	-0.3169E-03	-0.3304E-03	0.6838E-05	0.1945E-04
13	-0.2129E-03	-0.2235E-03	-0.2607E-03	-0.2833E-03	0.6110E-05	0.1683E-04
14	-0.1639E-03	-0.1707E-03	-0.2023E-03	-0.2091E-03	0.4655E-05	0.1334E-04
15	-0.1181E-03	-0.1223E-03	-0.1483E-03	-0.1533E-03	0.1358E-05	0.7468E-05
16	-0.6261E-04	-0.6295E-04	-0.8132E-04	-0.8215E-04	0.4898E-05	0.9117E-05
17	-0.4558E-05	-0.1891E-05	-0.1096E-04	-0.8729E-05	0.1043E-04	0.1159E-04
18	-0.4791E-04	-0.4946E-04	-0.6382E-04	-0.6508E-04	0.1314E-04	0.1688E-04
19	-0.1017E-03	-0.1064E-03	-0.1283E-03	-0.1326E-03	0.1271E-04	0.1964E-04
20	-0.1490E-03	-0.1571E-03	-0.1844E-03	-0.1923E-03	0.1212E-04	0.2119E-04
21	-0.2040E-03	-0.2154E-03	-0.2498E-03	-0.2606E-03	0.1038E-04	0.2148E-04
22	-0.2525E-03	-0.2690E-03	-0.3088E-03	-0.3230E-03	0.1014E-04	0.2245E-04
23	-0.2775E-03	-0.2978E-03	-0.3392E-03	-0.3570E-03	0.1125E-04	0.2478E-04
24	-0.3097E-03	-0.3291E-03	-0.3777E-03	-0.3953E-03	0.9844E-05	0.2507E-04
25	-0.3370E-03	-0.3570E-03	-0.4113E-03	-0.4300E-03	0.1130E-04	0.2788E-04
26	-0.3569E-03	-0.3751E-03	-0.4378E-03	-0.4606E-03	0.6692E-05	0.3060E-04
27	-0.3955E-03	-0.4160E-03	-0.4827E-03	-0.5030E-03	0.1557E-04	0.3424E-04
28	-0.4229E-03	-0.4460E-03	-0.5184E-03	-0.5417E-03	0.2158E-04	0.4132E-04
29	-0.4488E-03	-0.4729E-03	-0.5497E-03	-0.5750E-03	0.2415E-04	0.4617E-04
30	-0.4788E-03	-0.5038E-03	-0.5844E-03	-0.6125E-03	0.2759E-04	0.5068E-04
31	-0.5087E-03	-0.5343E-03	-0.6167E-03	-0.6500E-03	0.3244E-04	0.5698E-04
32	-0.5334E-03	-0.5627E-03	-0.6400E-03	-0.6808E-03	0.3564E-04	0.6062E-04
33	-0.5685E-03	-0.5985E-03	-0.6734E-03	-0.7206E-03	0.3792E-04	0.6585E-04
34	-0.6070E-03	-0.6368E-03	-0.7084E-03	-0.7604E-03	0.3666E-04	0.6658E-04
35	-0.6387E-03	-0.6685E-03	-0.7323E-03	-0.7887E-03	0.3647E-04	0.6770E-04
36	-0.6729E-03	-0.7169E-03	-0.7569E-03	-0.8205E-03	0.3846E-04	0.7119E-04
37	-0.6845E-03	-0.7168E-03	-0.7643E-03	-0.8312E-03	0.3991E-04	0.7264E-04
38	-0.7117E-03	-0.7457E-03	-0.7852E-03	-0.8612E-03	0.4277E-04	0.7691E-04
39	-0.7313E-03	-0.7633E-03	-0.8004E-03	-0.8810E-03	0.4437E-04	0.8186E-04
40	-0.7492E-03	-0.7819E-03	-0.8125E-03	-0.9028E-03	0.4811E-04	0.8763E-04
41	-0.7712E-03	-0.8035E-03	-0.8285E-03	-0.9242E-03	0.5233E-04	0.9621E-04
42	-0.7916E-03	-0.8252E-03	-0.8420E-03	-0.9450E-03	0.5814E-04	0.1062E-03
43	-0.8061E-03	-0.8417E-03	-0.8490E-03	-0.9586E-03	0.6930E-04	0.1217E-03
44	-0.8231E-03	-0.8619E-03	-0.8600E-03	-0.9762E-03	0.8239E-04	0.1377E-03
45	-0.8406E-03	-0.8792E-03	-0.8744E-03	-0.9915E-03	0.1030E-03	0.1599E-03
46	-0.8511E-03	-0.8902E-03	-0.8837E-03	-0.1004E-02	0.1143E-03	0.1725E-03
47	-0.8572E-03	-0.8965E-03	-0.8906E-03	-0.1012E-02	0.1231E-03	0.1823E-03
48	-0.8722E-03	-0.9126E-03	-0.9104E-03	-0.1036E-02	0.1473E-03	0.2088E-03
49	-0.8794E-03	-0.9213E-03	-0.9183E-03	-0.1044E-02	0.1573E-03	0.2190E-03
50	-0.8948E-03	-0.9349E-03	-0.9379E-03	-0.1059E-02	0.1723E-03	0.2339E-03
51	-0.9025E-03	-0.9426E-03	-0.9518E-03	-0.1071E-02	0.1834E-03	0.2456E-03
52	-0.7767E-03	-0.8069E-03	-0.8040E-03	-0.9155E-03	0.1823E-03	0.2391E-03
53	-0.6443E-03	-0.6622E-03	-0.6452E-03	-0.7469E-03	0.1773E-03	0.2293E-03
54	-0.5176E-03	-0.5224E-03	-0.4927E-03	-0.5847E-03	0.1741E-03	0.2203E-03
55	-0.3745E-03	-0.3696E-03	-0.3242E-03	-0.4046E-03	0.1728E-03	0.2128E-03
56	-0.2271E-03	-0.2137E-03	-0.1485E-03	-0.2186E-03	0.1691E-03	0.2013E-03
57	-0.8292E-04	-0.6265E-04	0.2328E-04	-0.3647E-04	0.1689E-03	0.1944E-03
58	-0.8433E-04	-0.6367E-04	0.2284E-04	-0.3589E-04	0.1653E-03	0.1905E-03
59	-0.2180E-03	-0.2041E-03	-0.1369E-03	-0.2021E-03	0.1604E-03	0.1926E-03
60	-0.3498E-03	-0.3452E-03	-0.2959E-03	-0.3703E-03	0.1685E-03	0.2067E-03
61	-0.4811E-03	-0.4875E-03	-0.4539E-03	-0.5395E-03	0.1789E-03	0.2228E-03
62	-0.6238E-03	-0.6403E-03	-0.6237E-03	-0.7183E-03	0.1843E-03	0.2346E-03
63	-0.7606E-03	-0.7897E-03	-0.7875E-03	-0.8939E-03	0.1894E-03	0.2460E-03
64	-0.8216E-03	-0.8554E-03	-0.8622E-03	-0.9703E-03	0.1888E-03	0.2473E-03
65	-0.8696E-03	-0.9047E-03	-0.9159E-03	-0.1028E-02	0.1917E-03	0.2535E-03
66	-0.8900E-03	-0.9276E-03	-0.9404E-03	-0.1055E-02	0.1953E-03	0.2576E-03

	37	38	39	40	41	42
0	0.0	0.0	0.0	0.0	0.0	0.0
1	-0.1644E-04	-0.1794E-04	-0.3424E-04	-0.3380E-04	-0.1862E-04	-0.1857E-04
2	-0.3574E-04	-0.3841E-04	-0.7211E-04	-0.7129E-04	-0.3918E-04	-0.3860E-04
3	-0.5950E-04	-0.6261E-04	-0.1159E-03	-0.1145E-03	-0.6362E-04	-0.6285E-04
4	-0.8229E-04	-0.8598E-04	-0.1574E-03	-0.1559E-03	-0.8807E-04	-0.8637E-04
5	-0.1062E-03	-0.1108E-03	-0.2013E-03	-0.1993E-03	-0.1112E-03	-0.1100E-03
6	-0.1292E-03	-0.1347E-03	-0.2442E-03	-0.2423E-03	-0.1321E-03	-0.1315E-03
7	-0.1547E-03	-0.1604E-03	-0.2907E-03	-0.2881E-03	-0.1529E-03	-0.1536E-03
8	-0.1787E-03	-0.1855E-03	-0.3362E-03	-0.3330E-03	-0.1722E-03	-0.1749E-03
9	-0.2046E-03	-0.2111E-03	-0.3825E-03	-0.3796E-03	-0.1925E-03	-0.1974E-03
10	-0.2278E-03	-0.2351E-03	-0.4242E-03	-0.4211E-03	-0.2115E-03	-0.2178E-03
11	-0.2544E-03	-0.2627E-03	-0.4752E-03	-0.4707E-03	-0.2341E-03	-0.2399E-03
12	-0.2360E-03	-0.2437E-03	-0.4398E-03	-0.4350E-03	-0.2122E-03	-0.2187E-03
13	-0.1928E-03	-0.1994E-03	-0.3594E-03	-0.3549E-03	-0.1678E-03	-0.1746E-03
14	-0.1499E-03	-0.1545E-03	-0.2774E-03	-0.2736E-03	-0.1234E-03	-0.1308E-03
15	-0.1088E-03	-0.1134E-03	-0.2005E-03	-0.1972E-03	-0.8186E-04	-0.9020E-04
16	-0.5800E-04	-0.6105E-04	-0.1089E-03	-0.1065E-03	-0.3176E-04	-0.4156E-04
17	-0.5916E-05	-0.6692E-05	-0.1401E-04	-0.1261E-04	0.1848E-04	0.8341E-05
18	-0.4481E-04	-0.4786E-04	-0.9030E-04	-0.8845E-04	-0.2313E-04	-0.3283E-04
19	-0.9374E-04	-0.9776E-04	-0.1804E-03	-0.1777E-03	-0.7163E-04	-0.8108E-04
20	-0.1348E-03	-0.1406E-03	-0.2588E-03	-0.2556E-03	-0.1143E-03	-0.1224E-03
21	-0.1841E-03	-0.1910E-03	-0.3487E-03	-0.3443E-03	-0.1624E-03	-0.1695E-03
22	-0.2288E-03	-0.2371E-03	-0.4303E-03	-0.4256E-03	-0.2068E-03	-0.2132E-03
23	-0.2509E-03	-0.2603E-03	-0.4739E-03	-0.4686E-03	-0.2278E-03	-0.2351E-03
24	-0.2788E-03	-0.2898E-03	-0.5256E-03	-0.5204E-03	-0.2536E-03	-0.2610E-03
25	-0.3039E-03	-0.3154E-03	-0.5722E-03	-0.5662E-03	-0.2737E-03	-0.2822E-03
26	-0.3276E-03	-0.3407E-03	-0.6190E-03	-0.6131E-03	-0.2940E-03	-0.3042E-03
27	-0.3560E-03	-0.3700E-03	-0.6724E-03	-0.6667E-03	-0.3142E-03	-0.3256E-03
28	-0.3833E-03	-0.3982E-03	-0.7295E-03	-0.7282E-03	-0.3310E-03	-0.3431E-03
29	-0.4100E-03	-0.4270E-03	-0.7881E-03	-0.8071E-03	-0.3488E-03	-0.3621E-03
30	-0.4465E-03	-0.4634E-03	-0.8688E-03	-0.9008E-03	-0.3709E-03	-0.3848E-03
31	-0.4868E-03	-0.5057E-03	-0.9715E-03	-0.9943E-03	-0.3933E-03	-0.4071E-03
32	-0.5312E-03	-0.5528E-03	-0.1084E-02	-0.1097E-02	-0.4125E-03	-0.4286E-03
33	-0.5938E-03	-0.6166E-03	-0.1211E-02	-0.1242E-02	-0.4412E-03	-0.4623E-03
34	-0.6691E-03	-0.6942E-03	-0.1385E-02	-0.1411E-02	-0.4782E-03	-0.4992E-03
35	-0.7476E-03	-0.7769E-03	-0.1519E-02	-0.1577E-02	-0.5121E-03	-0.5335E-03
36	-0.8541E-03	-0.8897E-03	-0.1612E-02	-0.1852E-02	-0.5541E-03	-0.5704E-03
37	-0.9101E-03	-0.9468E-03	-0.1648E-02	-0.1982E-02	-0.5709E-03	-0.5848E-03
38	-0.9867E-03	-0.1024E-02	-0.1728E-02	-0.2130E-02	-0.5959E-03	-0.6105E-03
39	-0.1064E-02	-0.1101E-02	-0.1851E-02	-0.2241E-02	-0.6179E-03	-0.6328E-03
40	-0.1147E-02	-0.1185E-02	-0.2116E-02	-0.2341E-02	-0.6394E-03	-0.6550E-03
41	-0.1239E-02	-0.1276E-02	-0.2462E-02	-0.2443E-02	-0.6666E-03	-0.6841E-03
42	-0.1343E-02	-0.1379E-02	-0.2951E-02	-0.2538E-02	-0.6915E-03	-0.7094E-03
43	-0.1459E-02	-0.1492E-02	-0.3491E-02	-0.2621E-02	-0.7098E-03	-0.7300E-03
44	-0.1580E-02	-0.1616E-02	-0.3844E-02	-0.2701E-02	-0.7316E-03	-0.7543E-03
45	-0.1724E-02	-0.1761E-02	-0.4366E-02	-0.2776E-02	-0.7528E-03	-0.7776E-03
46	-0.1800E-02	-0.1838E-02	-0.4628E-02	-0.2880E-02	-0.7654E-03	-0.7914E-03
47	-0.1864E-02	-0.1904E-02	-0.4782E-02	-0.3045E-02	-0.7750E-03	-0.8027E-03
48	-0.1960E-02	-0.2009E-02	-0.4968E-02	-0.3592E-02	-0.7941E-03	-0.8240E-03
49	-0.2027E-02	-0.2078E-02	-0.5081E-02	-0.4183E-02	-0.8053E-03	-0.8393E-03
50	-0.2120E-02	-0.2170E-02	-0.5285E-02	-0.4650E-02	-0.8251E-03	-0.8611E-03
51	-0.2191E-02	-0.2218E-02	-0.5450E-02	-0.4874E-02	-0.8397E-03	-0.8755E-03
52	-0.2103E-02	-0.2130E-02	-0.5314E-02	-0.4743E-02	-0.7284E-03	-0.7659E-03
53	-0.1982E-02	-0.2003E-02	-0.5081E-02	-0.4512E-02	-0.6116E-03	-0.6490E-03
54	-0.1862E-02	-0.1878E-02	-0.4847E-02	-0.4282E-02	-0.5003E-03	-0.5384E-03
55	-0.1719E-02	-0.1731E-02	-0.4621E-02	-0.4012E-02	-0.3795E-03	-0.4184E-03
56	-0.1574E-02	-0.1581E-02	-0.4288E-02	-0.3737E-02	-0.2604E-03	-0.2998E-03
57	-0.1424E-02	-0.1426E-02	-0.3992E-02	-0.3452E-02	-0.1399E-03	-0.1806E-03
58	-0.1424E-02	-0.1425E-02	-0.3989E-02	-0.3449E-02	-0.1437E-03	-0.1842E-03
59	-0.1543E-02	-0.1550E-02	-0.4212E-02	-0.3668E-02	-0.2592E-03	-0.2983E-03
60	-0.1657E-02	-0.1669E-02	-0.4428E-02	-0.3884E-02	-0.3665E-03	-0.4055E-03
61	-0.1780E-02	-0.1796E-02	-0.4671E-02	-0.4123E-02	-0.4767E-03	-0.5162E-03
62	-0.1921E-02	-0.1942E-02	-0.4946E-02	-0.4393E-02	-0.5969E-03	-0.6347E-03
63	-0.2056E-02	-0.2083E-02	-0.5215E-02	-0.4655E-02	-0.7122E-03	-0.7492E-03
64	-0.2107E-02	-0.2134E-02	-0.5298E-02	-0.4741E-02	-0.7635E-03	-0.7999E-03
65	-0.2176E-02	-0.2205E-02	-0.5451E-02	-0.4880E-02	-0.8030E-03	-0.8404E-03
66	-0.2205E-02	-0.2237E-02	-0.5524E-02	-0.4948E-02	-0.8192E-03	-0.8563E-03

	43	44	45	46	47	48
0	0.0	0.0	0.0	0.0	0.0	0.0
1	-0.2173E-04	-0.2226E-04	-0.3026E-04	-0.3055E-04	-0.2265E-04	-0.1620E-04
2	-0.4689E-04	-0.4898E-04	-0.6469E-04	-0.6459E-04	-0.4820E-04	-0.3438E-04
3	-0.7551E-04	-0.7987E-04	-0.1045E-03	-0.1048E-03	-0.7992E-04	-0.5591E-04
4	-0.1027E-03	-0.1092E-03	-0.1430E-03	-0.1432E-03	-0.1093E-03	-0.7657E-04
5	-0.1321E-03	-0.1394E-03	-0.1826E-03	-0.1836E-03	-0.1405E-03	-0.9728E-04
6	-0.1592E-03	-0.1689E-03	-0.2219E-03	-0.2241E-03	-0.1706E-03	-0.1163E-03
7	-0.1888E-03	-0.2001E-03	-0.2638E-03	-0.2634E-03	-0.1995E-03	-0.1333E-03
8	-0.2175E-03	-0.2304E-03	-0.3033E-03	-0.3056E-03	-0.2282E-03	-0.1513E-03
9	-0.2483E-03	-0.2617E-03	-0.3464E-03	-0.3469E-03	-0.2591E-03	-0.1703E-03
10	-0.2751E-03	-0.2894E-03	-0.3829E-03	-0.3879E-03	-0.2856E-03	-0.1875E-03
11	-0.3073E-03	-0.3226E-03	-0.4285E-03	-0.4306E-03	-0.3174E-03	-0.2068E-03
12	-0.2844E-03	-0.2983E-03	-0.3966E-03	-0.3950E-03	-0.2905E-03	-0.1851E-03
13	-0.2322E-03	-0.2435E-03	-0.3245E-03	-0.3225E-03	-0.2325E-03	-0.1427E-03
14	-0.1797E-03	-0.1863E-03	-0.2505E-03	-0.2484E-03	-0.1754E-03	-0.1018E-03
15	-0.1299E-03	-0.1344E-03	-0.1807E-03	-0.1771E-03	-0.1226E-03	-0.6513E-04
16	-0.6993E-04	-0.7061E-04	-0.9675E-04	-0.9325E-04	-0.5844E-04	-0.2255E-04
17	-0.8389E-05	-0.5092E-05	-0.1115E-04	-0.5965E-05	0.6838E-05	0.1964E-04
18	-0.5713E-04	-0.5717E-04	-0.8002E-04	-0.7628E-04	-0.4733E-04	-0.1712E-04
19	-0.1160E-03	-0.1196E-03	-0.1584E-03	-0.1571E-03	-0.1034E-03	-0.5853E-04
20	-0.1668E-03	-0.1739E-03	-0.2329E-03	-0.2298E-03	-0.1613E-03	-0.9752E-04
21	-0.2243E-03	-0.2350E-03	-0.3145E-03	-0.3108E-03	-0.2228E-03	-0.1400E-03
22	-0.2774E-03	-0.2913E-03	-0.3888E-03	-0.3888E-03	-0.2817E-03	-0.1804E-03
23	-0.3045E-03	-0.3217E-03	-0.4270E-03	-0.4282E-03	-0.3121E-03	-0.2000E-03
24	-0.3381E-03	-0.3556E-03	-0.4734E-03	-0.4707E-03	-0.3455E-03	-0.2220E-03
25	-0.3669E-03	-0.3855E-03	-0.5135E-03	-0.5115E-03	-0.3740E-03	-0.2398E-03
26	-0.3980E-03	-0.4163E-03	-0.5568E-03	-0.5545E-03	-0.4029E-03	-0.2588E-03
27	-0.4293E-03	-0.4494E-03	-0.6020E-03	-0.5979E-03	-0.4346E-03	-0.2785E-03
28	-0.4580E-03	-0.4802E-03	-0.6452E-03	-0.6393E-03	-0.4660E-03	-0.2946E-03
29	-0.4877E-03	-0.5102E-03	-0.6879E-03	-0.6850E-03	-0.4935E-03	-0.3129E-03
30	-0.5217E-03	-0.5472E-03	-0.7385E-03	-0.7285E-03	-0.5295E-03	-0.3334E-03
31	-0.5563E-03	-0.5831E-03	-0.7919E-03	-0.7767E-03	-0.5643E-03	-0.3541E-03
32	-0.5874E-03	-0.6182E-03	-0.8402E-03	-0.8264E-03	-0.6000E-03	-0.3753E-03
33	-0.6307E-03	-0.6627E-03	-0.9034E-03	-0.8791E-03	-0.6415E-03	-0.4021E-03
34	-0.6809E-03	-0.7131E-03	-0.9783E-03	-0.9300E-03	-0.6909E-03	-0.4347E-03
35	-0.7252E-03	-0.7577E-03	-0.1043E-02	-0.9741E-03	-0.7315E-03	-0.4645E-03
36	-0.7766E-03	-0.8089E-03	-0.1114E-02	-0.1018E-02	-0.7811E-03	-0.4997E-03
37	-0.7966E-03	-0.8294E-03	-0.1142E-02	-0.1040E-02	-0.8036E-03	-0.5169E-03
38	-0.8286E-03	-0.8630E-03	-0.1186E-02	-0.1062E-02	-0.8377E-03	-0.5411E-03
39	-0.8557E-03	-0.8884E-03	-0.1221E-02	-0.1084E-02	-0.8643E-03	-0.5615E-03
40	-0.8818E-03	-0.9151E-03	-0.1254E-02	-0.1106E-02	-0.8921E-03	-0.5830E-03
41	-0.9123E-03	-0.9455E-03	-0.1289E-02	-0.1128E-02	-0.9276E-03	-0.6094E-03
42	-0.9403E-03	-0.9741E-03	-0.1321E-02	-0.1153E-02	-0.9611E-03	-0.6350E-03
43	-0.9632E-03	-0.9974E-03	-0.1345E-02	-0.1176E-02	-0.9902E-03	-0.6572E-03
44	-0.9888E-03	-0.1025E-02	-0.1376E-02	-0.1202E-02	-0.1023E-02	-0.6809E-03
45	-0.1013E-02	-0.1050E-02	-0.1404E-02	-0.1229E-02	-0.1055E-02	-0.7036E-03
46	-0.1029E-02	-0.1067E-02	-0.1421E-02	-0.1243E-02	-0.1075E-02	-0.7187E-03
47	-0.1038E-02	-0.1087E-02	-0.1430E-02	-0.1258E-02	-0.1087E-02	-0.7290E-03
48	-0.1060E-02	-0.1098E-02	-0.1455E-02	-0.1271E-02	-0.1116E-02	-0.7500E-03
49	-0.1071E-02	-0.1110E-02	-0.1464E-02	-0.1283E-02	-0.1136E-02	-0.7632E-03
50	-0.1090E-02	-0.1129E-02	-0.1484E-02	-0.1301E-02	-0.1169E-02	-0.7844E-03
51	-0.1102E-02	-0.1141E-02	-0.1494E-02	-0.1314E-02	-0.1193E-02	-0.7992E-03
52	-0.9674E-03	-0.9985E-03	-0.1304E-02	-0.1121E-02	-0.1040E-02	-0.6762E-03
53	-0.8254E-03	-0.8482E-03	-0.1103E-02	-0.9196E-03	-0.8796E-03	-0.5511E-03
54	-0.6910E-03	-0.7039E-03	-0.9150E-03	-0.7275E-03	-0.7285E-03	-0.4391E-03
55	-0.5417E-03	-0.5465E-03	-0.7043E-03	-0.5192E-03	-0.5649E-03	-0.3299E-03
56	-0.3889E-03	-0.3824E-03	-0.4841E-03	-0.2969E-03	-0.3961E-03	-0.2329E-03
57	-0.2347E-03	-0.2205E-03	-0.2666E-03	-0.7783E-04	-0.2343E-03	-0.1415E-03
58	-0.2360E-03	-0.2209E-03	-0.2657E-03	-0.7730E-04	-0.2382E-03	-0.1518E-03
59	-0.3780E-03	-0.3716E-03	-0.4667E-03	-0.2817E-03	-0.3889E-03	-0.2455E-03
60	-0.5157E-03	-0.5191E-03	-0.6634E-03	-0.4838E-03	-0.5356E-03	-0.3351E-03
61	-0.6540E-03	-0.6673E-03	-0.8614E-03	-0.6853E-03	-0.6842E-03	-0.4373E-03
62	-0.8033E-03	-0.8243E-03	-0.1074E-02	-0.8952E-03	-0.8486E-03	-0.5553E-03
63	-0.9481E-03	-0.9797E-03	-0.1281E-02	-0.1103E-02	-0.1011E-02	-0.6755E-03
64	-0.1015E-02	-0.1047E-02	-0.1374E-02	-0.1213E-02	-0.1082E-02	-0.7302E-03
65	-0.1066E-02	-0.1101E-02	-0.1448E-02	-0.1267E-02	-0.1137E-02	-0.7670E-03
66	-0.1087E-02	-0.1124E-02	-0.1479E-02	-0.1300E-02	-0.1160E-02	-0.7821E-03

