Strengthening of Simple Span and Continuous Span Plate Girder Bridges by Post-tensioning

Parmjit S. Parmar

A Thesis

in

The Department

of

Civil Engineering

Presented in Partial Fulfillment of the Requirements
for the Degree of Master of Engineering at
Concordia University
Montréal, Québec, Canada

June 1987

ISBN 0-315-37087-4

ABSTRACT

Strengthening of Simple Span and Continuous Span Plate Girder Bridges

by Post-tensioning

Parmjit S. Parmar

The strengthening of existing bridges by post-tensioning is one of the good methods from structural efficiency and economy. Both simple span and continuous span plate girders can be strengthened by post-tensioning .

The analysis has been done by the influence coefficient method. The theoretical results have been satisfactorily confirmed by experimental observations. Simple span and continuous span bridges can be strengthened up to 10 to 14 percent of their allowable capacity. The percentage increase in strength can be more if the eccentricity of the cables is more with respect to the kern of the section.

Straight cable configuration has been recommended for use on existing bridges.The drapped cable configuration has also been used for testing, but relatively small local stresses were observed near the pulley axle. The value of local stresses for bottom flange (near the pulley axle) is 15 percent of induced stress. The local stresses can be controlled to a minimum by providing guides of bigger radius, so that the stresses are distributed on a large area and thus will not give concentration of stresses at one point.

For the continuous beam drapped cables were used, but they caused loss of pre-stressing due to friction. However, a wax and steel stirrup arrangement was used to overcome friction. For this reason, for a continuous beam, straight cable configuration is recommended when used on existing bridges.

ACKNOWLEDGEMENT

The author wishes to express his gratitude to the thesis supervisor, Dr. M. S. Troitsky, Professor in Civil Engineering Department, for suggesting the topic, his continued interest, advice and guidance in the course of this investigation.

The assistance provided by Dr. Z. A. Zielinski and Dr. K. Ghavami for reviewing the thesis as well as cooperation offered during author's studies is highly appreciated.

The financial support provided by NSERC is gratefully acknowledged.

The author also extends his gratitude to Yvon Pichette, engineer of BBR Quebec Inc., for supplying the post-tensioning equipment and necessary help from time to time. Thanks is also due to Danny Roy for his help in the Structures Laboratory to complete the experiment.

Finally, I appreciate the patience of my family during the course of my studies.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATIONS

Notation	Description
$A, \bar{A}$	= Area of cross-section of girder and ideal model beam, respectively ;
$\bar{A}_a$	= Actual area of cross-section of the model beam ;
$\bar{c}_t , \bar{c}_b$	= Distance of center of gravity from top and bottom for model beam, respectively;
$d, \bar{d}$	= Depth of prototype girder and model beam , respectively;
$\bar{d}_a$	= Actual depth of model beam;
e, e_1	= Eccentricity of cables from bottom and top, respectively;
E, E_c	= Modulus of elasticity for structural steel and high strength steel, respectively;
f	= Stress at top or bottom in prototype girder;
$f_{11}..f_{1n}$	= Flexibility coefficients for the girder or the beam;
$I, \bar{I}$	= Ideal moment of inertia for prototype and model beam , respectively;
$\bar{I}_a$	= Actual moment of inertia of model beam;
K	= Reduction factor;
L	= Span of the girder or beam;
L_1	= Distance from support to the bent of cable;
m_o	= Moment diagram for released beam due to dead load or live load;

$m_1..m_n$	=	Moment diagram for 1,2, .., n unit loads respectively;
n_B	=	Axial force diagram due to pre-stress force in beam;
n_C	=	Axial force diagram due to pre-stress in cable;
$n_1,..n_n$	=	Axial force diagram for 1,2, .., n unit loads respectively;
$P, \overline{P}$	=	Pre-stress force in prototype girder and model beam, respectively;
ΔP	=	Total increment pre-stressing force;
$\Delta P_{DL}, \Delta P_{LL}$	=	Increment of pre-stressing force due to dead load and live load respectively;
q	=	Dead load per unit length;
$Q, \overline{Q}$	=	Concentrated load in prototype girder and model beam, respectively;
$S, \overline{S}$	=	Sectional modulus of prototype girder and model beam, respectively;
s	=	Shearing force;
t	=	Thickness of web plate;
u	=	Strain energy;
u_{1D}	=	Flexibility coefficients for dead load;
u_{1L}	=	Flexibility coefficients for live load;
w, w	=	Deflection for prototype girder and model beam, respectively;
$x_1,..x_n$	=	Redundants in the beam or girder;
x,y,z	=	Coordinate system;
α	=	Angle of bent of cable;
β	=	Numerical factor for local stresses;
y	=	Distance from the center of gravity to top or bottom fibre;

Y	= Reduction factor for load;
(η,ξ)	= Direction coordinates;
σ_x, σ_y	= Stress in x - and y - directions, respectively;
τ_{xy}	= Shear stress;
$\bar{\varepsilon}_t$, $\bar{\varepsilon}_b$	= Ideal strain at top and bottom for model beam, respectively;
$\bar{\varepsilon}_{ta}$, $\bar{\varepsilon}_{ba}$	= Actual strain at top and bottom for model beam, respectively;
ε_t, ε_b	= Ideal strain at top and bottom for prototype girder, respectively ;
μ	= Poisson's ratio.