	49	50	51	52	53	54
0	0.0	0.0	0.0	0.0	0.0	0.0
1	-0.2740E-04	-0.2784E-04	-0.3559E-04	-0.3487E-04	0.0	-0.5219E-03
2	-0.5674E-04	-0.5936E-04	-0.7390E-04	-0.7502E-04	0.0	-0.2262E-02
3	-0.9199E-04	-0.9665E-04	-0.1196E-03	-0.1208E-03	0.0	-0.2932E-02
4	-0.1256E-03	-0.1322E-03	-0.1627E-03	-0.1657E-03	0.0	-0.4026E-02
5	-0.1602E-03	-0.1684E-03	-0.2071E-03	-0.2126E-03	0.0	-0.4622E-02
6	-0.1951E-03	-0.2054E-03	-0.2528E-03	-0.2599E-03	0.0	0.8127E-02
7	-0.2292E-03	-0.2412E-03	-0.2978E-03	-0.3058E-03	0.0	0.7629E-02
8	-0.2640E-03	-0.2775E-03	-0.3435E-03	-0.3533E-03	0.0	0.6287E-02
9	-0.3005E-03	-0.3147E-03	-0.3912E-03	-0.4027E-03	0.0	0.5020E-02
10	-0.3326E-03	-0.3482E-03	-0.4334E-03	-0.4518E-03	0.0	0.4175E-02
11	-0.3711E-03	-0.3873E-03	-0.4854E-03	-0.5005E-03	0.0	0.1284E-01
12	-0.3424E-03	-0.3568E-03	-0.4495E-03	-0.4600E-03	0.0	0.9468E-02
13	-0.2777E-03	-0.2884E-03	-0.3646E-03	-0.3762E-03	0.0	0.8400E-02
14	-0.2128E-03	-0.2197E-03	-0.2801E-03	-0.2898E-03	0.0	0.7971E-02
15	-0.1517E-03	-0.1557E-03	-0.2013E-03	-0.2061E-03	0.0	0.7704E-02
16	-0.7958E-04	-0.7938E-04	-0.1079E-03	-0.1098E-03	0.0	0.7306E-02
17	-0.5431E-05	-0.5335E-06	-0.1174E-04	-0.1125E-04	0.0	0.7033E-02
18	-0.6556E-04	-0.6459E-04	-0.9025E-04	-0.9190E-04	0.0	0.6555E-02
19	-0.1341E-03	-0.1381E-03	-0.1812E-03	-0.1851E-03	0.0	0.6362E-02
20	-0.1988E-03	-0.2058E-03	-0.2628E-03	-0.2698E-03	0.0	0.5933E-02
21	-0.2691E-03	-0.2801E-03	-0.3538E-03	-0.3628E-03	0.0	0.4790E-02
22	-0.3341E-03	-0.3491E-03	-0.4383E-03	-0.4543E-03	0.0	0.4598E-02
23	-0.3678E-03	-0.3861E-03	-0.4842E-03	-0.5006E-03	0.0	0.4467E-02
24	-0.4086E-03	-0.4268E-03	-0.5366E-03	-0.5496E-03	0.0	0.4293E-02
25	-0.4428E-03	-0.4615E-03	-0.5816E-03	-0.5972E-03	0.0	0.4144E-02
26	-0.4795E-03	-0.4985E-03	-0.6303E-03	-0.6475E-03	0.0	0.3976E-02
27	-0.5175E-03	-0.5383E-03	-0.6789E-03	-0.6998E-03	0.0	0.3852E-02
28	-0.5540E-03	-0.5779E-03	-0.7320E-03	-0.7605E-03	0.0	0.3529E-02
29	-0.5898E-03	-0.6164E-03	-0.7854E-03	-0.8378E-03	0.0	0.3579E-02
30	-0.6360E-03	-0.6650E-03	-0.8583E-03	-0.9273E-03	0.0	0.3206E-02
31	-0.6842E-03	-0.7161E-03	-0.9412E-03	-0.1022E-02	0.0	0.3032E-02
32	-0.7352E-03	-0.7723E-03	-0.1032E-02	-0.1141E-02	0.0	0.2982E-02
33	-0.8010E-03	-0.8437E-03	-0.1150E-02	-0.1301E-02	0.0	0.7412E-02
34	-0.8827E-03	-0.9292E-03	-0.1283E-02	-0.1500E-02	0.0	0.3032E-02
35	-0.9608E-03	-0.1015E-02	-0.1401E-02	-0.1749E-02	0.0	0.2609E-02
36	-0.1066E-02	-0.1144E-02	-0.1584E-02	-0.2082E-02	0.0	0.2380E-02
37	-0.1123E-02	-0.1208E-02	-0.1646E-02	-0.2248E-02	0.0	0.3032E-02
38	-0.1195E-02	-0.1290E-02	-0.1716E-02	-0.2440E-02	0.0	0.2212E-02
39	-0.1262E-02	-0.1363E-02	-0.1782E-02	-0.2648E-02	0.0	0.2088E-02
40	-0.1336E-02	-0.1449E-02	-0.1840E-02	-0.2872E-02	0.0	0.2112E-02
41	-0.1430E-02	-0.1563E-02	-0.1884E-02	-0.3094E-02	0.0	0.2088E-02
42	-0.1527E-02	-0.1689E-02	-0.1903E-02	-0.3488E-02	0.0	0.1814E-02
43	-0.1626E-02	-0.1819E-02	-0.1907E-02	-0.4033E-02	0.0	0.1764E-02
44	-0.1727E-02	-0.1952E-02	-0.1926E-02	-0.4406E-02	0.0	0.1640E-02
45	-0.1852E-02	-0.2111E-02	-0.1955E-02	-0.4698E-02	0.0	0.1591E-02
46	-0.1937E-02	-0.2218E-02	-0.1984E-02	-0.4816E-02	0.0	0.3467E-02
47	-0.2012E-02	-0.2317E-02	-0.2009E-02	-0.4927E-02	0.0	0.2411E-02
48	-0.2136E-02	-0.2485E-02	-0.2071E-02	-0.5017E-02	0.0	0.9692E-03
49	-0.2258E-02	-0.2658E-02	-0.2129E-02	-0.5092E-02	0.0	0.8450E-03
50	-0.2461E-02	-0.3027E-02	-0.2367E-02	-0.5188E-02	0.0	0.8698E-03
51	-0.2488E-02	-0.3373E-02	-0.2730E-02	-0.5222E-02	0.0	0.6461E-03
52	-0.2314E-02	-0.3365E-02	-0.2652E-02	-0.4998E-02	0.0	0.1417E-02
53	-0.2130E-02	-0.3174E-02	-0.2416E-02	-0.4736E-02	0.0	0.1646E-02
54	-0.1953E-02	-0.2987E-02	-0.2187E-02	-0.4494E-02	0.0	0.7828E-03
55	-0.1756E-02	-0.2777E-02	-0.1933E-02	-0.4228E-02	0.0	0.5219E-03
56	-0.1547E-02	-0.2554E-02	-0.1651E-02	-0.3925E-02	0.0	0.6089E-03
57	-0.1337E-02	-0.2330E-02	-0.1370E-02	-0.3626E-02	0.0	0.4225E-03
58	-0.1336E-02	-0.2329E-02	-0.1365E-02	-0.3615E-02	0.0	0.1491E-03
59	-0.1512E-02	-0.2511E-02	-0.1591E-02	-0.3847E-02	0.0	-0.3231E-03
60	-0.1681E-02	-0.2686E-02	-0.1816E-02	-0.4086E-02	0.0	0.1491E-03
61	-0.1863E-02	-0.2877E-02	-0.2055E-02	-0.4334E-02	0.0	0.9941E-04
62	-0.2064E-02	-0.3090E-02	-0.2316E-02	-0.4602E-02	0.0	-0.2299E-03
63	-0.2262E-02	-0.3299E-02	-0.2577E-02	-0.4894E-02	0.0	-0.1988E-03
64	-0.2343E-02	-0.3373E-02	-0.2692E-02	-0.5048E-02	0.0	0.4970E-03
65	-0.2431E-02	-0.3478E-02	-0.2811E-02	-0.5135E-02	0.0	-0.2920E-03
66	-0.2470E-02	-0.3531E-02	-0.2883E-02	-0.5191E-02	0.0	-0.9195E-03

	55	56	57	58	59
0	0.0	0.0	0.0	0.0	0.0
1	-0.9604E-04	0.6999E-05	0.5771E-05	0.1875E-01	0.7080E-04
2	0.2650E-01	0.2489E-04	0.2333E-04	0.1995E-01	0.9805E-04
3	0.2552E-01	0.2139E-04	0.3981E-04	0.2019E-01	0.2171E-02
4	0.2218E-01	0.6610E-05	0.7395E-04	0.2048E-01	0.1127E-03
5	0.5737E-02	0.1205E-04	0.5097E-04	0.2076E-01	0.4044E-04
6	0.2982E-01	0.5055E-05	0.5136E-04	0.2052E-01	0.1736E-04
7	0.2310E-02	-0.4277E-05	0.7095E-04	0.2041E-01	0.2206E-04
8	0.2817E-01	0.5832E-05	0.1021E-03	0.1987E-01	0.7584E-04
9	0.1272E-02	-0.8943E-05	0.7546E-04	0.1918E-01	0.7240E-04
10	0.7038E-03	0.1905E-04	0.7885E-04	0.1807E-01	0.2725E-04
11	-0.1136E-01	-0.5832E-05	0.8637E-04	-0.3387E-02	0.2910E-04
12	0.9163E-02	-0.3888E-05	0.7788E-04	-0.2393E-02	0.2100E-04
13	0.1113E-01	-0.8943E-05	0.6164E-04	-0.1610E-02	0.1154E-04
14	-0.1268E-01	-0.3227E-04	0.4306E-04	-0.9807E-03	0.1152E-03
15	0.9090E-02	-0.4083E-04	0.2740E-04	-0.9648E-03	0.8879E-04
16	-0.2985E-02	-0.5560E-04	0.1086E-04	-0.7309E-03	0.1151E-03
17	-0.1351E-01	-0.6610E-04	-0.4946E-05	0.3421E-03	0.2005E-01
18	0.1125E-02	-0.4472E-04	0.2662E-04	0.2108E-01	0.7662E-03
19	0.1962E-01	-0.3655E-04	0.4161E-04	0.2129E-01	0.2188E-01
20	0.7507E-02	-0.3850E-04	0.6159E-04	0.2127E-01	0.4135E-03
21	0.1887E-01	-0.2877E-04	0.6721E-04	0.2037E-01	0.2593E-01
22	0.1735E-01	-0.2644E-04	0.9243E-04	0.1853E-01	0.2773E-01
23	0.3594E-01	-0.3655E-04	0.9015E-04	-0.2839E-02	0.2839E-01
24	0.2662E-01	-0.6105E-04	0.9272E-04	-0.1809E-03	0.2900E-01
25	0.3338E-01	-0.3811E-04	0.1261E-03	-0.1572E-02	0.2913E-01
26	0.1406E-01	-0.2333E-04	0.1237E-03	-0.5489E-03	0.2859E-01
27	0.1591E-01	-0.3111E-04	0.1613E-03	-0.2177E-04	0.2828E-01
28	0.1733E-01	-0.4472E-04	0.2340E-01	0.2109E-01	0.7332E-04
29	0.2533E-01	-0.2955E-04	0.2375E-01	0.2257E-01	0.1428E-03
30	0.1741E-01	-0.5094E-04	0.2384E-01	0.2305E-01	0.1523E-03
31	0.4539E-01	-0.3072E-04	0.2393E-01	0.2294E-01	0.2666E-03
32	0.2186E-01	-0.3189E-04	0.2386E-01	0.2282E-01	0.4238E-03
33	0.3084E-01	-0.4394E-04	0.2378E-01	0.2272E-01	0.2107E-03
34	0.2779E-02	-0.8593E-04	0.2371E-01	0.2257E-01	0.9699E-04
35	0.2037E-01	-0.6338E-04	0.2362E-01	0.2201E-01	0.9117E-04
36	0.3444E-01	-0.4783E-04	0.2351E-01	0.2016E-01	0.2211E-03
37	0.3566E-01	-0.4627E-04	0.2341E-01	0.1845E-01	0.8637E-04
38	0.4265E-01	-0.6494E-04	0.2329E-01	0.1661E-01	0.3902E-03
39	0.4258E-01	-0.8399E-04	0.2315E-01	-0.3143E-02	0.6857E-02
40	0.4132E-01	-0.9760E-04	0.2303E-01	-0.1581E-02	0.3298E-04
41	0.1902E-01	-0.8749E-04	0.2298E-01	-0.9898E-03	0.3878E-03
42	0.3838E-01	-0.1058E-03	0.2280E-01	0.1193E-03	0.2377E-03
43	0.2154E-01	-0.1100E-03	0.2220E-01	-0.2082E-03	0.4001E-04
44	0.1941E-01	-0.1361E-03	0.2190E-01	-0.6239E-03	0.2454E-04
45	0.1865E-01	-0.1489E-03	0.2182E-01	-0.4726E-03	0.4267E-04
46	0.3466E-01	-0.1555E-03	0.2173E-01	0.1782E-03	0.9123E-02
47	0.3057E-01	-0.1520E-03	0.2173E-01	-0.5791E-03	0.8331E-04
48	0.3330E-01	-0.1913E-03	0.2180E-01	0.2896E-04	0.3554E-03
49	0.3739E-01	-0.1944E-03	0.2183E-01	0.1069E-02	0.5598E-02
50	0.3472E-01	-0.2216E-03	0.2187E-01	0.5401E-03	0.8031E-04
51	0.3516E-01	-0.2228E-03	0.2189E-01	0.1487E-02	0.1880E-02
52	0.3756E-01	-0.2551E-03	0.2188E-01	0.2221E-01	0.4366E-02
53	0.3762E-01	-0.2702E-03	0.2186E-01	0.2341E-01	0.4148E-02
54	0.3761E-01	-0.2846E-03	0.2185E-01	0.2427E-01	0.1706E-02
55	0.3765E-01	-0.3060E-03	0.2183E-01	0.2482E-01	0.1229E-03
56	0.3673E-01	-0.3185E-03	0.2164E-01	0.2525E-01	-0.5684E-04
57	0.3634E-01	-0.3492E-03	0.2161E-01	0.2548E-01	0.8389E-05
58	0.4540E-01	-0.3601E-03	0.2168E-01	0.2579E-01	0.3693E-03
59	0.1891E-01	-0.3200E-03	0.2174E-01	0.2585E-01	0.5998E-03
60	0.1799E-01	-0.2944E-03	0.2178E-01	0.2590E-01	0.6180E-03
61	0.4472E-01	-0.2905E-03	0.2182E-01	0.2595E-01	0.1790E-01
62	0.3972E-01	-0.3041E-03	0.2187E-01	0.2599E-01	0.8491E-04
63	0.1942E-01	-0.2438E-03	0.2191E-01	0.2604E-01	0.2197E-01
64	0.1769E-01	-0.1754E-03	0.2193E-01	0.2606E-01	0.2713E-01
65	0.4687E-01	-0.2073E-03	0.2194E-01	0.2607E-01	0.2882E-01
66	0.4848E-01	-0.2205E-03	0.2195E-01	0.2606E-01	0.2927E-01

FIGURES

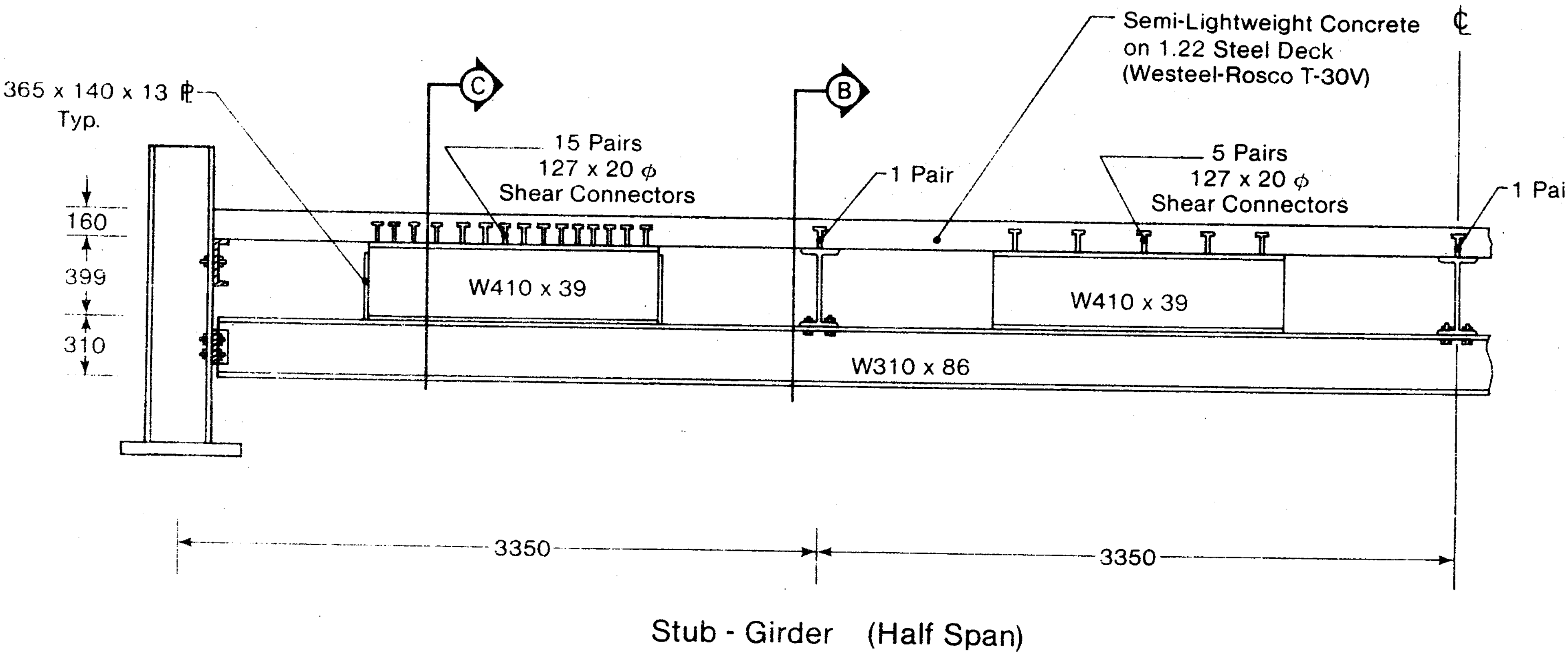


Figure 1 Elevation of Full-Size Stub-Girder

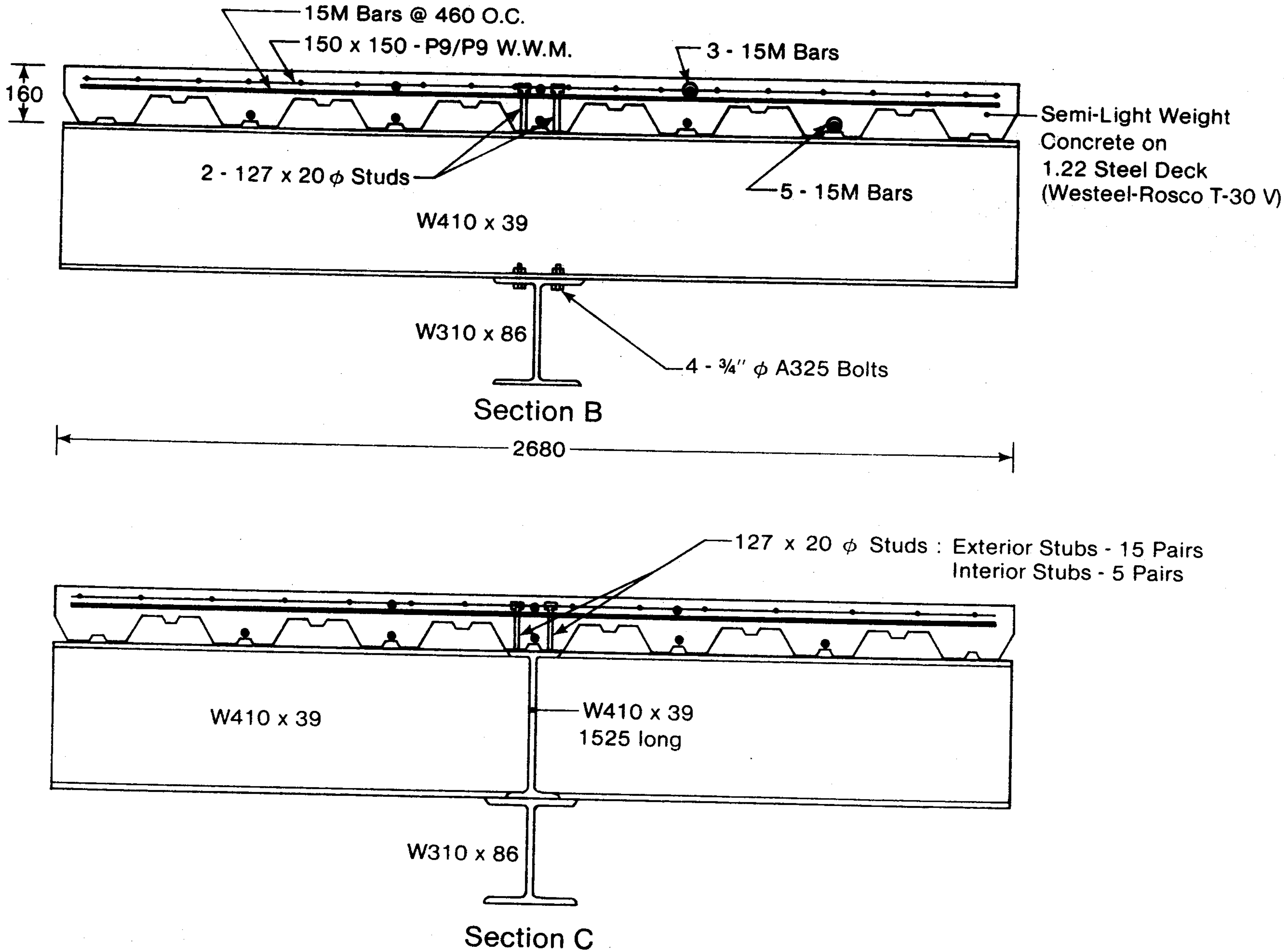


Figure 2 Sections B and C of the Full-Size Stub-Girder

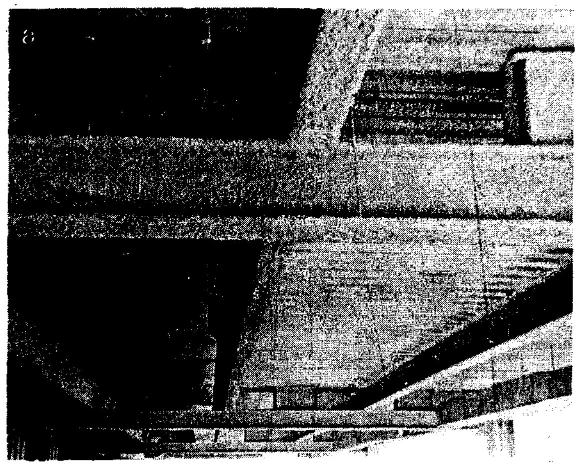


Figure 3 Stub-Girder Floor System Prior to Installation of Ductwork (Sprayed-On Fire Protection Has Been Applied)

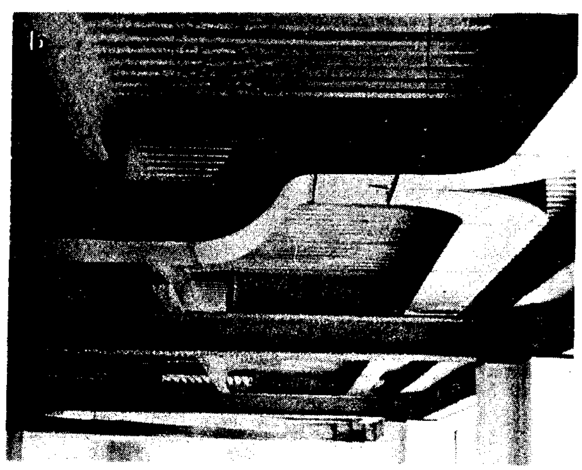


Figure 4 Stub-Girder Floor System After Installation of Ductwork

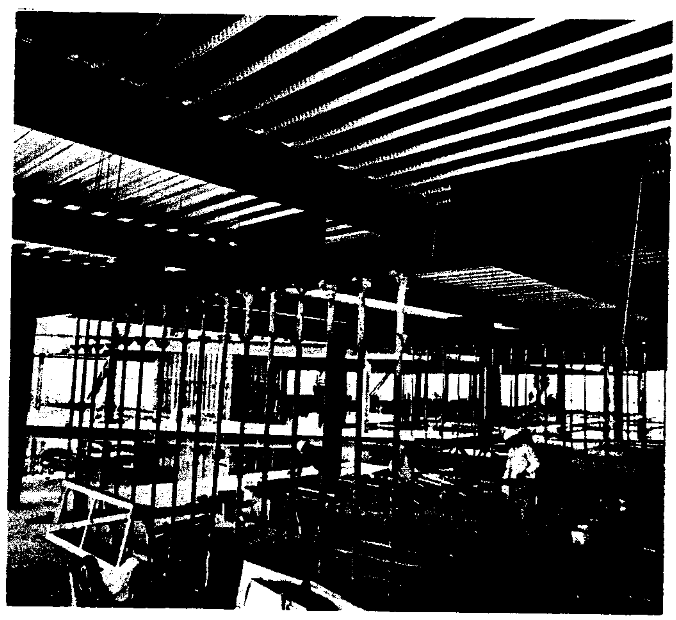


Figure 5 View of Stub-Girder Floor System in the Construction Phase

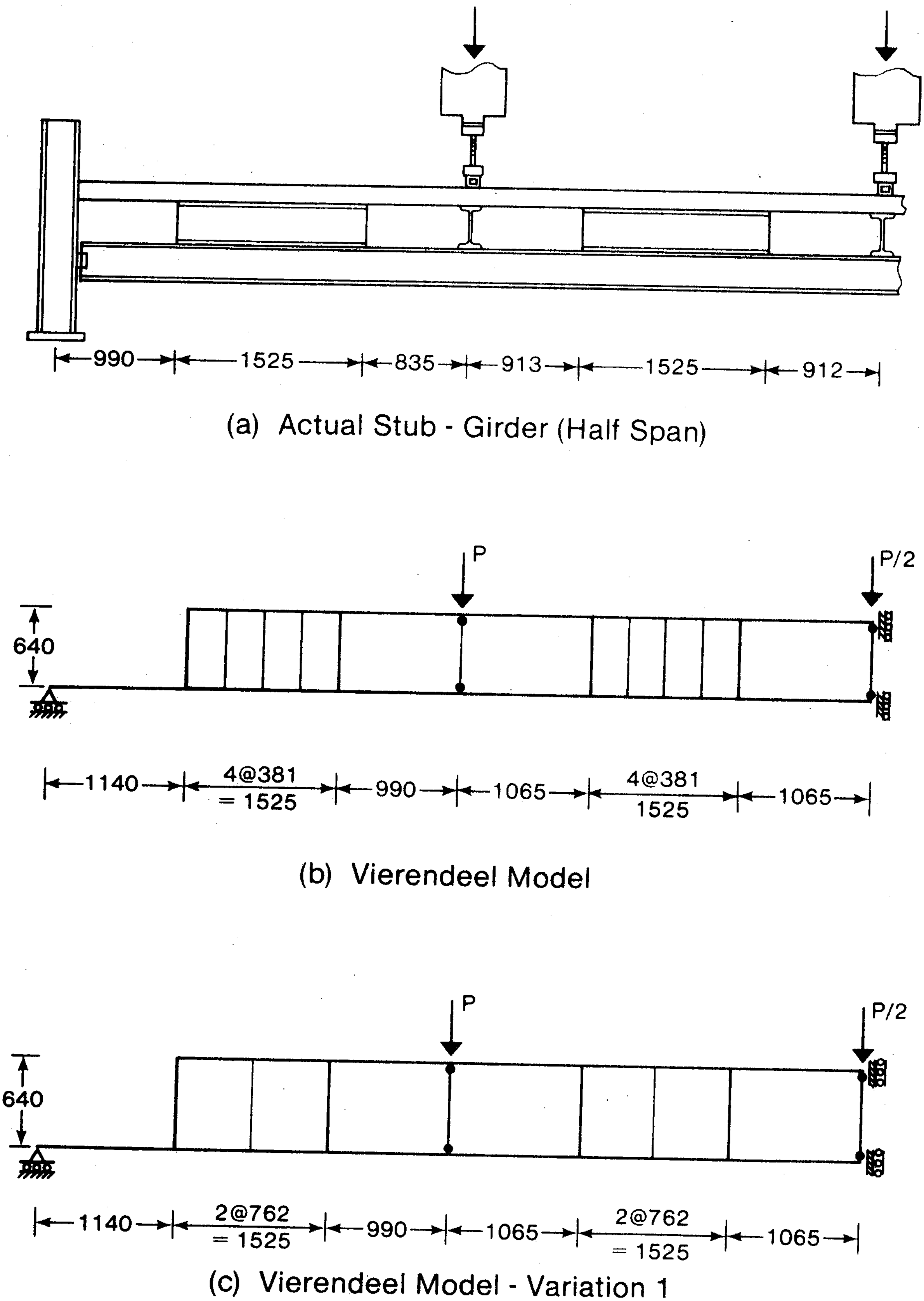
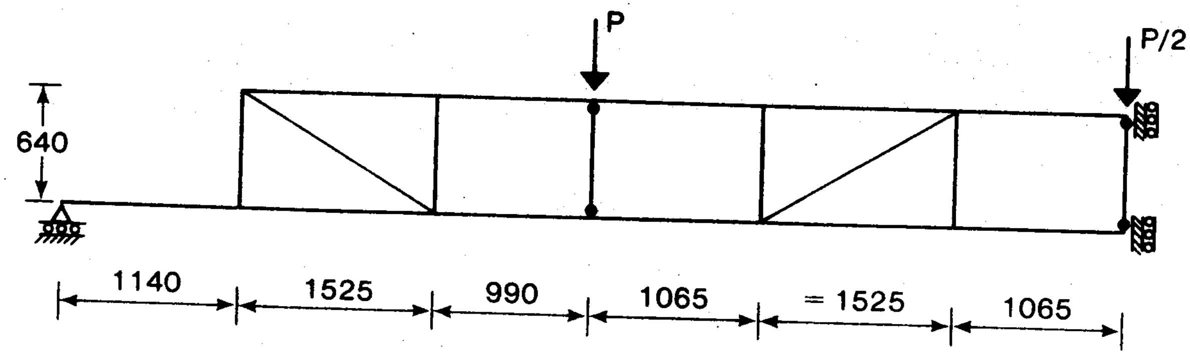
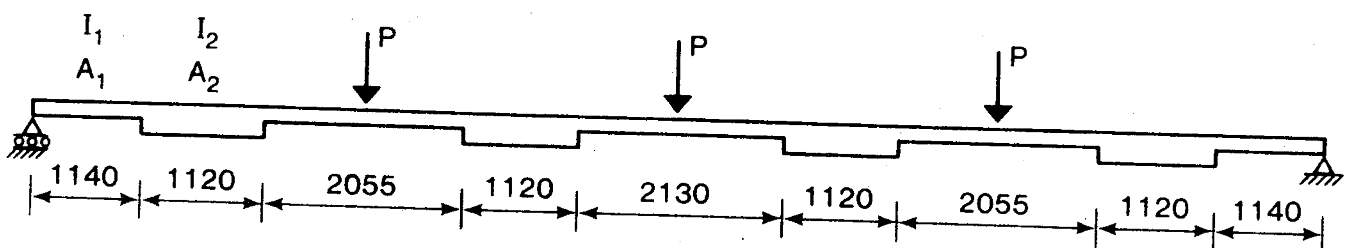


Figure 6 Stub-Girder Modeling Schemes



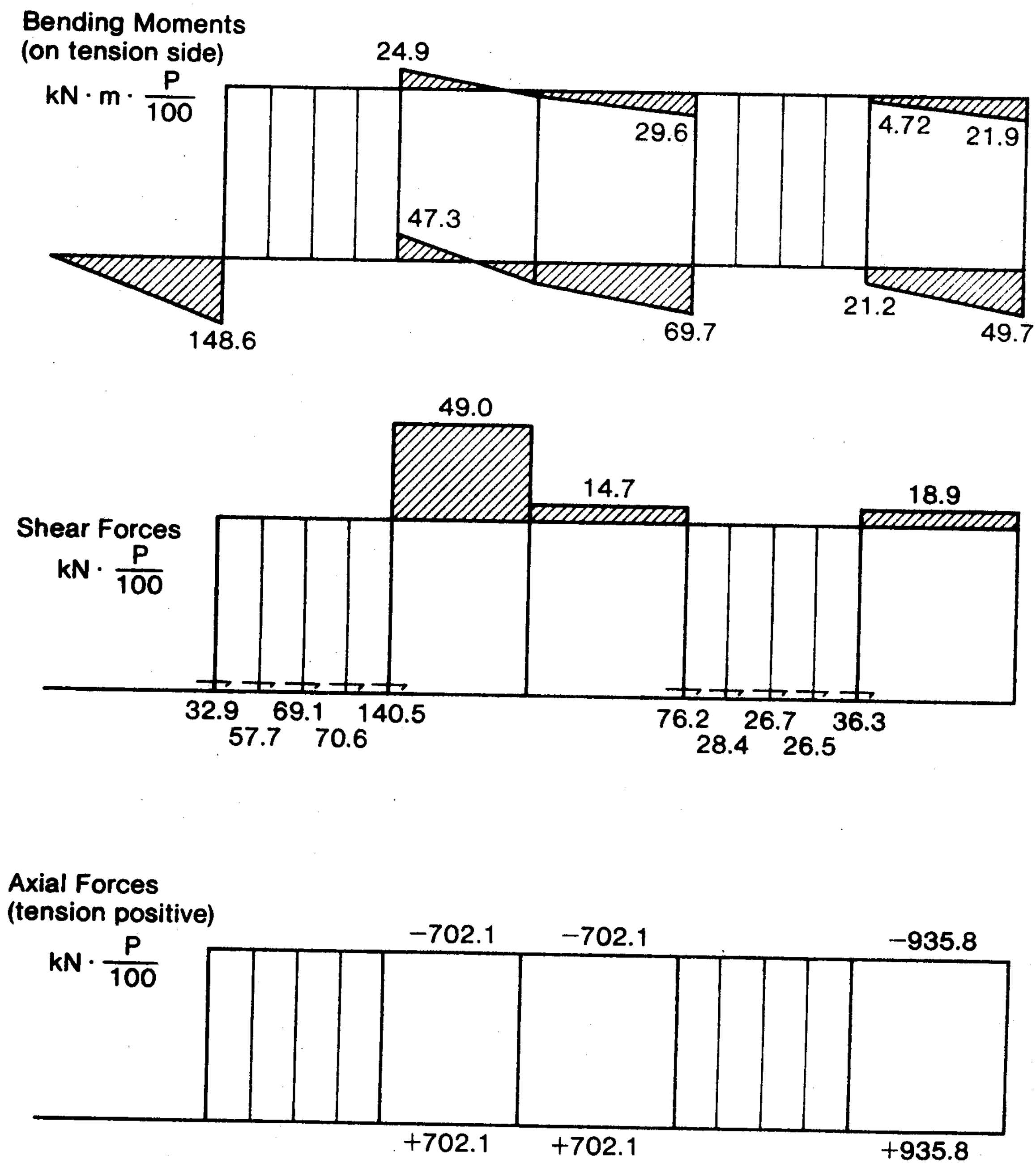
(d) Vierendeel Model - Variation 2



$I_1 = 3738 \times 10^6\ mm^4$
$A_1 = 48.5 \times 10^3\ mm^2$
$I_2 = 3950 \times 10^6\ mm^4$
$A_2 = 53.4 \times 10^3\ mm^2$

(e) Non-Prismatic Beam Model

Figure 6 (continued) Stub-Girder Modeling Schemes



Note : Multiply All Forces By P/100
P = Applied Load Per Jack

Figure 7 Bending Moments, Shear Forces and Axial Forces in Stub-Girder

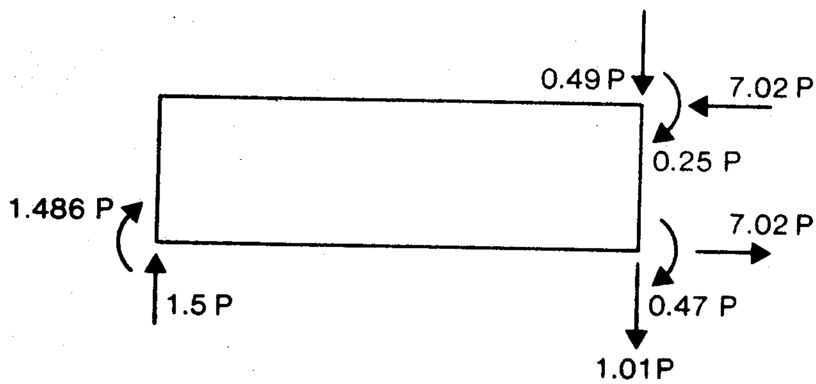


Figure 8 Stress Resultants Acting on Exterior Stub

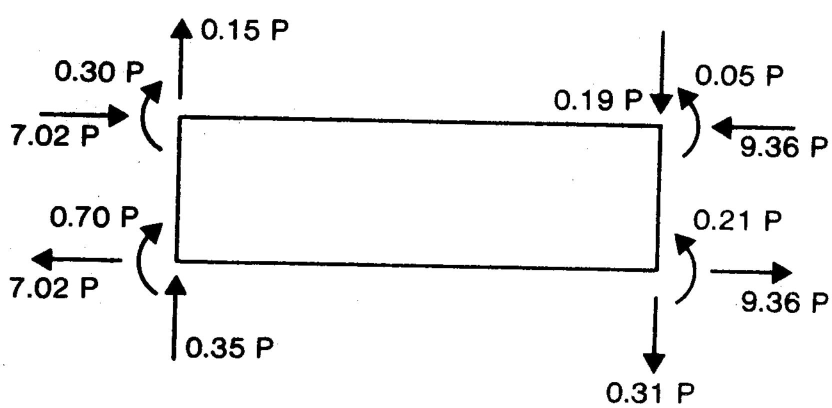


Figure 9 Stress Resultants Acting on Interior Stub

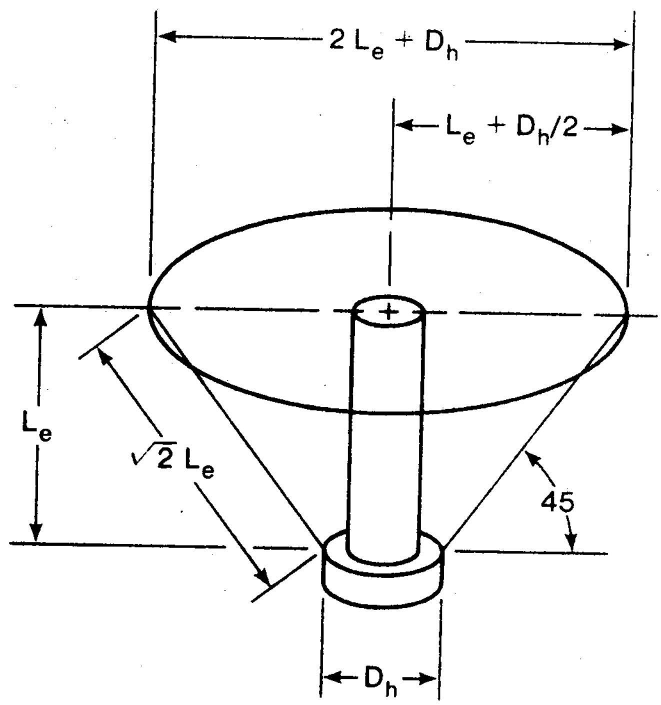


Figure 10 Dimensions of Stud Shear Connector and Surrounding Failure Cone

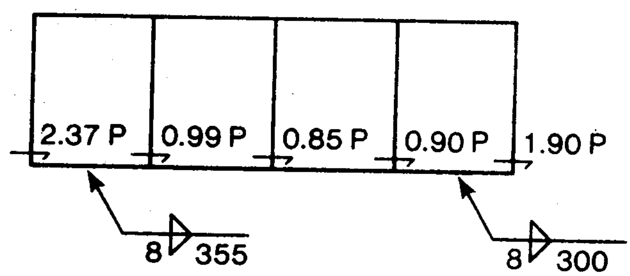


Figure 11 Shear Forces to Be Transferred by Welds for Exterior Stub

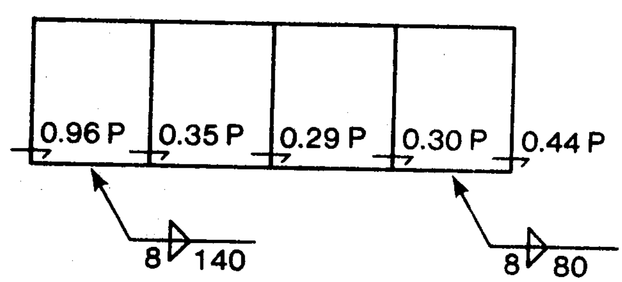


Figure 12 Shear Forces to Be Transferred by Welds for Interior Stub

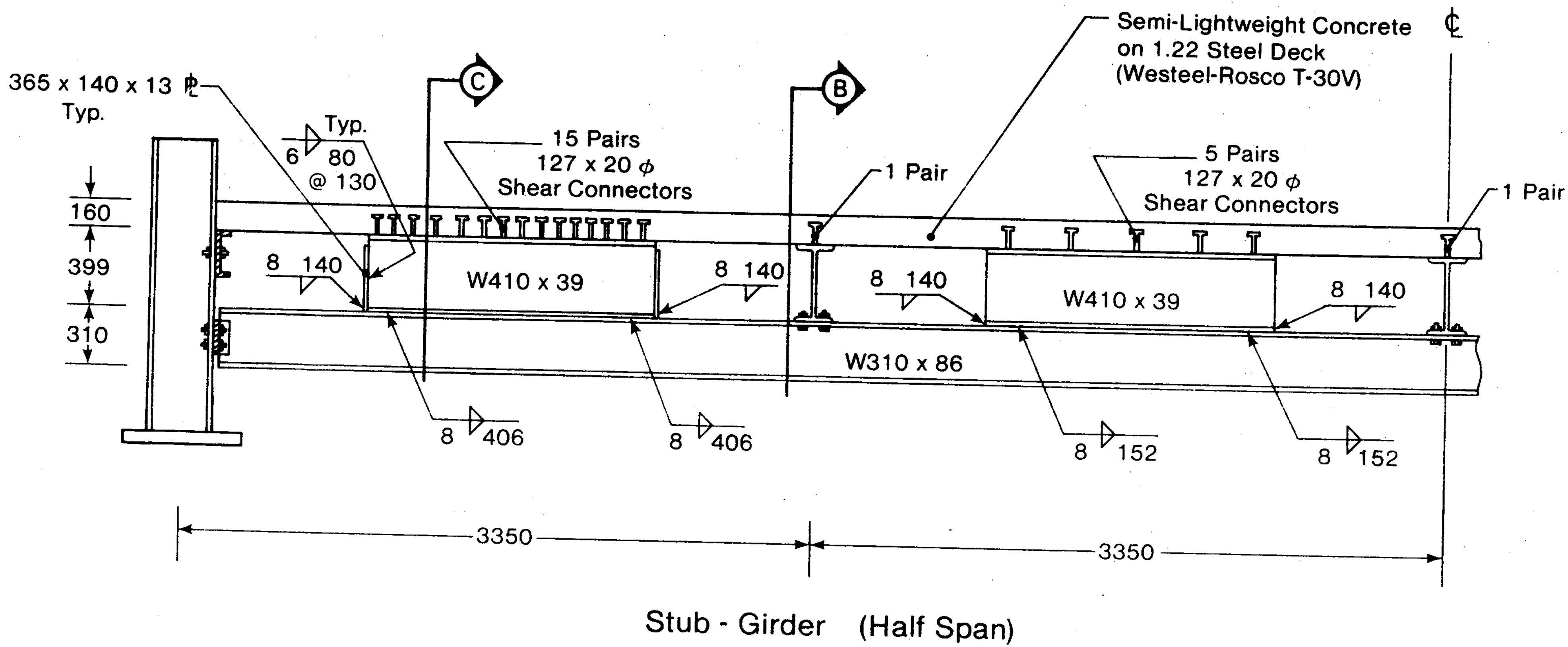


Figure 13 Elevation of Full-Size Stub-Girder

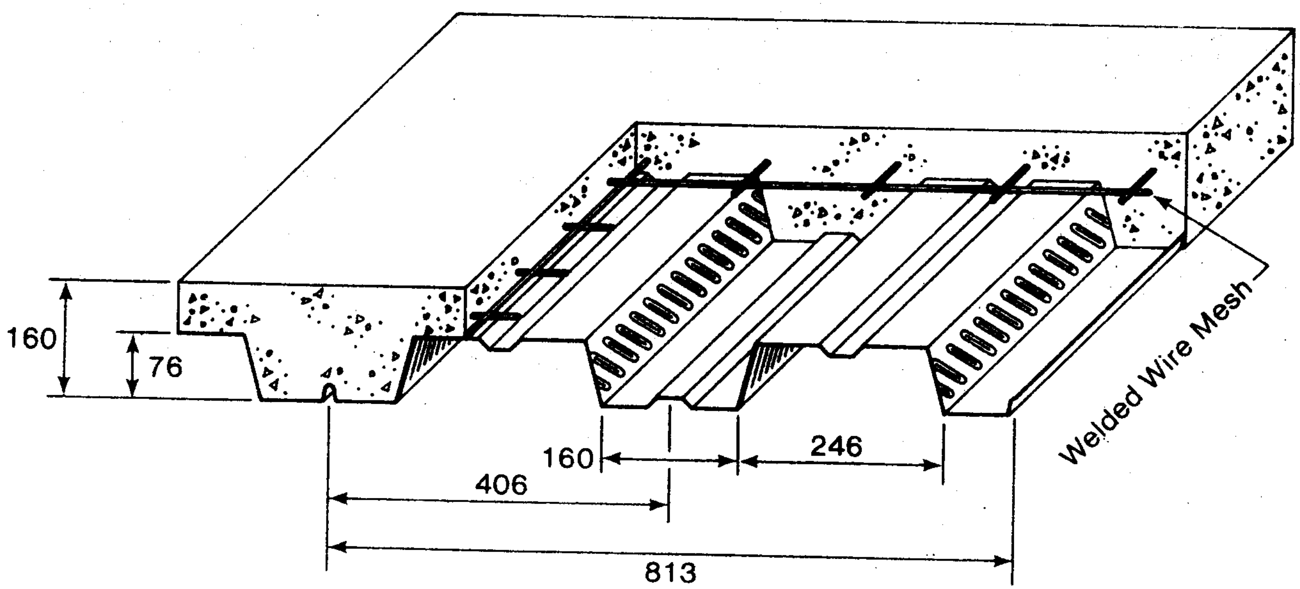


Figure 14 Cross Section of Corrugated Steel Deck and Concrete Slab

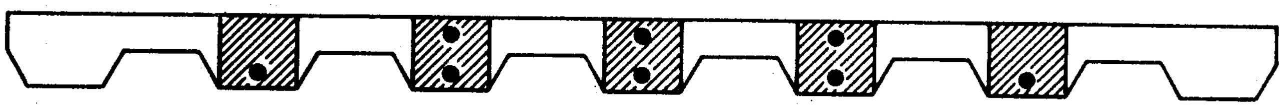


Figure 15 Simplified Cross-Sectional Model of Slab

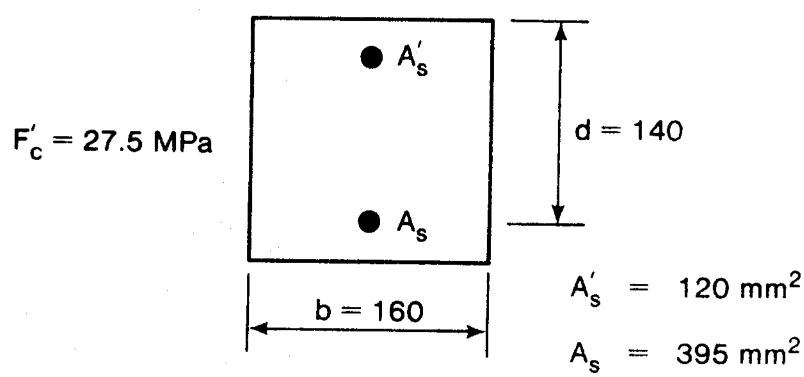


Figure 16 Details of One Element of the Simplified Cross-Sectional Slab Model

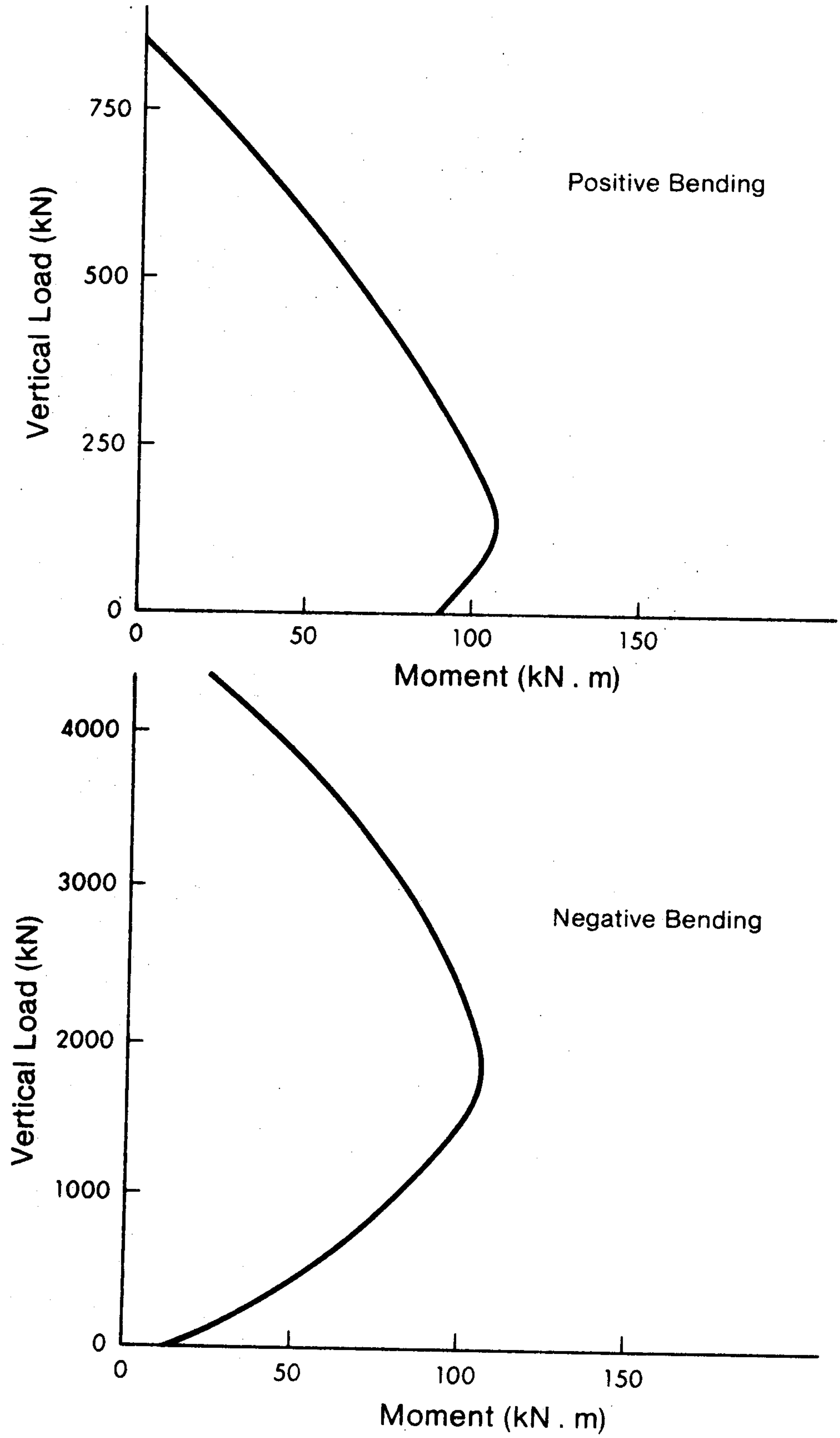
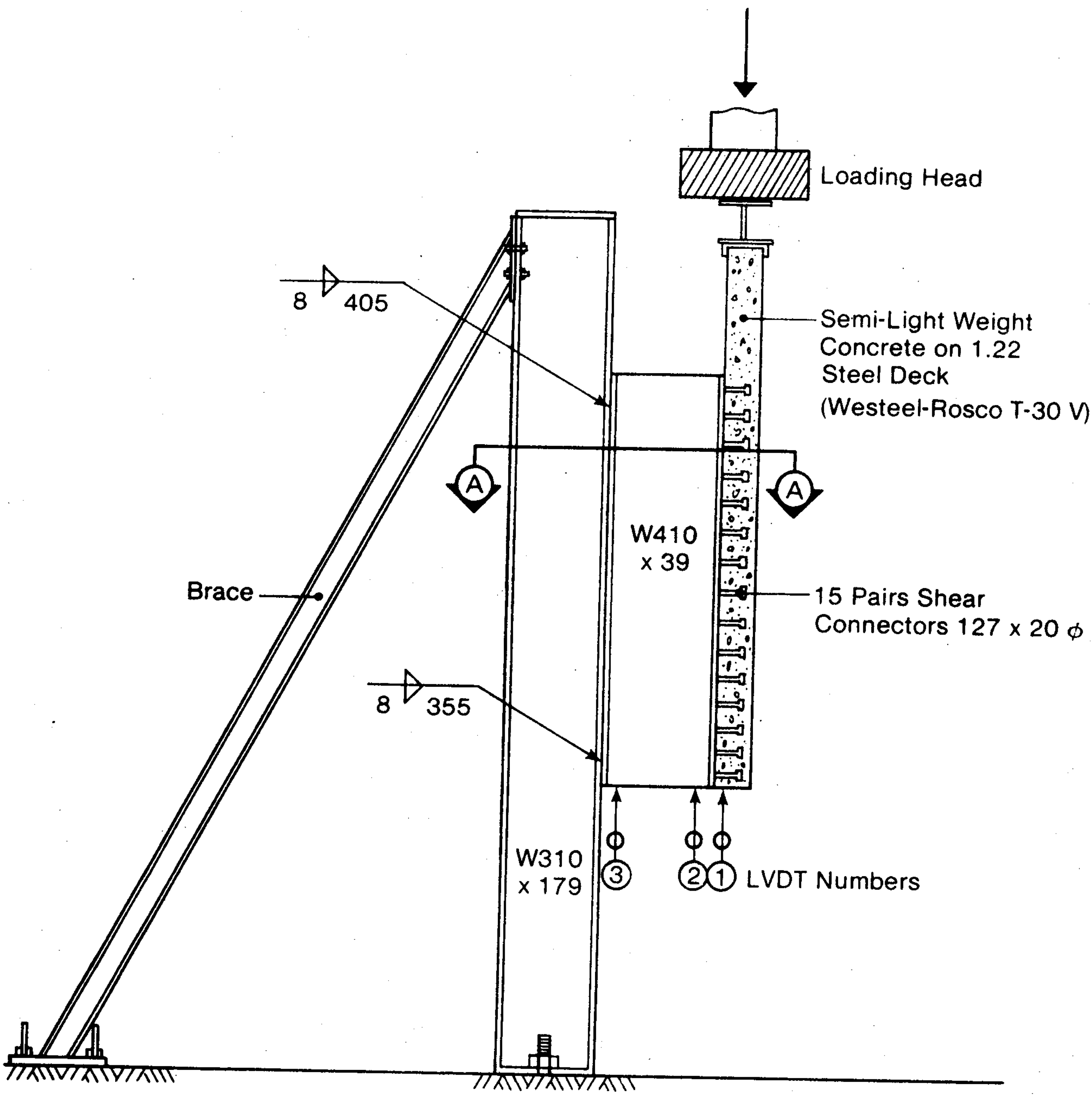
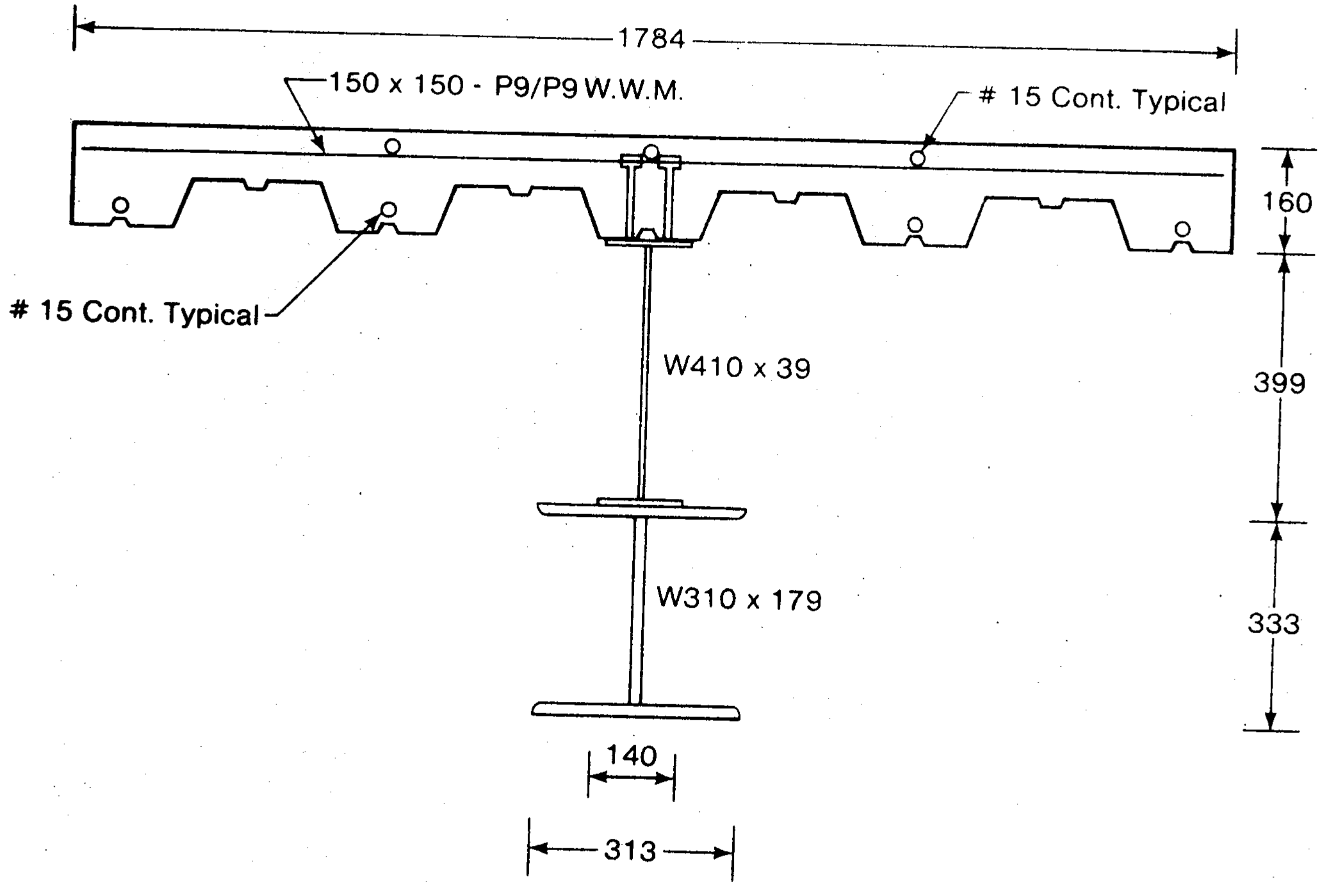


Figure 17 Interaction Diagrams for Cross-Sectional Slab Model



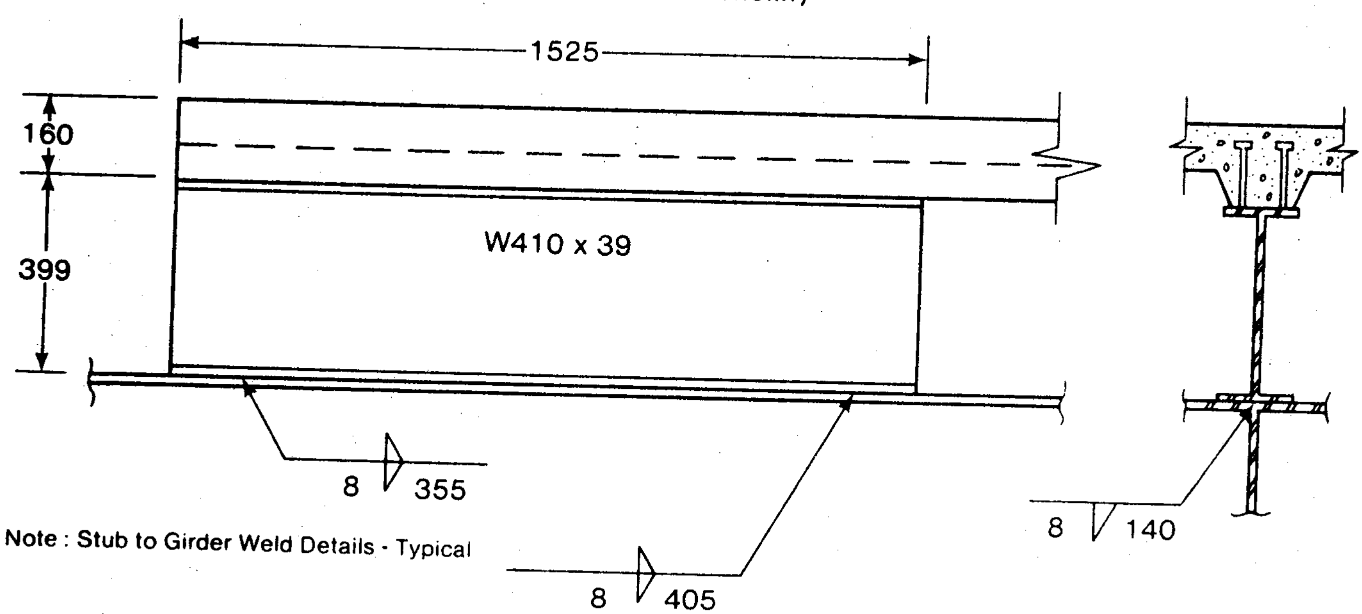
Single Stub Test Set-Up

Figure 18 Test Setup for a Stub-Slab Assembly (Stub Specimen). (No stiffening details are shown.)



Section A-A

Figure 19 Cross Section of a Typical Stub Specimen (stiffeners are not shown)



Specimen I

Figure 20 Stub Specimen I : No Stiffeners

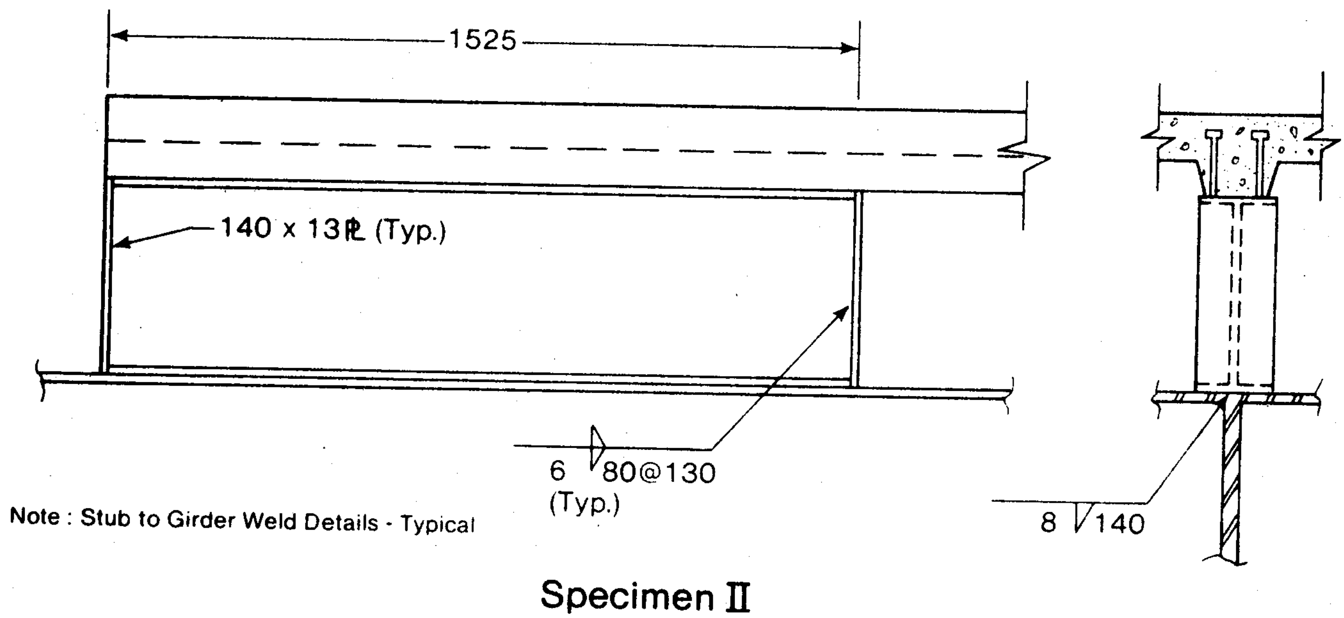


Figure 21 Stub Specimen II : Full End-Plate Stiffeners at Both Ends of Stub

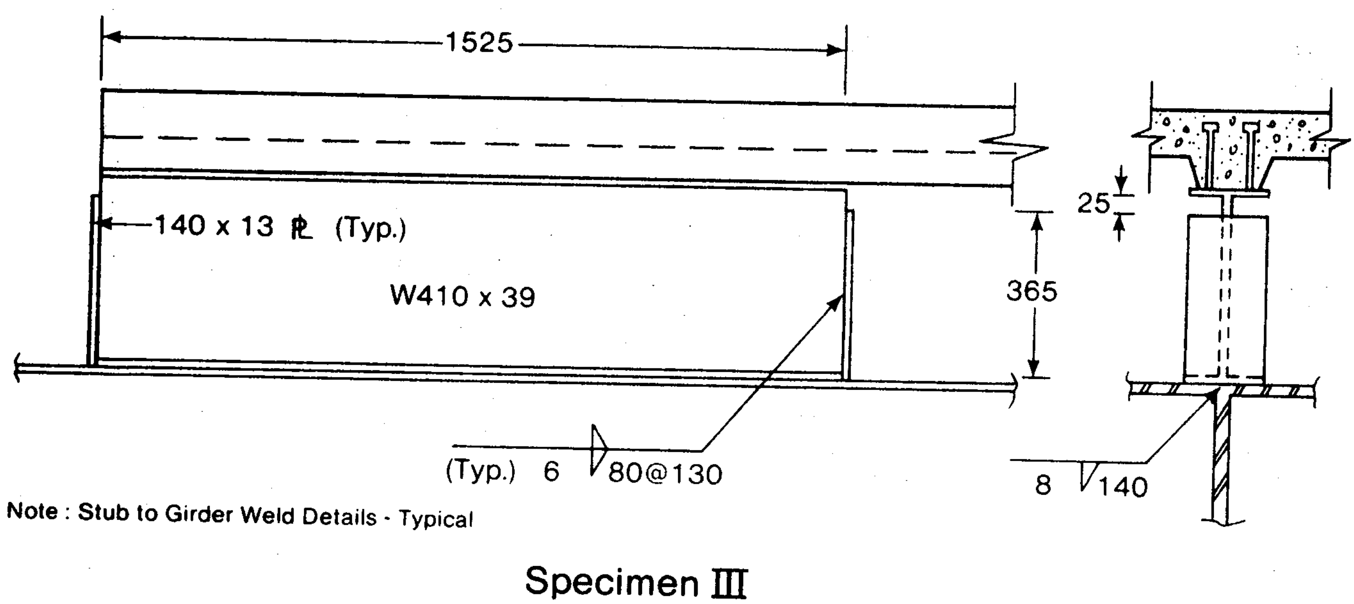
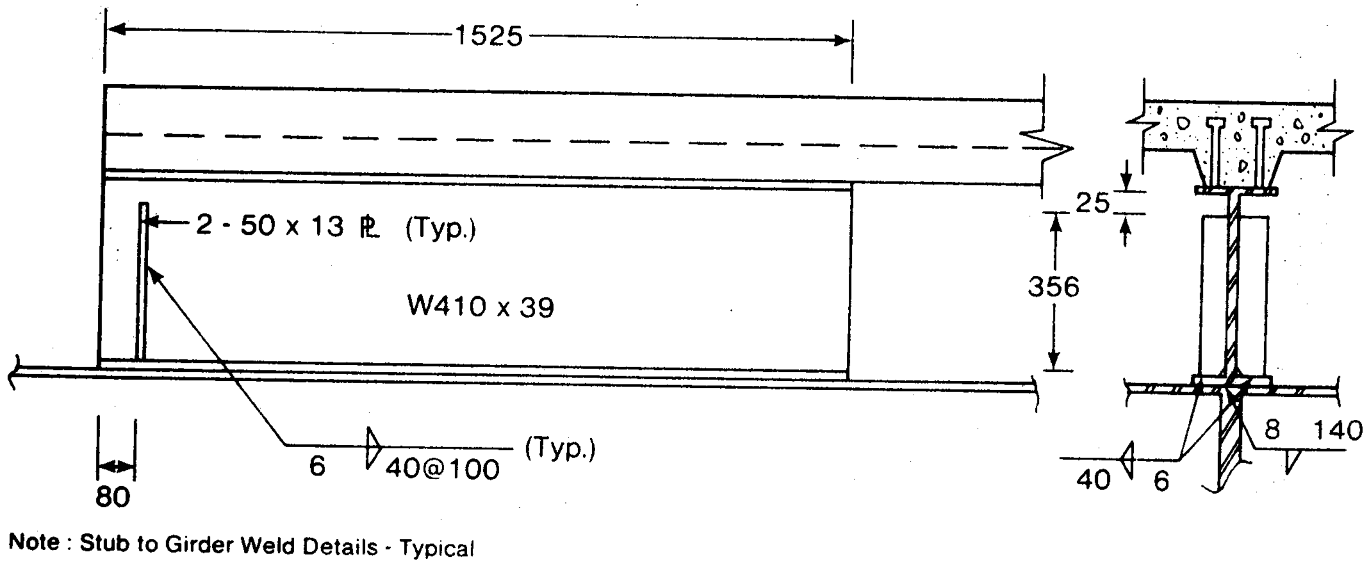
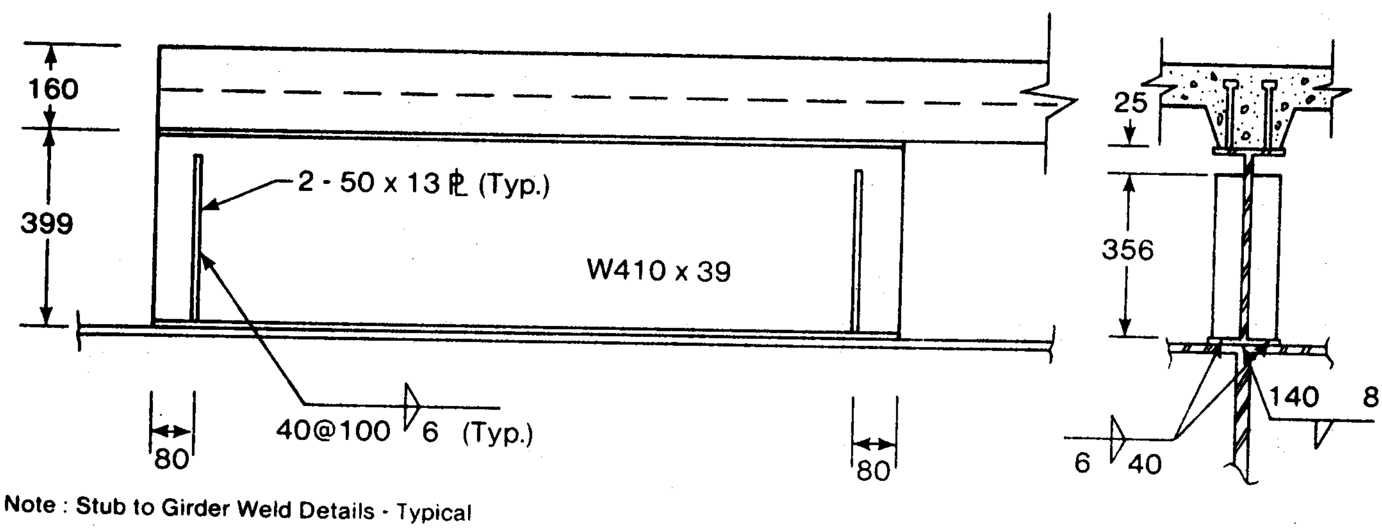


Figure 22 Stub Specimen III : Partial End-Plate Stiffeners at Both Ends of Stub



Specimen IV

Figure 23 Stub Specimen IV : Fitted Stiffener Only in Compression Zone of Stub



Specimen V

Figure 24 Stub Specimen V : Fitted Stiffeners at Both Ends of Stub

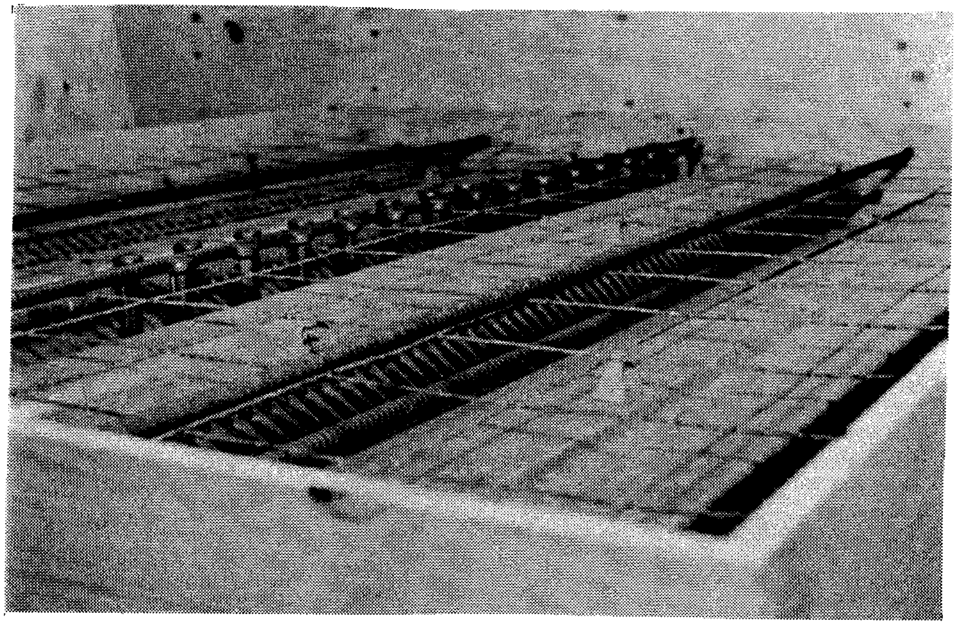


Figure 25 Stub Specimen before Casting of Concrete Slab

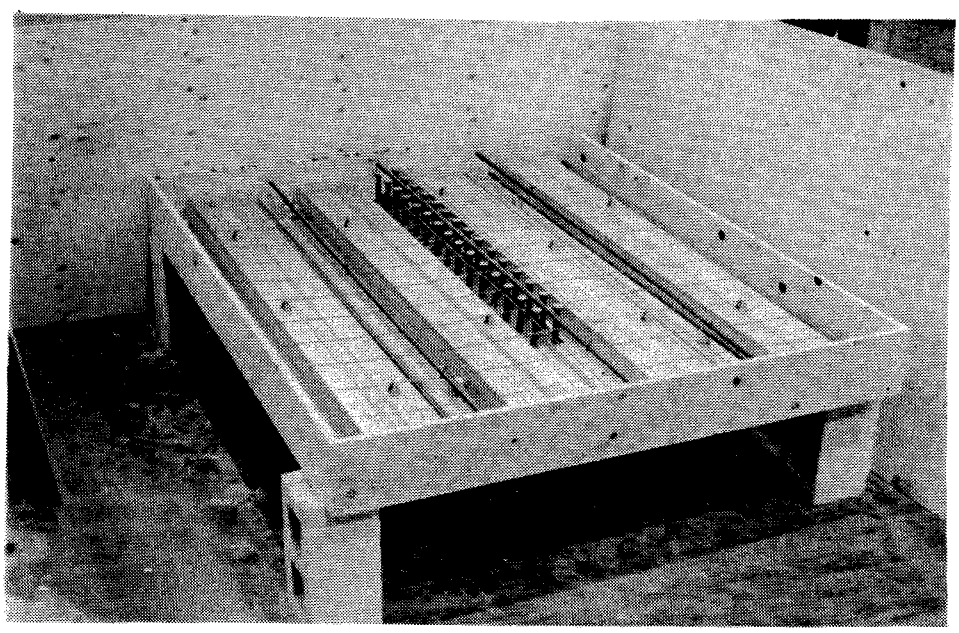


Figure 26 Stub Specimen before Casting of Concrete Slab

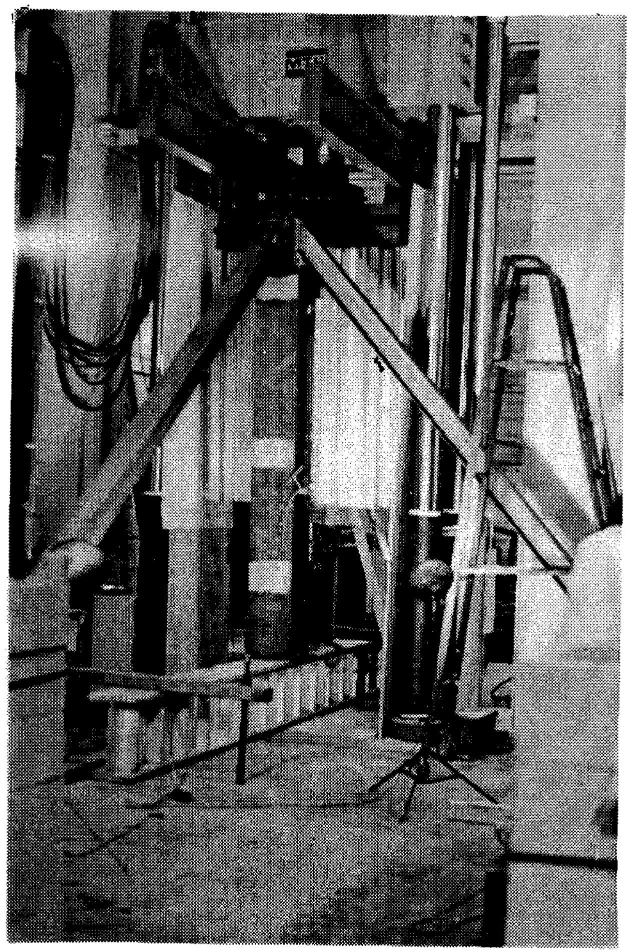


Figure 27 Stub Specimen Test Setup

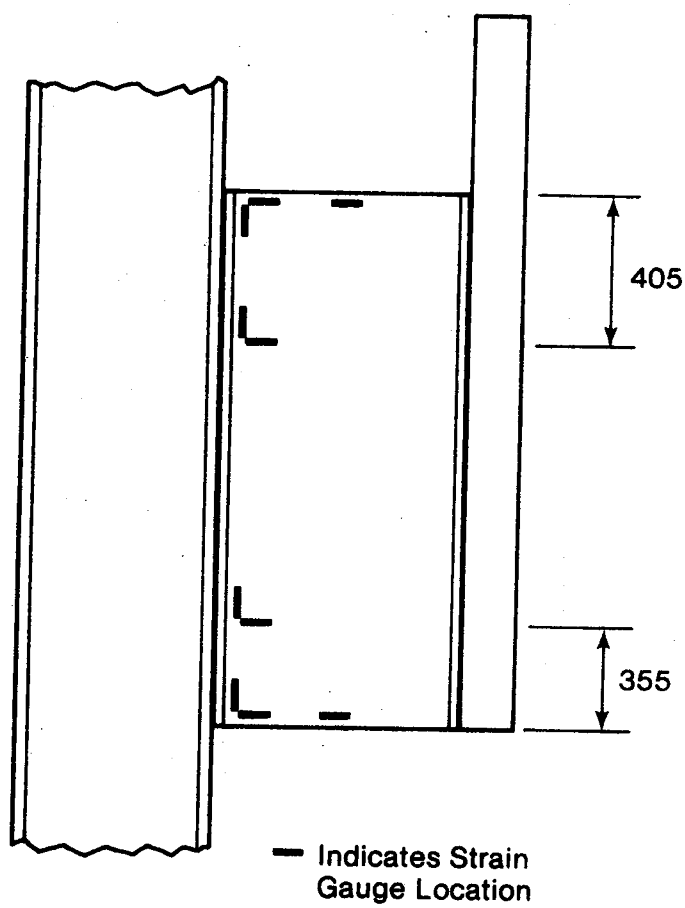


Figure 28 Locations of Strain Gauges on Single Stub

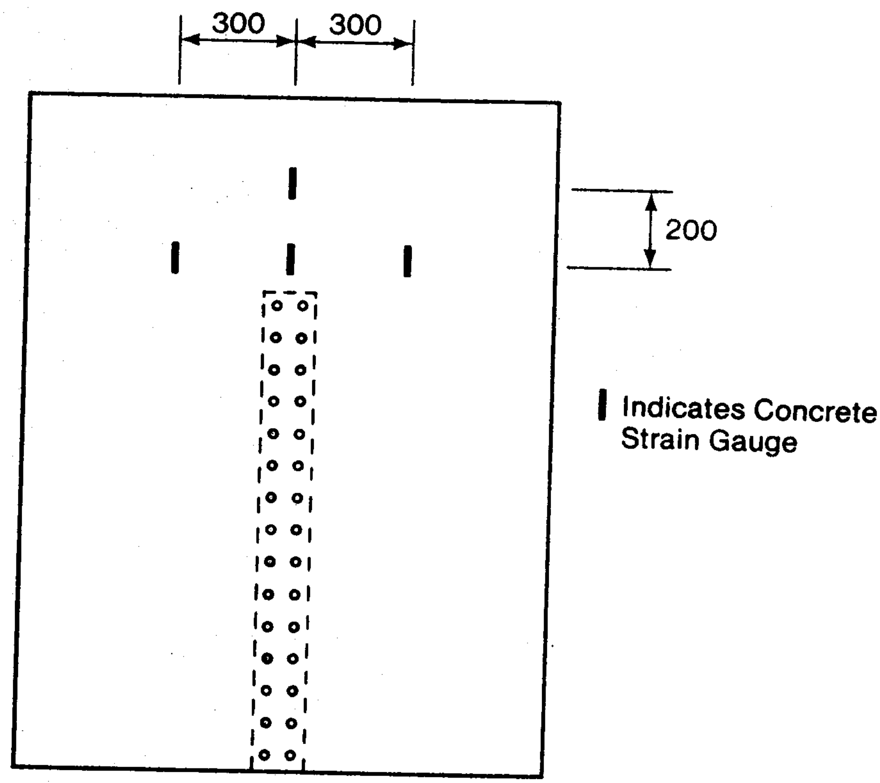


Figure 29 Locations of Concrete Strain Gauges on Slab of Stub Specimen

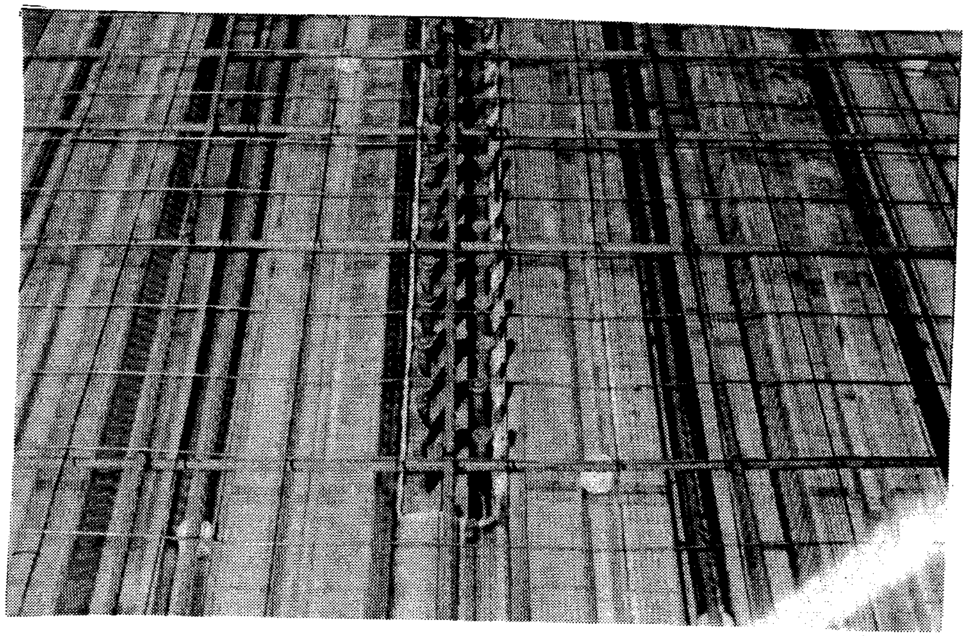


Figure 30 Placement of Stud Shear Connectors on Exterior Stub of Full-Size Stub-Girder

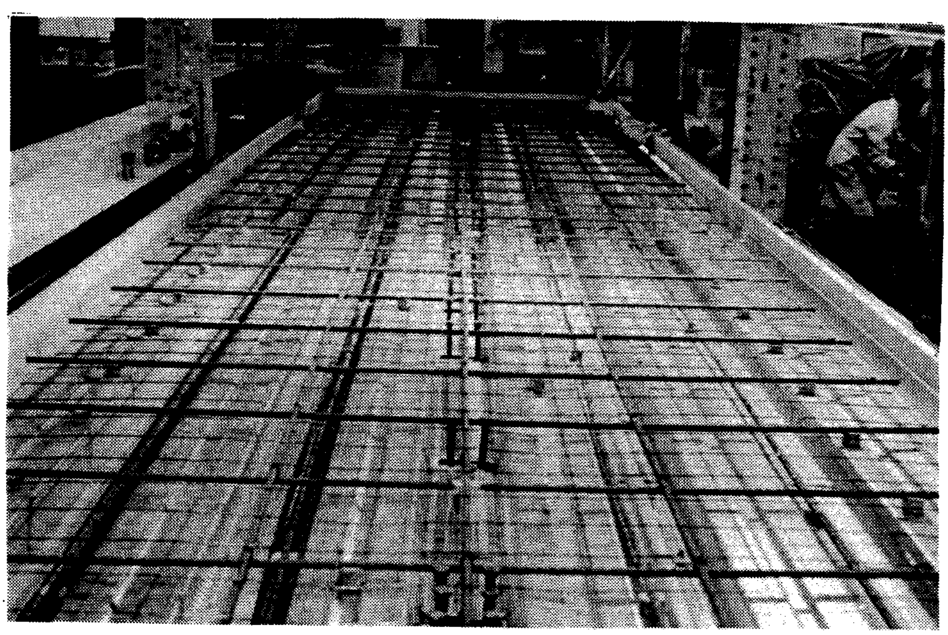
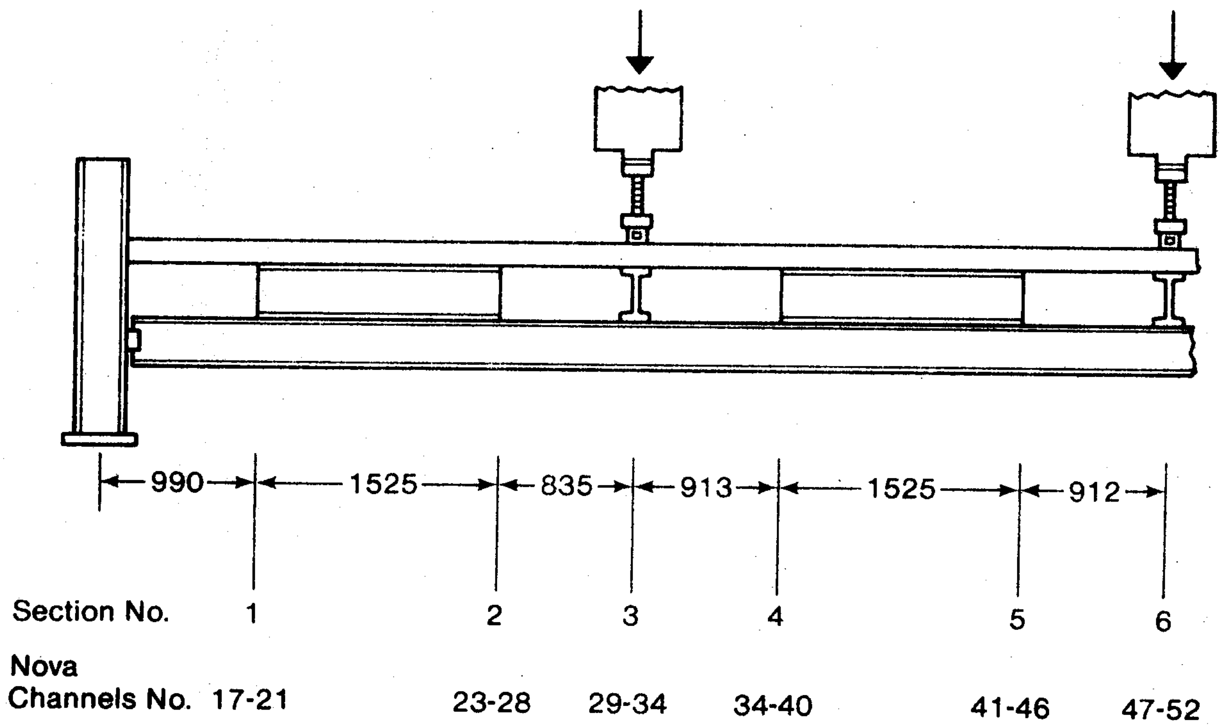


Figure 31 Full-Size Stub-Girder before Casting of Slab



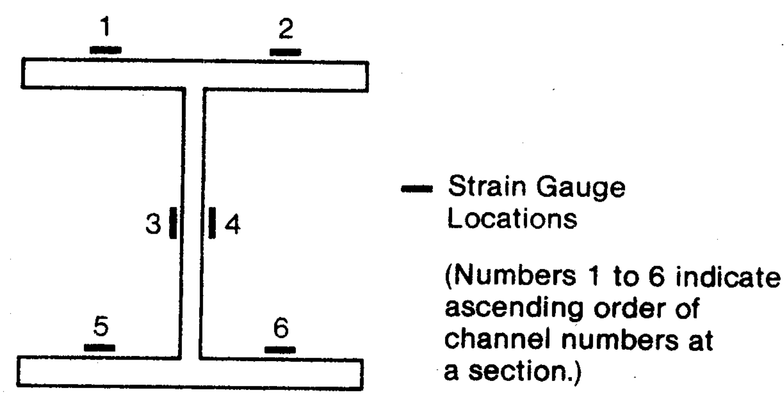


Figure 32 Locations of Strain Gauges at Various Sections of the Full-Size Stub-Girder

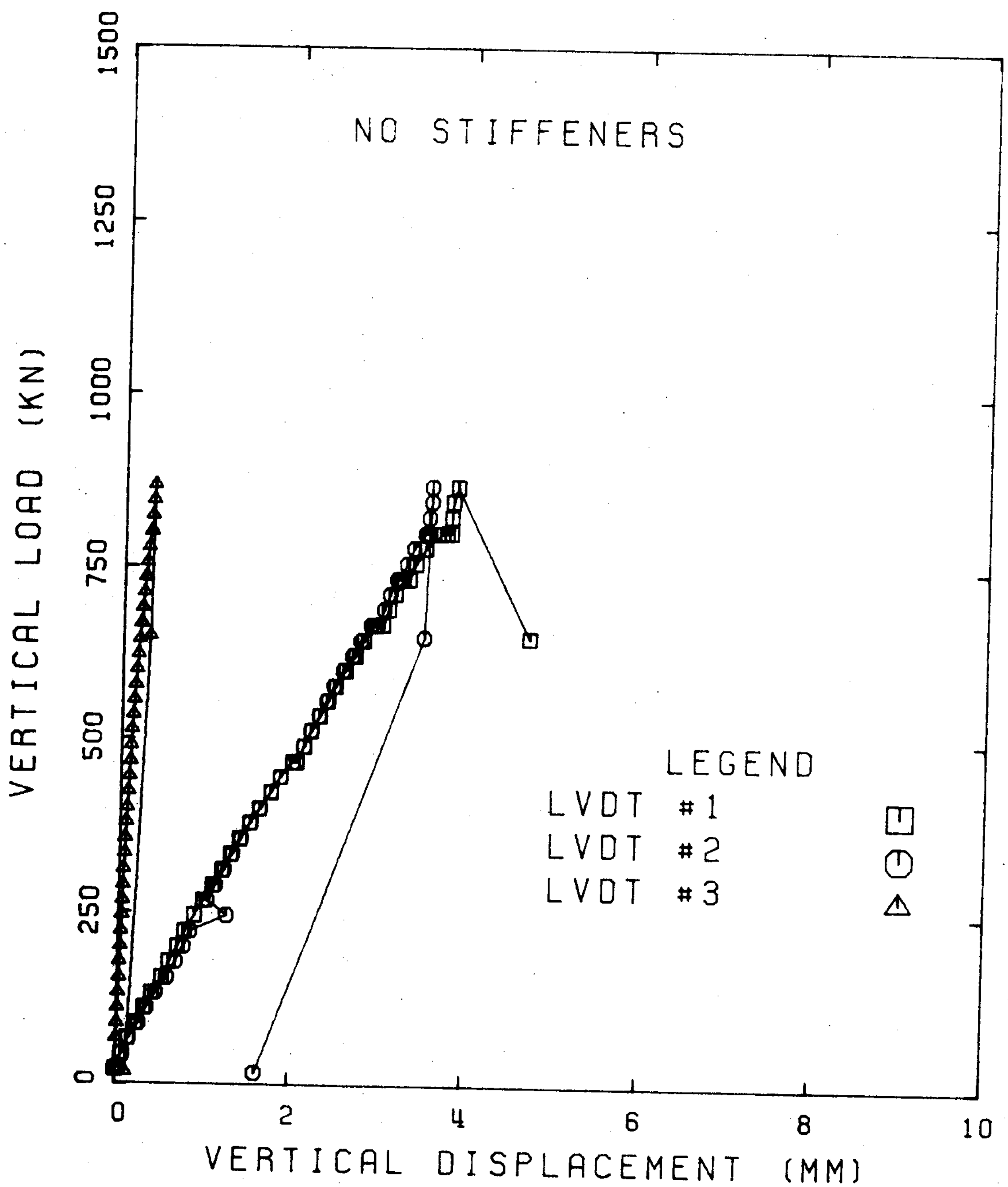


Figure 33 Load-Deformation Measurements for Stub Specimen I (No Stiffeners)

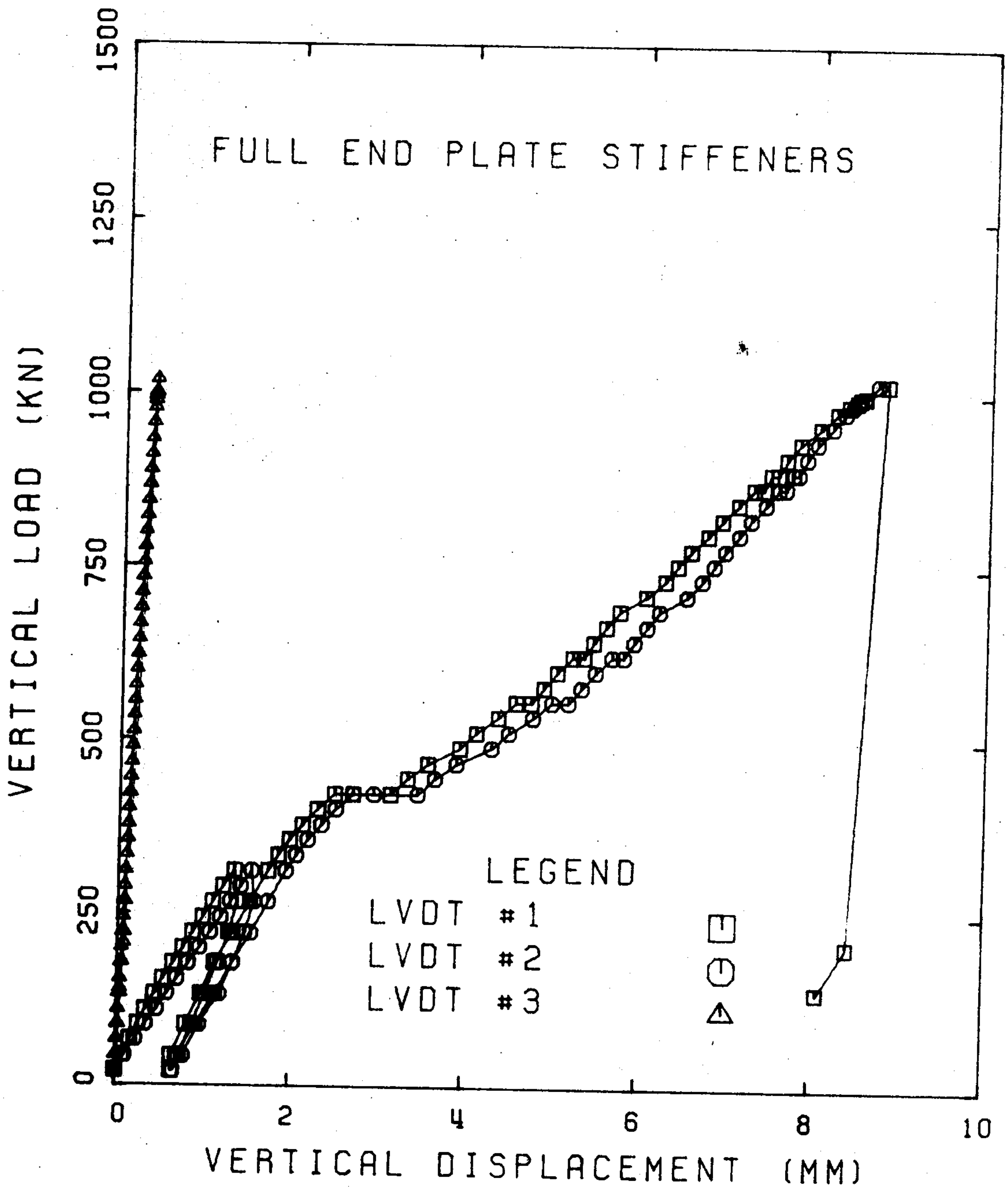


Figure 34 Load-Deformation Measurements for Stub Specimen II (Full end-plate stiffeners at both ends of stub)

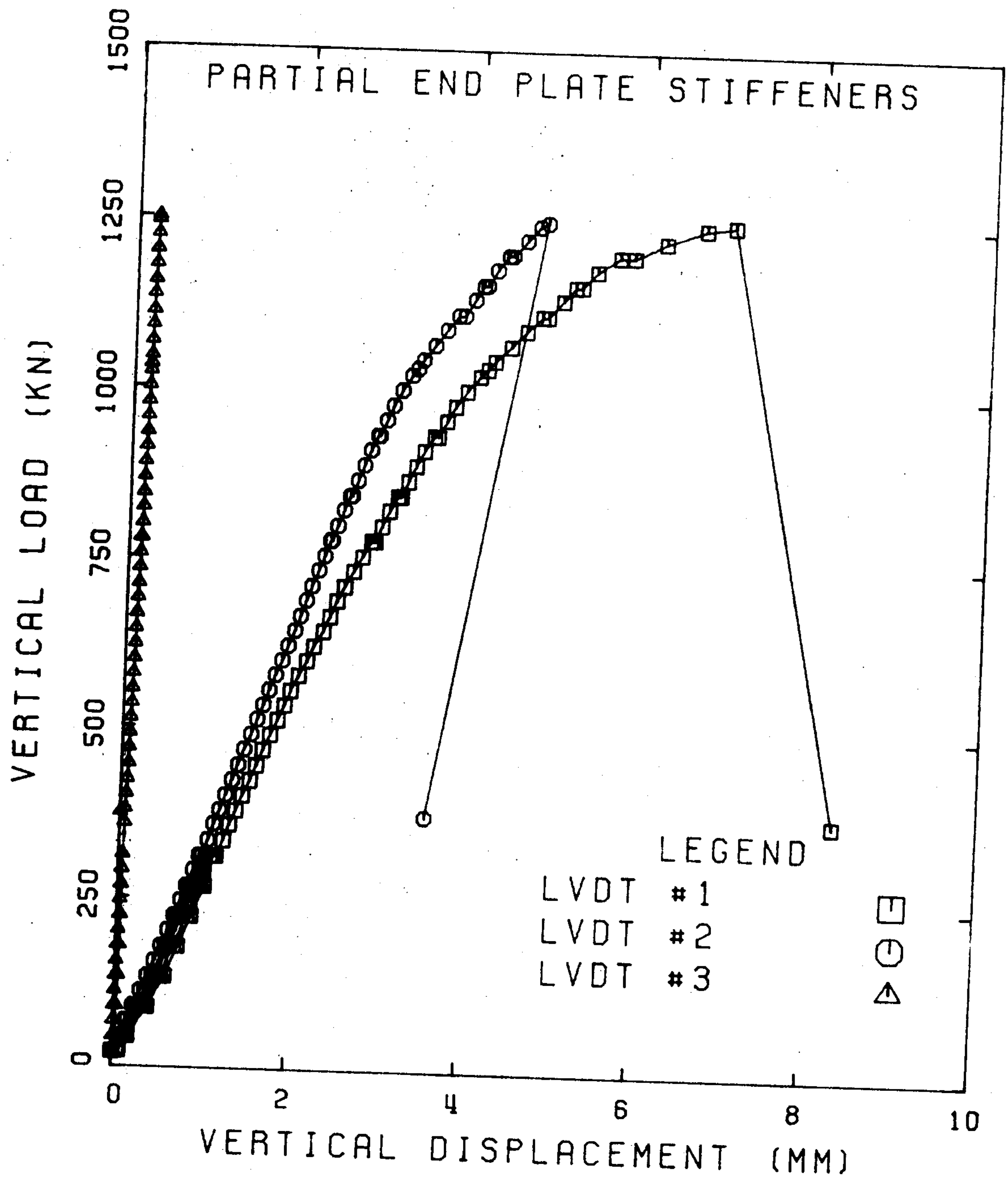


Figure 35 Load-Deformation Measurements for Stub Specimen III (Partial end-plate stiffeners at both ends of stub)

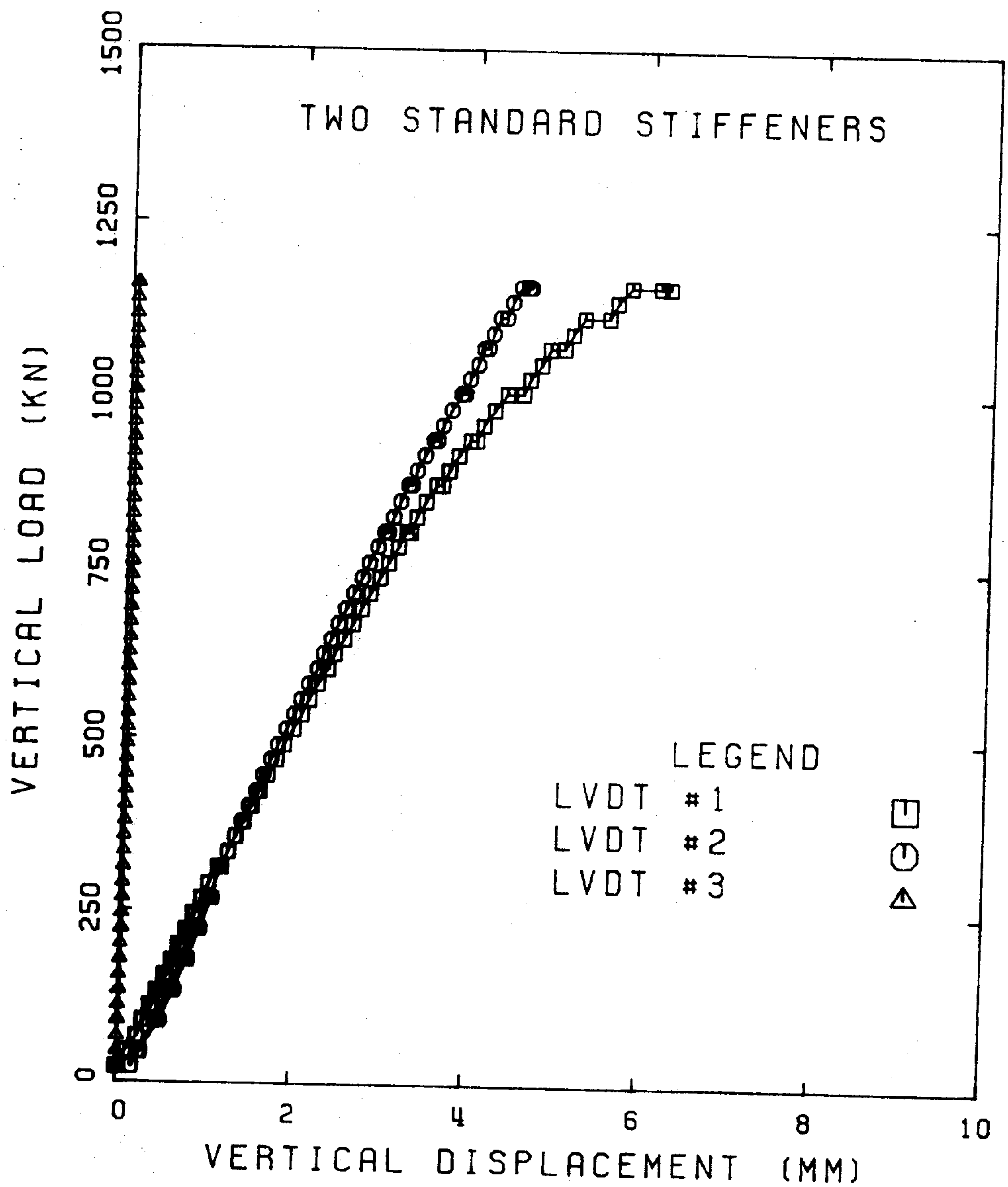


Figure 37 Load-Deformation Measurements for Stub Specimen V (Fitted stiffeners at both ends of stub)

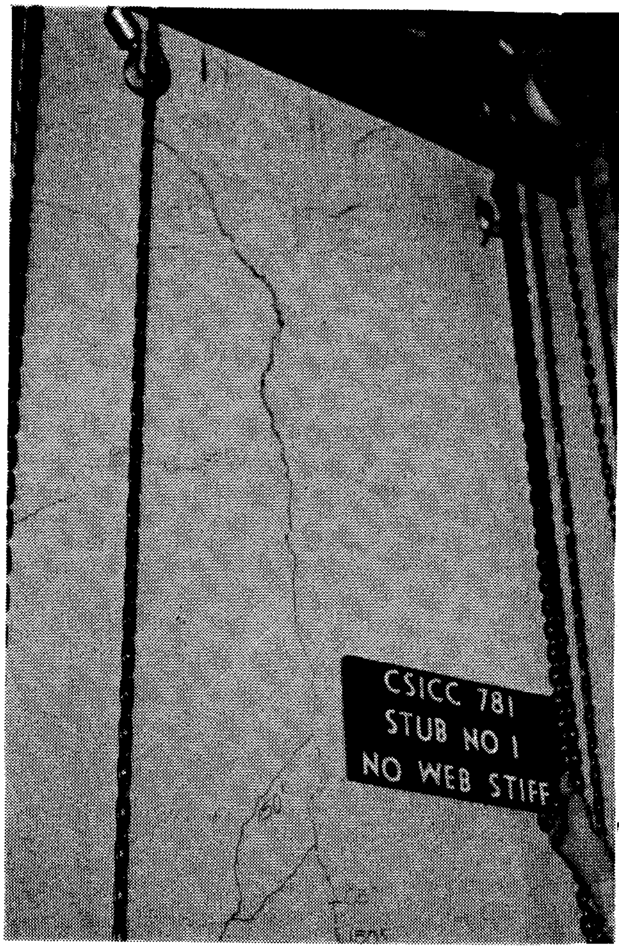


Figure 38

Cracking Pattern in Slab of Stub Specimen I

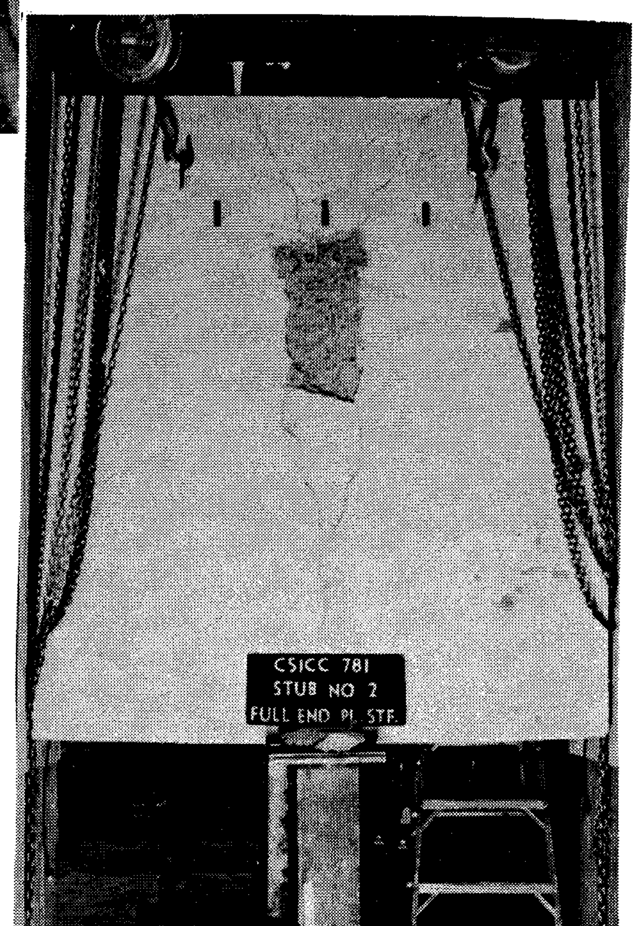


Figure 39

Cracking Pattern in Slab of Stub Specimen II

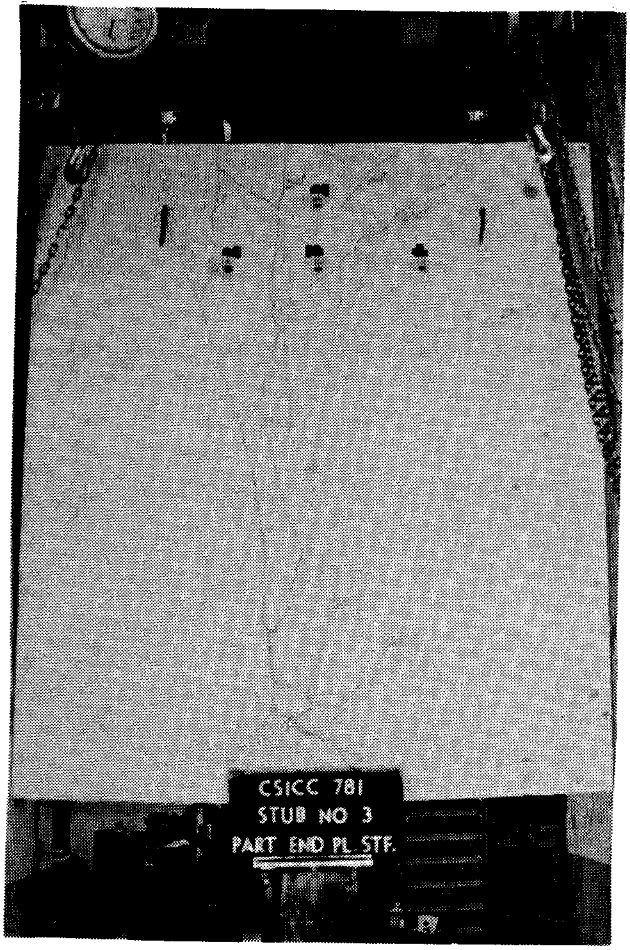


Figure 40

Cracking Pattern in Slab of Stub Specimen III

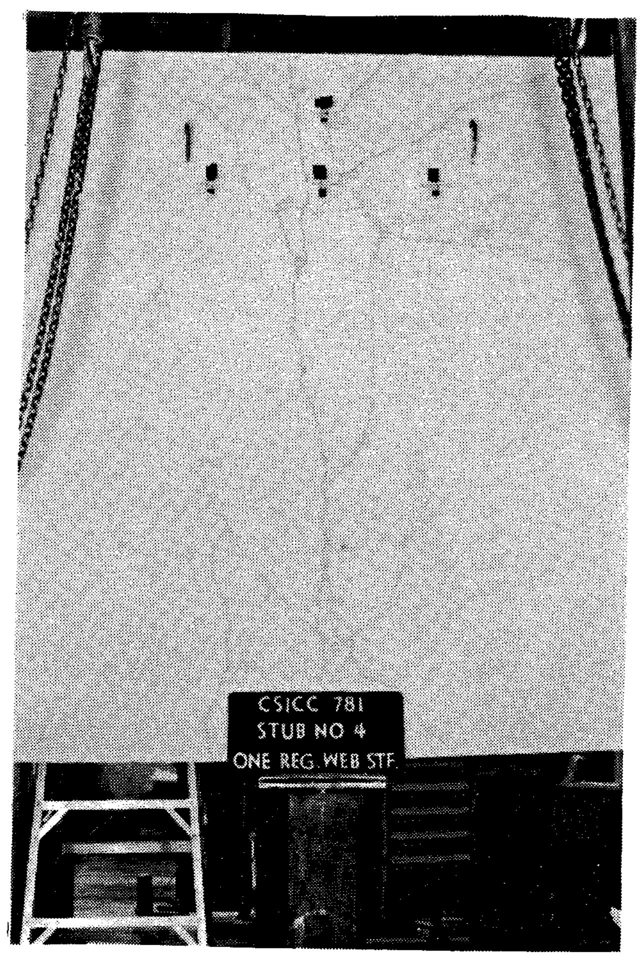


Figure 41

Cracking Pattern in Slab of Stub Specimen IV

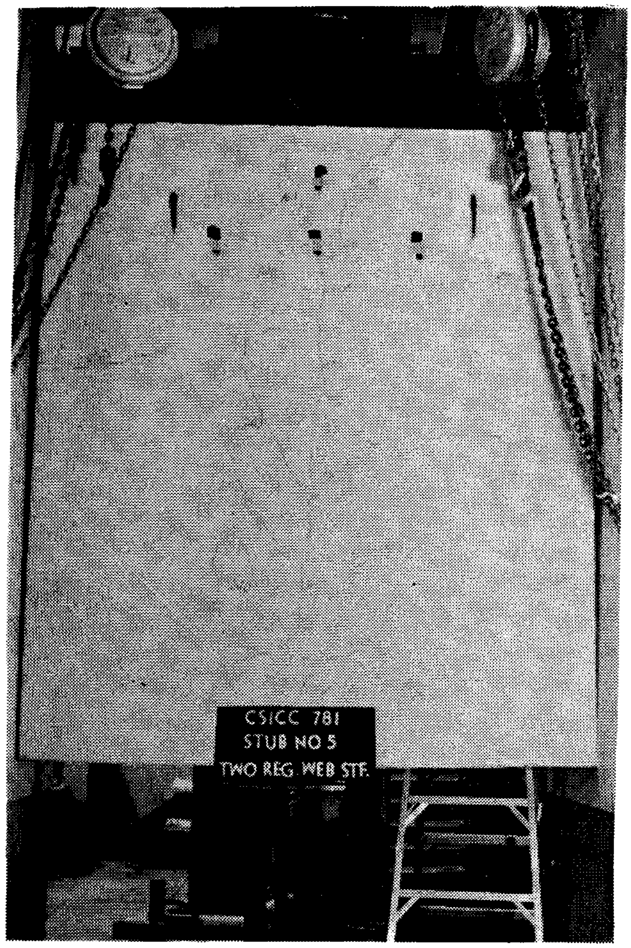


Figure 42 Cracking Pattern in Slab of Stub Specimen V

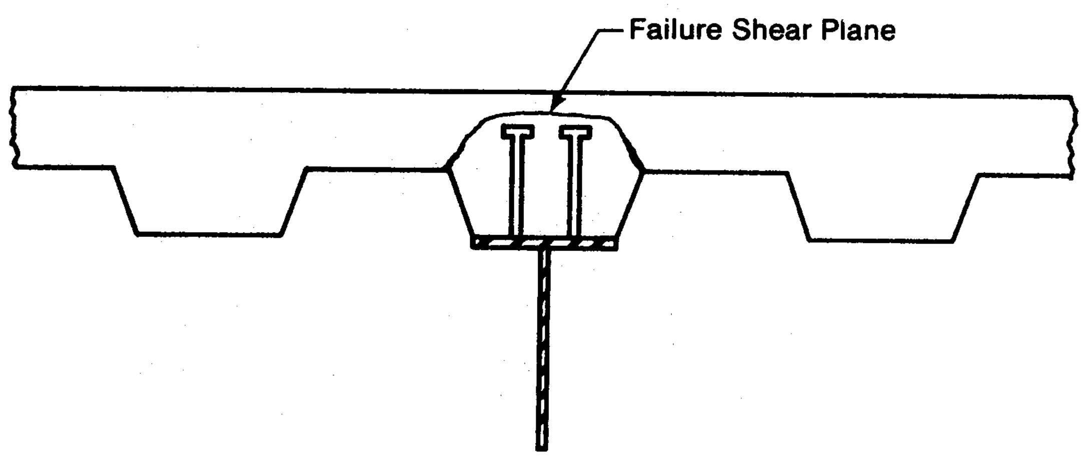


Figure 43 Location of Typical Shear Failure Plane in Slab of Stub

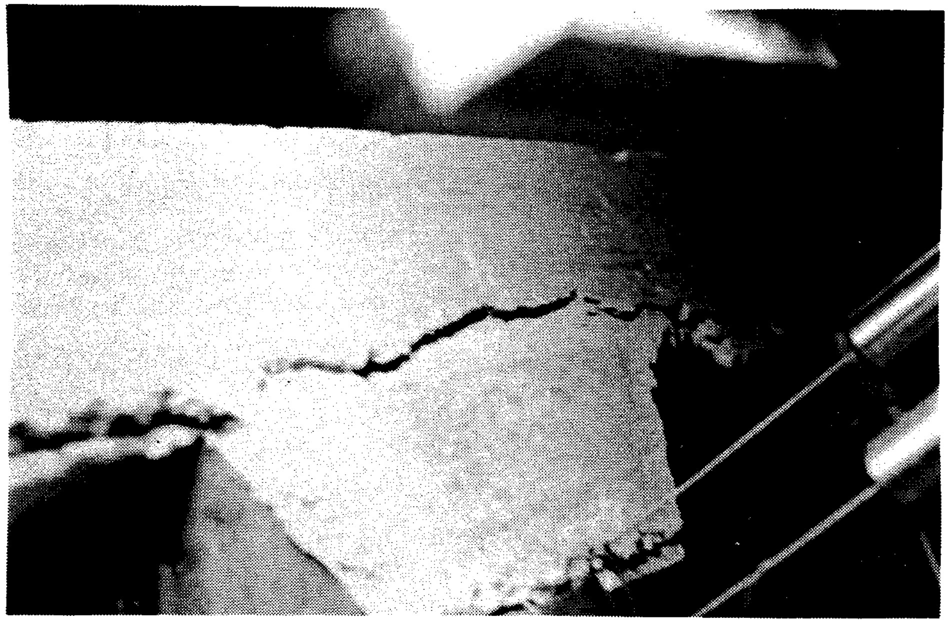


Figure 44 Shear Failure Plane in One of the Test Specimens

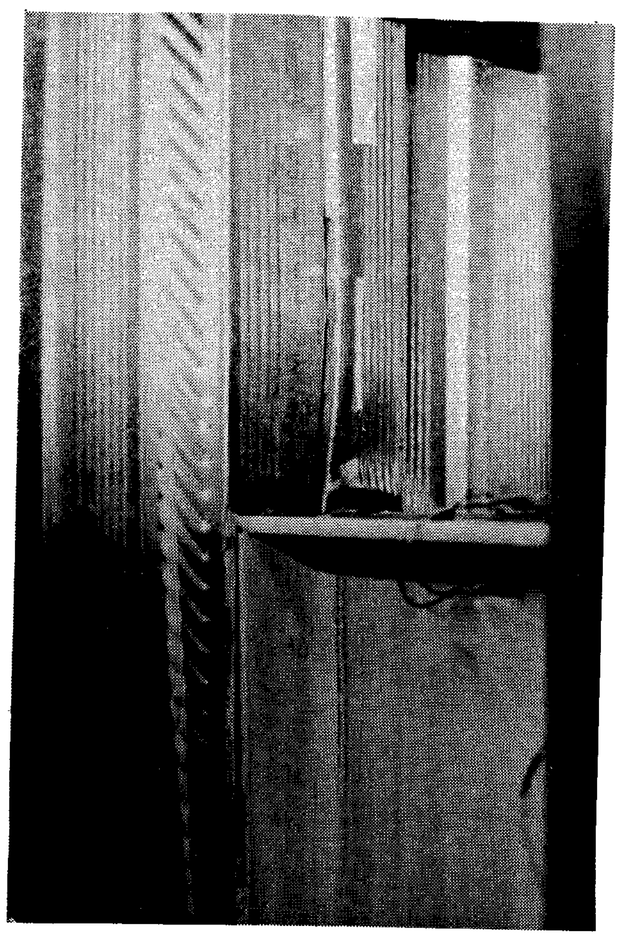


Figure 45 Buckling of Steel Deck in a Stub Specimen

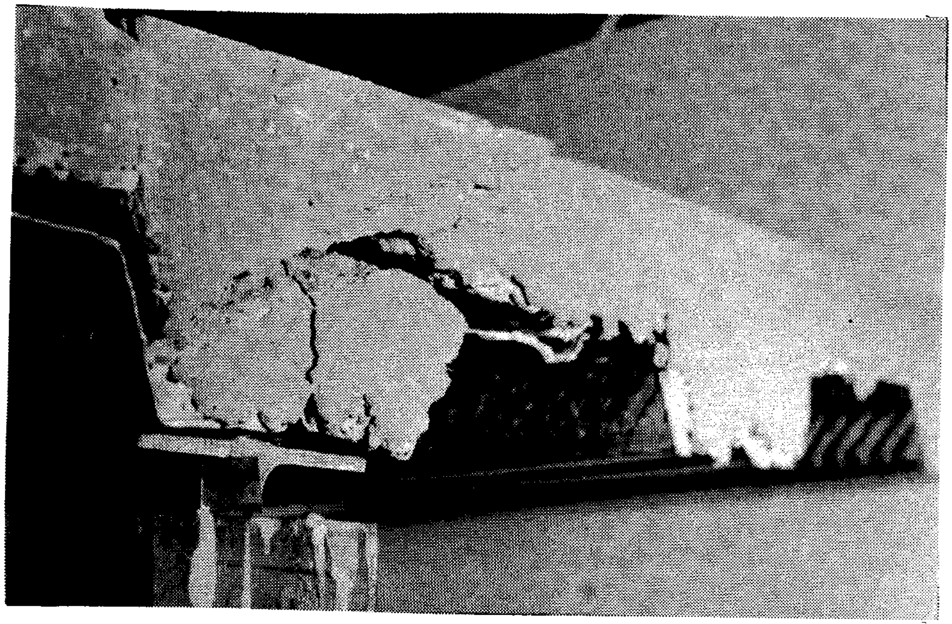


Figure 46 Shear Failure Plane at End of Slab in Stub Specimen III (Partial End-Plate Stiffeners)

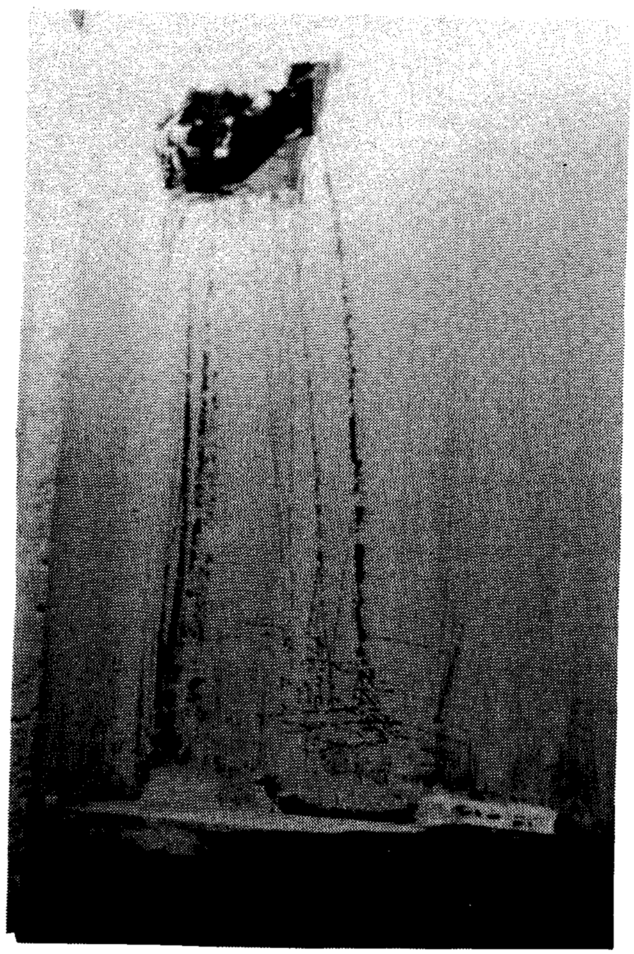


Figure 47 Signs of Local Yielding, Evidenced by Whitewash Flaking in Web of Stub Specimen IV

Figure 48 Weld Failure for Stiffener in Tension Region of Stub Specimen V (Two fitted web stiffeners)

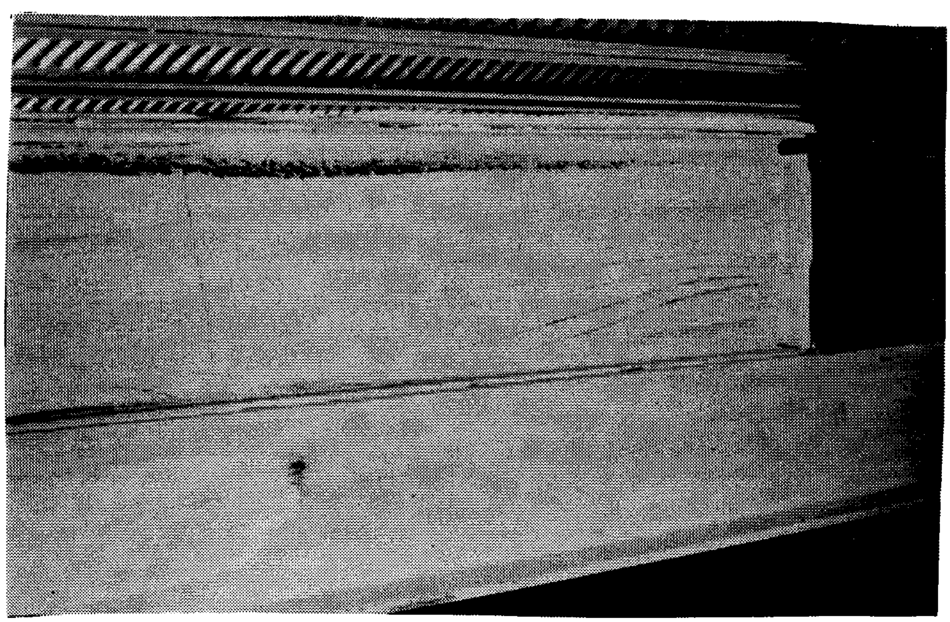


Figure 49 Yield Line Pattern in South Exterior Stub of Full-Size Stub-Girder (Load of 210 kN (47.2 kips)/jack)

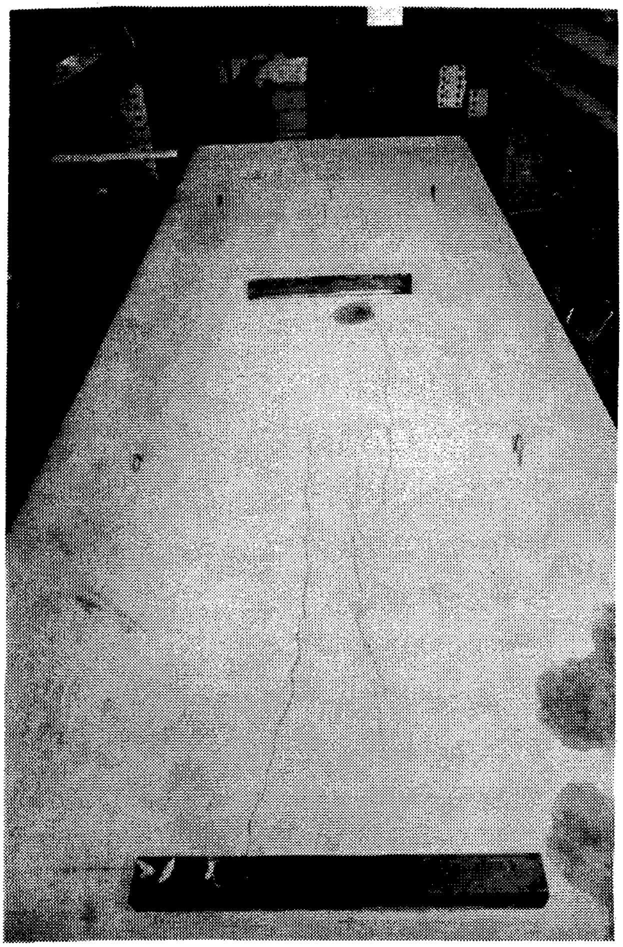


Figure 52

Crack Pattern in Southern Half of Slab of Full-Size Stub-Girder

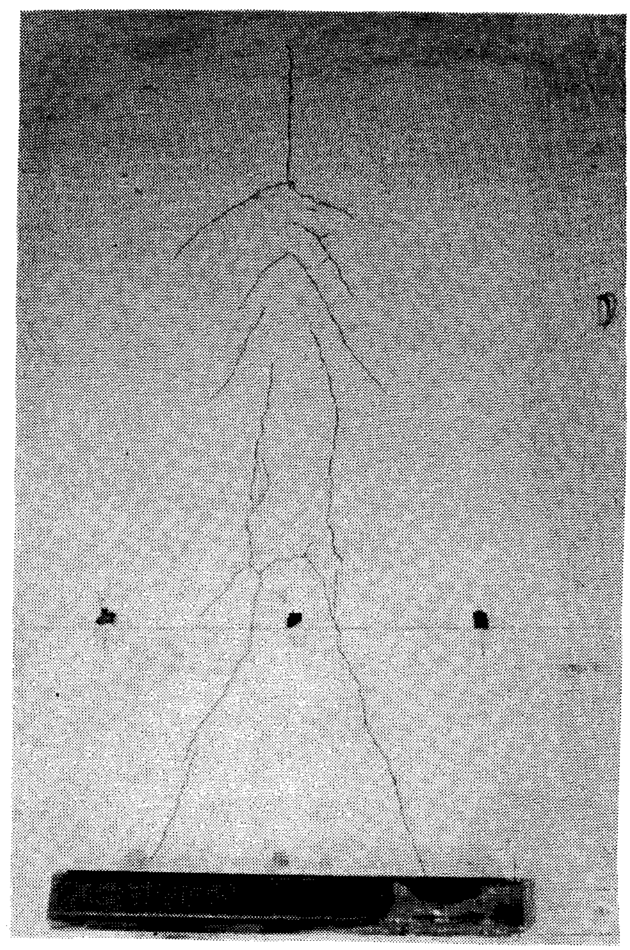


Figure 53

Crack Pattern in Slab of Full-Size Stub-Girder, Directly above North Exterior Stub

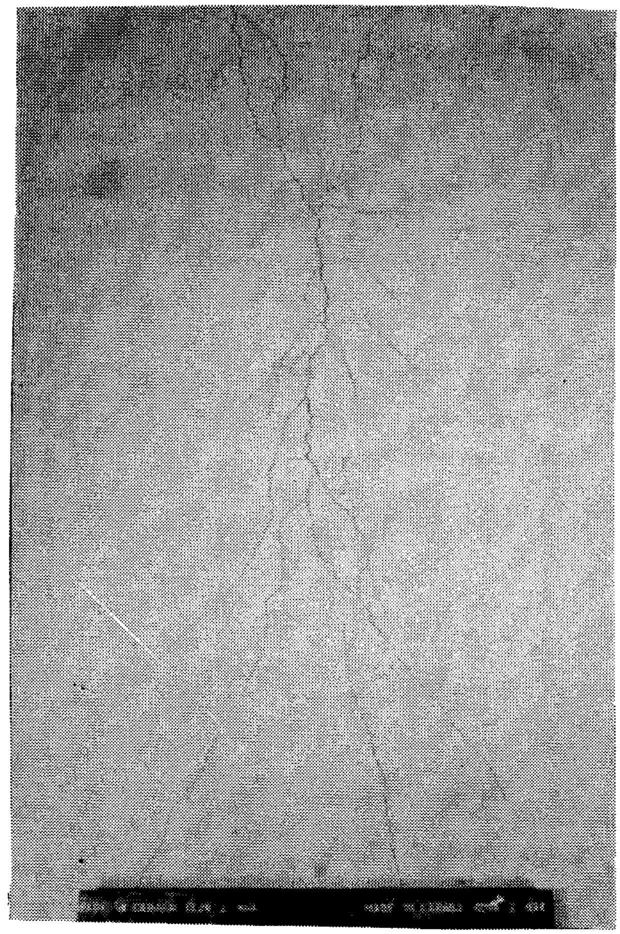


Figure 54 Crack Pattern in Slab of Full-Size Stub-Girder, Directly above South Exterior Stub

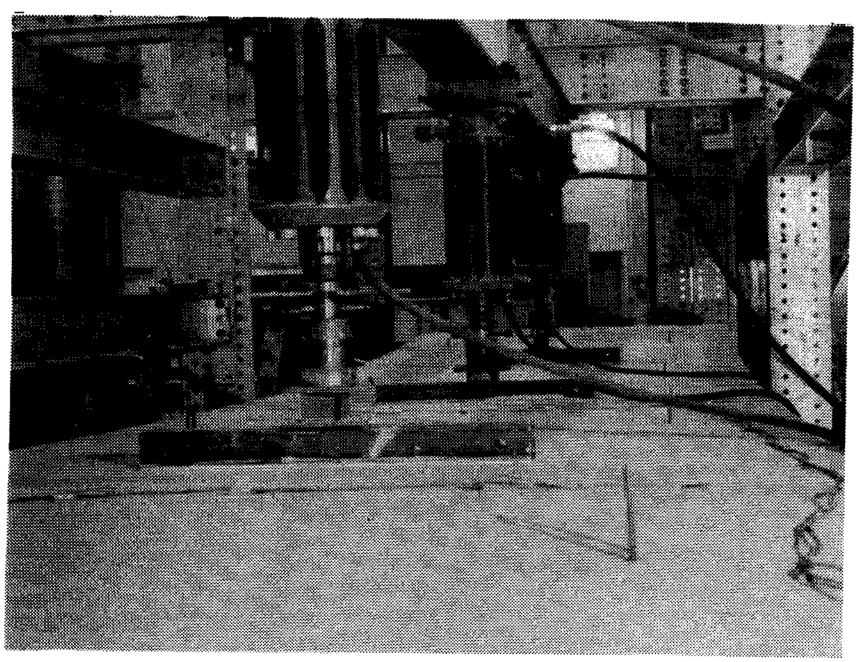


Figure 55 Overall View along Slab of Full-Size Stub-Girder after Failure (Notice amount of deflection)

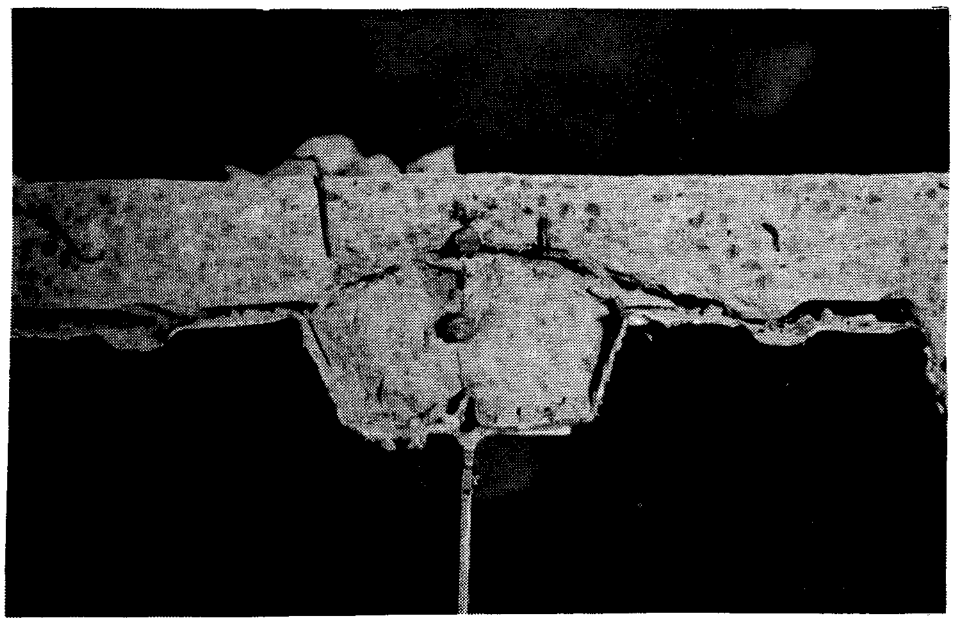


Figure 56 Shear Failure Plane in Concrete Slab of Full-Size Stub-Girder, Directly above South Interior Stub

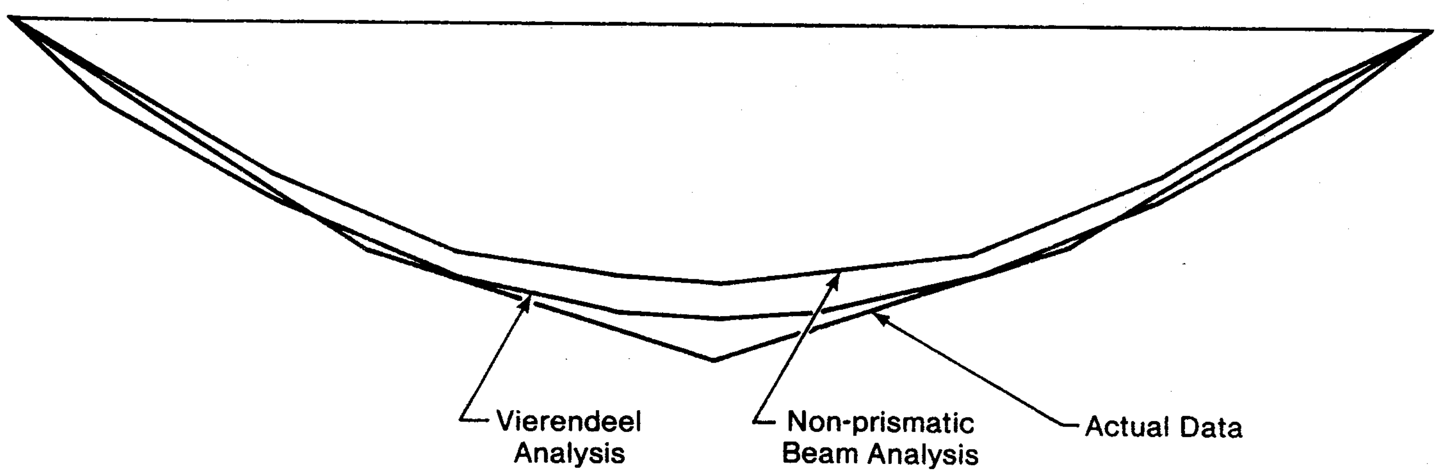


Figure 57 Comparison of Theoretical and Measured Deflections of the Full-Size Stub-Girder

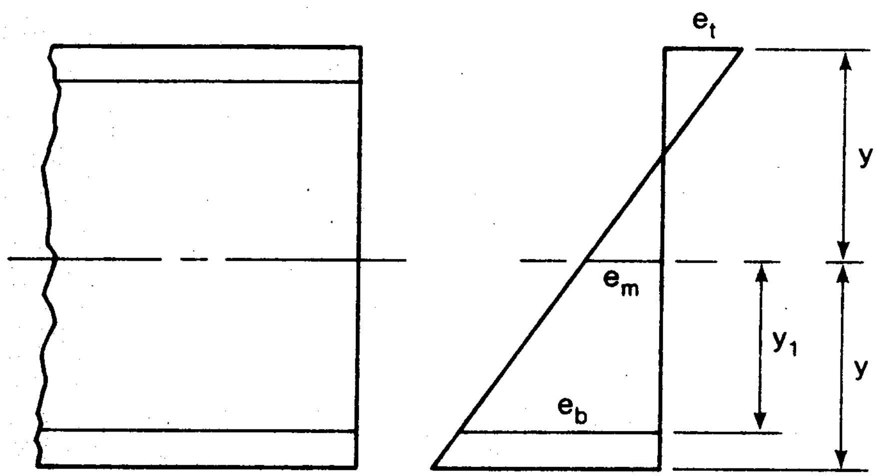


Figure 58 Designation of Cross-Sectional Strains, for Use in Stress Resultant Computations

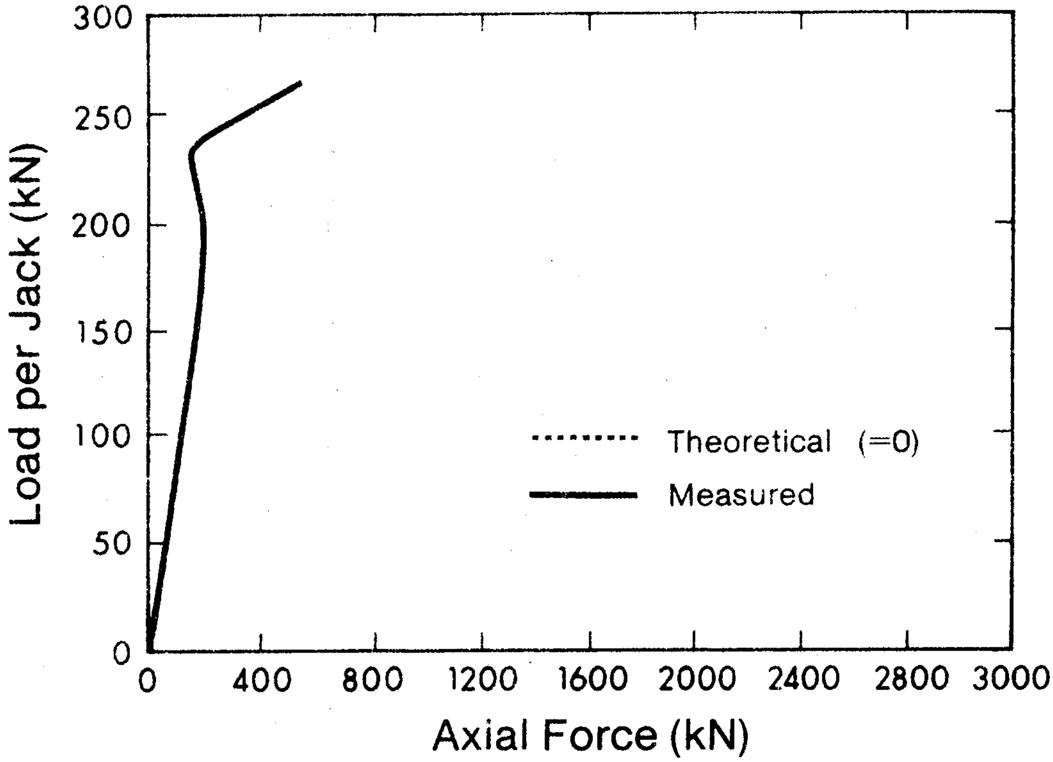


Figure 59 Comparison of Theoretical and Measured Axial Forces at Section 1 (Exterior End of Exterior Stub) of Bottom Chord of Full-Size Stub-Girder

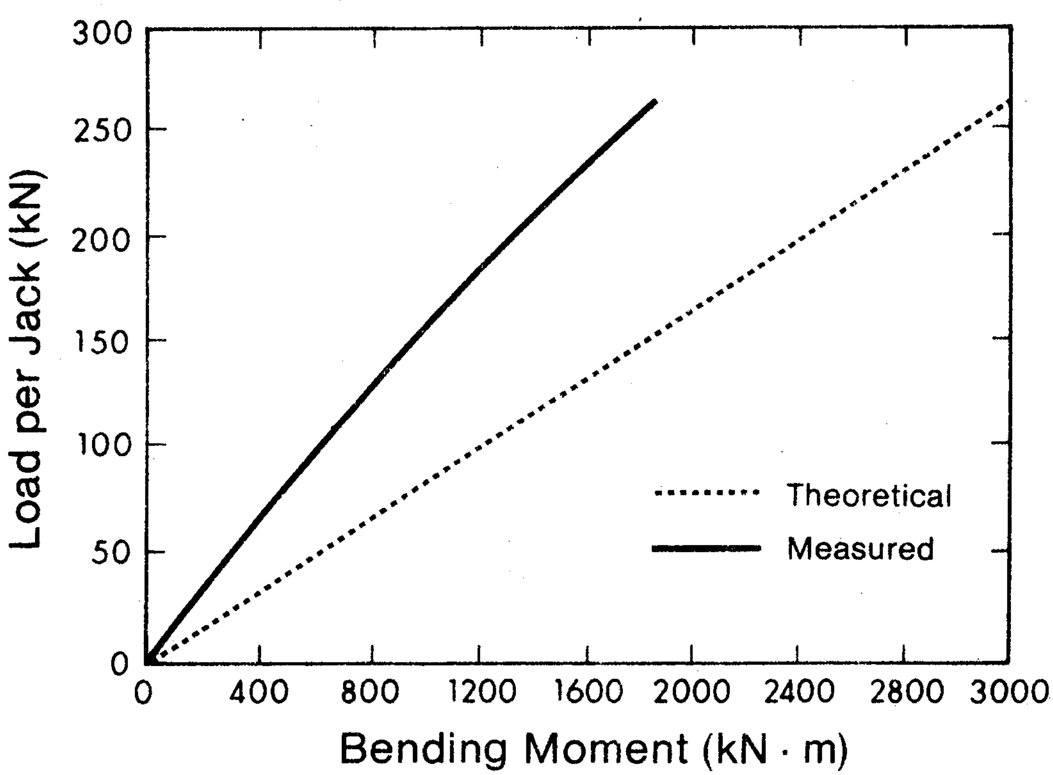


Figure 60 Comparison of Theoretical and Measured Bending Moments at Section 1 of Bottom Chord

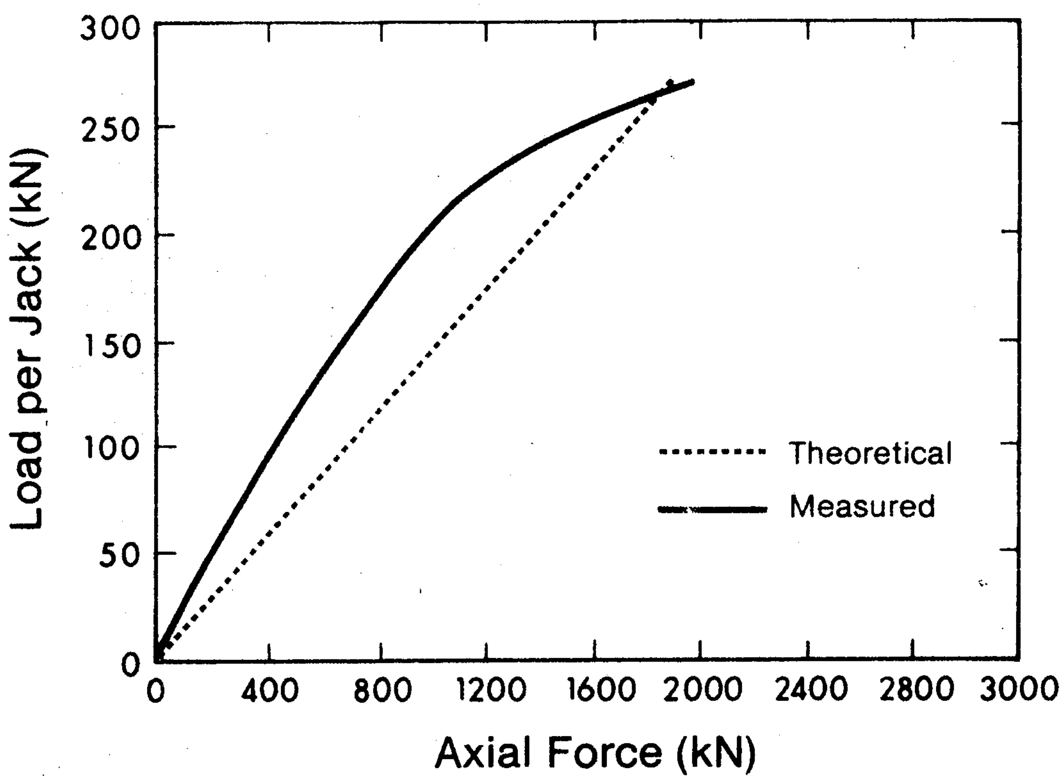


Figure 61 Comparison of Theoretical and Measured Axial Forces at Section 2 (Interior End of Exterior Stub) of Bottom Chord of Full-Size Stub-Girder

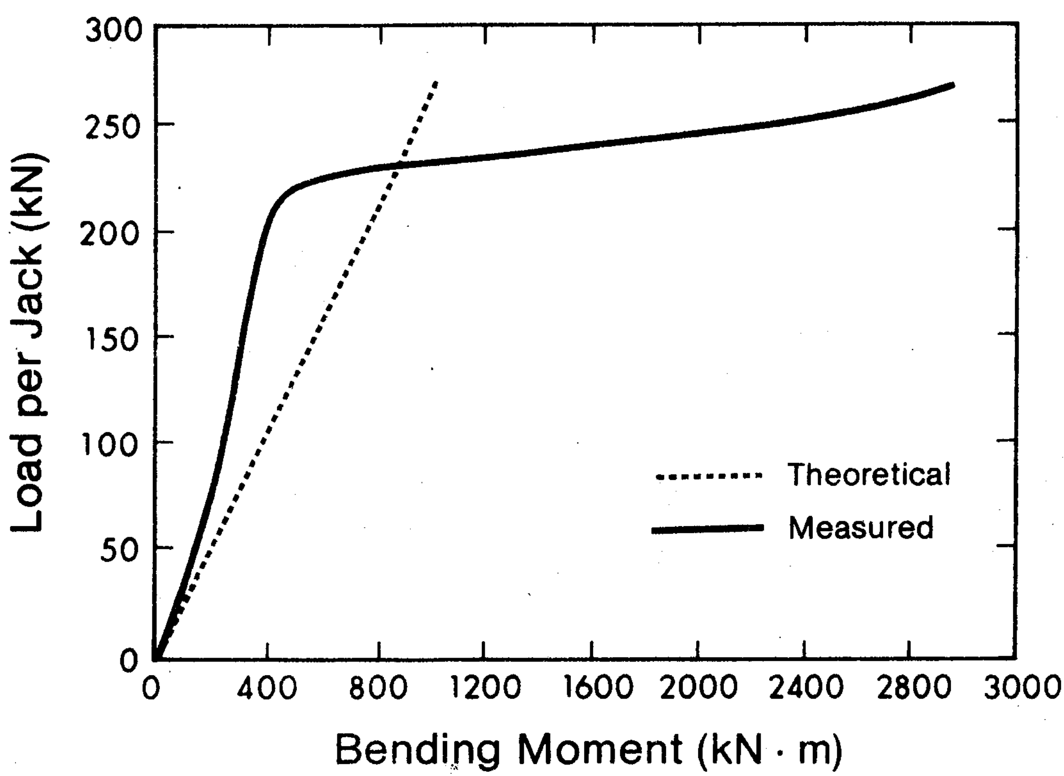


Figure 62 Comparison of Theoretical and Measured Bending Moments at Section 2 of Bottom Chord

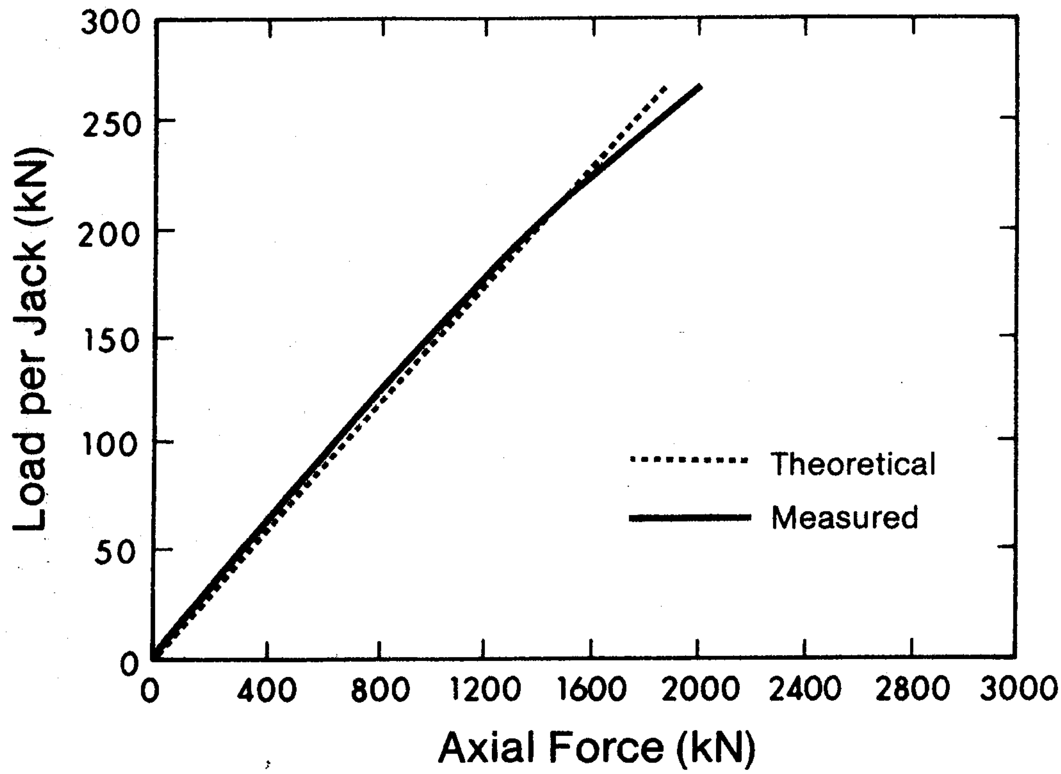


Figure 63 Comparison of Theoretical and Measured Axial Forces at Section 3 (At Floot Beam Location, Between Exterior and Interior Stub) of Bottom Chord of Full-Size Stub-Girder

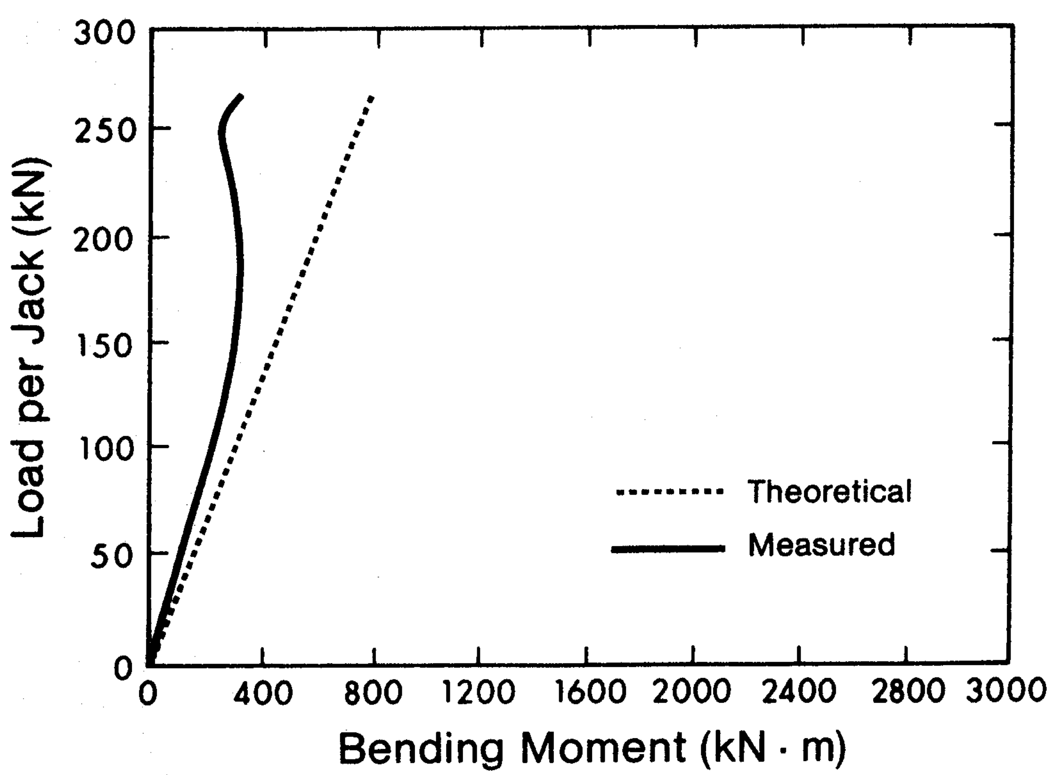


Figure 64 Comparison of Theoretical and Measured Bending Moments at Section 3 of Bottom Chord

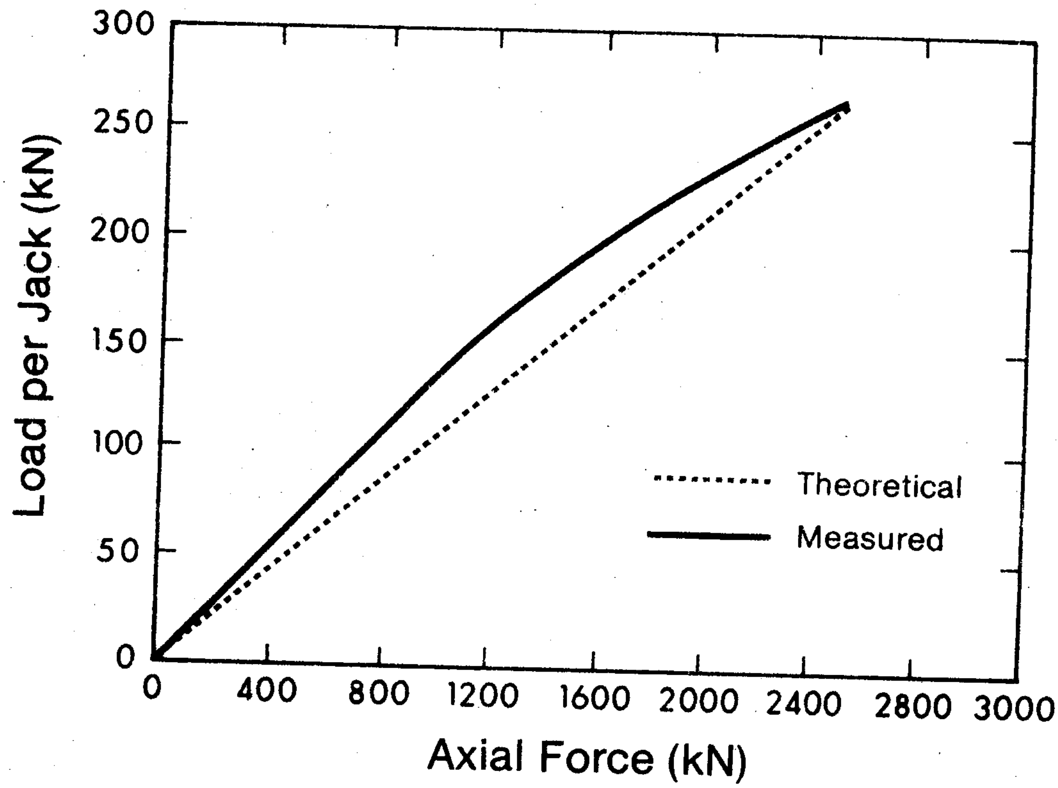


Figure 67 Comparison of Theoretical and Measured Axial Forces at Section 5 (Interior End of Interior Stub) of Bottom Chord of Full-Size Stub-Girder

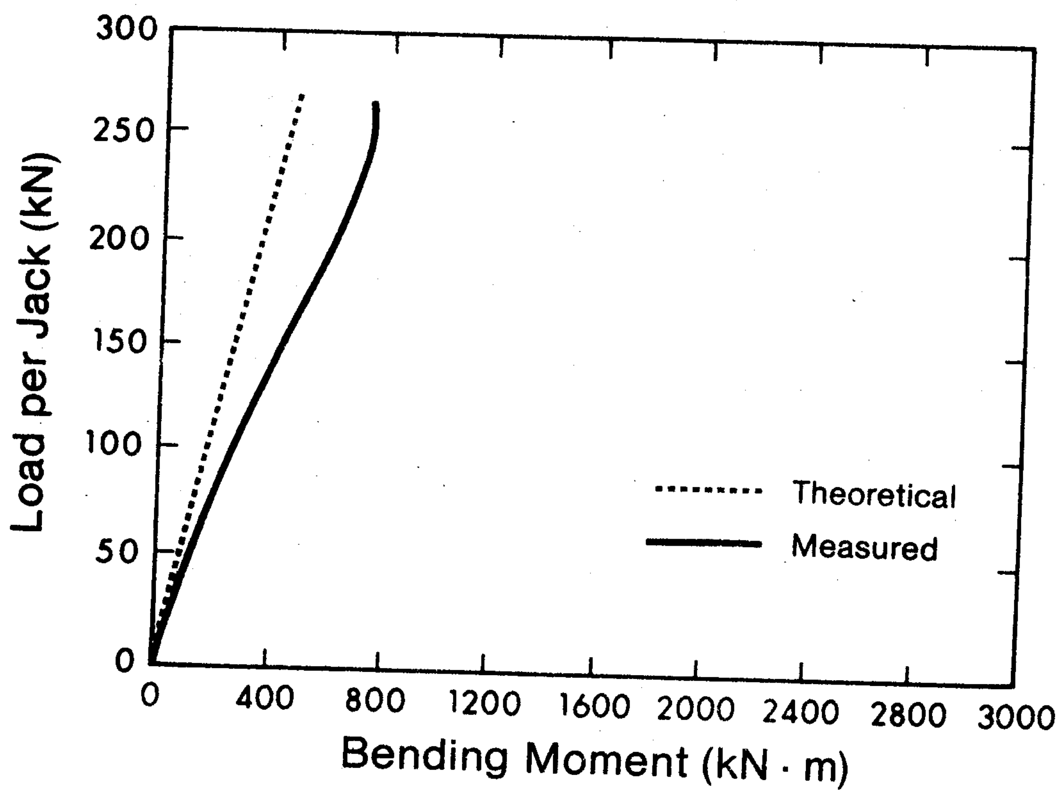


Figure 68 Comparison of Theoretical and Measured Bending Moments at Section 5 of Bottom Chord

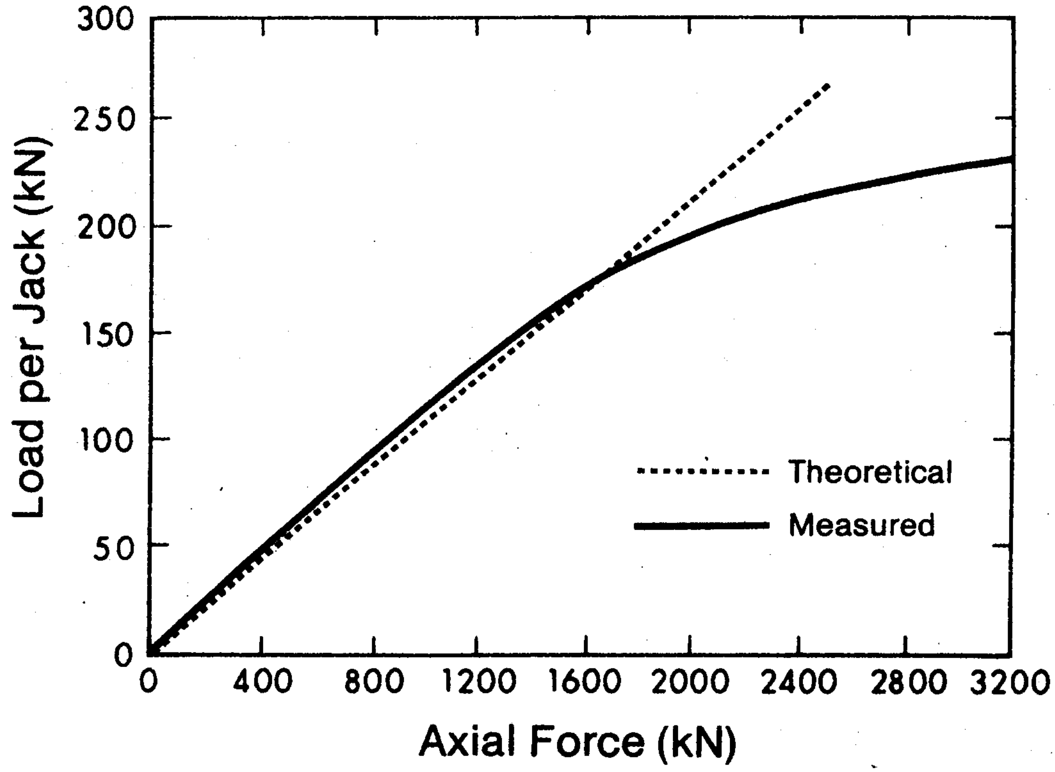


Figure 69 Comparison of Theoretical and Measured Axial Forces at Section 6 (Midspan) of Bottom Chord of Full-Size Stub-Girder

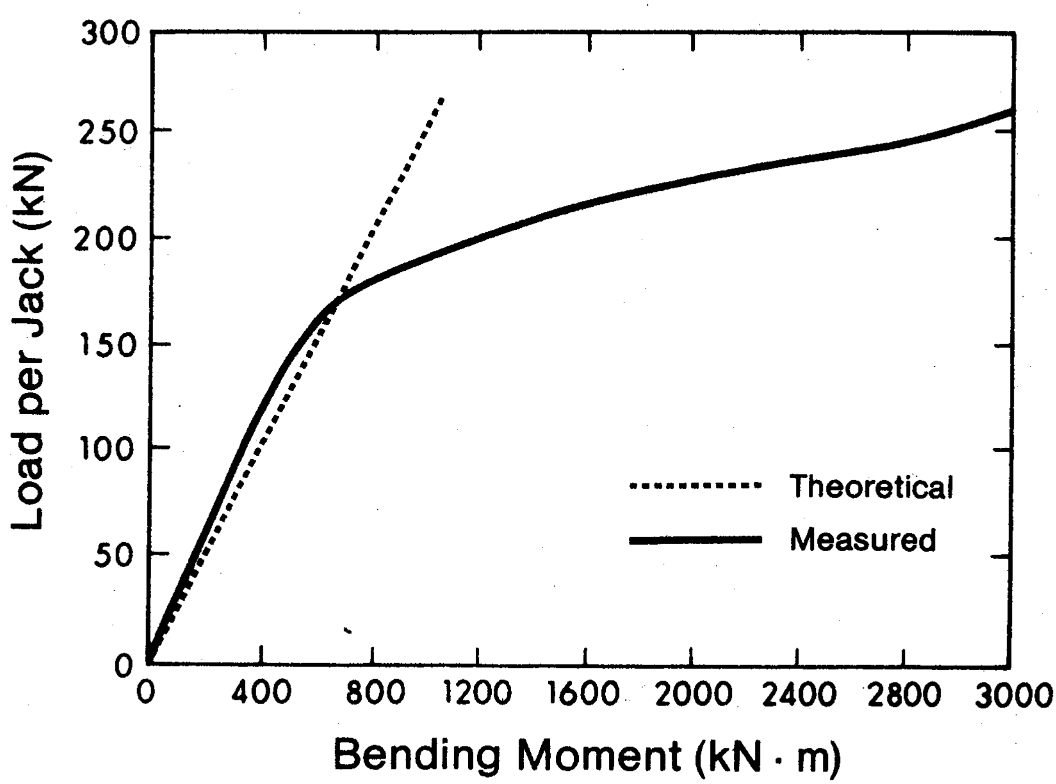


Figure 70 Comparison of Theoretical and Measured Bending Moments at Section 6 of Bottom Chord

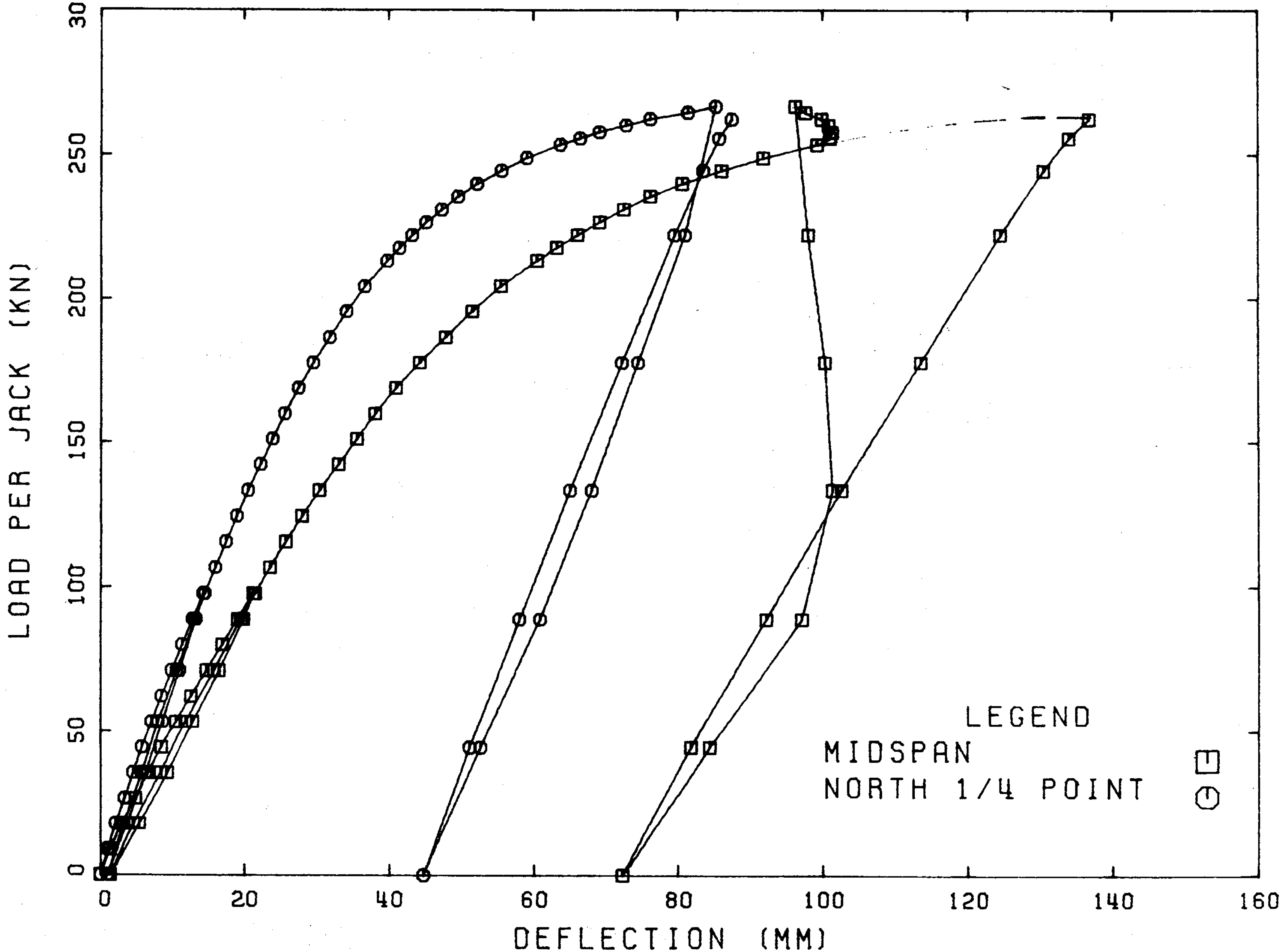


Figure 71 Load-Deflection Curves for the Full-Size Stub-Girder

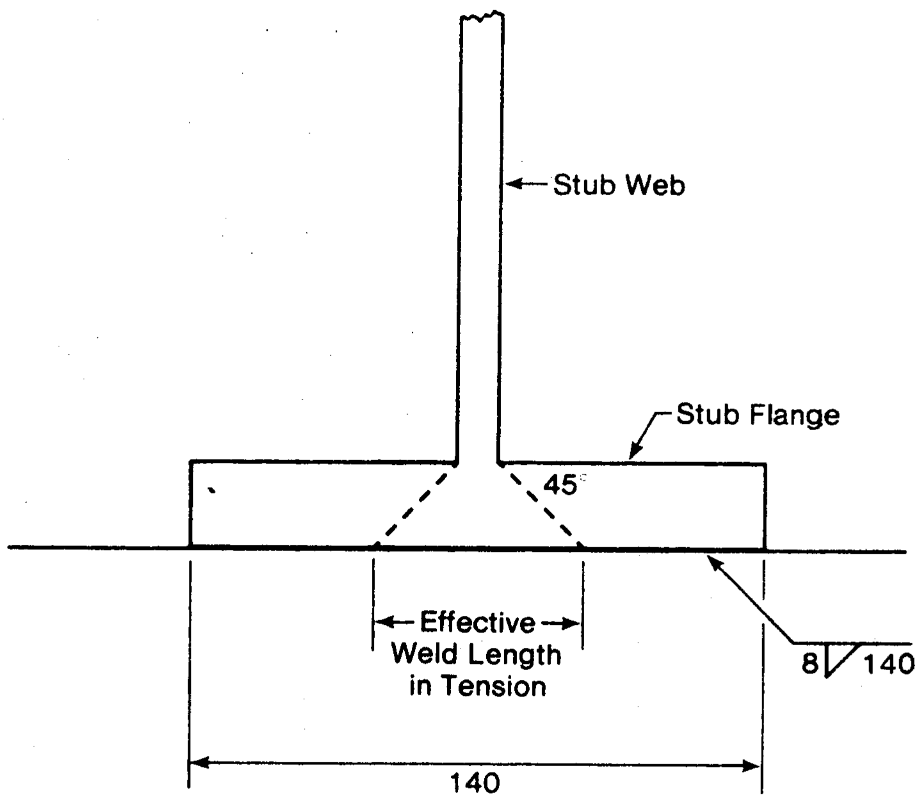


Figure 72 Effective Length of a Transverse Stub-to-Chord Weld