CHAPTER 1

1.0 INTRODUCTION

1.1 HISTORICAL REVIEW

Strengthening of plate girder bridges by post-tensioning is not new, but due to lack of knowledge their applications are overlooked in this area. In Europe, considerable research and practical application has been done in this area [12]. In North America, however, some bridges have been strengthened by post-tensioning [4]. Recently, some research has been done at Iowa State University related to strengthening of single span composite plate girder bridges by post-tensioning and its applications to existing bridges have been shown feasible and economical [4].

In Canada, three bridges have been strengthened by post-tensioning, Lewes, Yukon river bridge on Alaska highway [1], Welland Canal bridge in Ontario [9] and Champlain bridge in Montreal (under repair). Post-tensioning method for strengthening bridges require more research to be done in order to make it more understandable and economical.

The scope of this thesis is study of strengthening of plate girder bridges (simple span and continuous span) by post-tensioning.

1.2 REASONS FOR STRENGTHENING OF BRIDGES

a) A bridge that is to be strengthened to carry heavier vehicles than it was originally designed to carry. Sometimes after the construction of the bridge, due to change in load specifications, the heavier vehicles have to pass on the bridge and its members need strengthening.

b) A bridge that is to be widened to carry additional lanes of traffic.

c) A bridge having deteriorated concrete deck, which is to be replaced by new concrete deck which may overstress the structural members.

d) In case of accidents which affect the bridge superstructure, bridge members may require strengthening.

e) Strengthening of bridges which are preserved as part of historic and cultural heritage of a country.

1.3 TYPES OF STRENGTHENING TECHNIQUES

Basically there are two methods of strengthening bridges and each method is discussed seperately as follows :

1.3.1 Conventional Methods :

The conventional methods of strengthening include;

a) Cutting out the cracked material and adding cover plates to the top and bottom flanges of members carrying deck slab to increase the load carrying capacity of existing bridge.

b) Strengthening of connection by introducing stiffeners.

c) By increasing the cross-section of the structural members for new requirements by connecting an appropriate structural member to bottom flange of existing member [8] .

d) Increasing the load carrying capacity by adding shear connectors on top flange of girder to make the section composite.

1.3.1.1 Advantages

a) Conventional methods of strengthening bridges are well understood.

b) These methods require less supervision in the field.

1.3.1.2 Disadvantages

a) Expensive and involves difficulties to carry out strengthening operations. For example, to weld plates to the flanges of girder, heavy framing is required.

b) Reduce the number of lanes temporarily and interrupt the traffic, which is a very serious problem in areas where there is a heavy traffic.

c) Excessive field welding makes the sections too heavy and in cases where the specified clearance underneath the bridge is required, these methods cannot be applied.

d) Demolishing a part of deck slab while maintaining the partial traffic flow on the other lanes, requires adequate traffic facilities such as heavy fixed barricades bolted to the structure to seperate traffic from the work area. These facilities can exceed the total cost of alteration of structural members to be strengthened.

e) When welding the plates to the upper or lower flange of girder in case of plate girder bridges, the girder must be jacked at appropriate points to relieve the stresses of dead load of structure. If the girder is not stress relieved, the added plates will be stressed under live loads only, while existing structure will be stressed under live loads and dead loads. This will result in flanges which are heavy, uneconomical and impossible to provide, from practical point of view. Jacking plate girder is a complicated operation which adds to the cost and sometimes it is impossible to achieve these conditions.

f) The demolished portion of the deck slab, when altering the flanges, has to be restored. The joint which is made to the existing slab can become source of water seepage and can cause excessive corrosion to the steel members of the bridge.

1.3.2 Post -tensioning Methods

The strengthening of plate girder bridges by means of post-tensioning appears to be very favourable from structural, engineering and economic point of view. The application for post-tensioning high strength steel cables, tie rods, offer a wide range of solution for strengthening of plate girder bridges. The fact that post-tensioning can offer new and more economical way of reinforcing existing bridges, usually, is overlooked mainly due to lack of knowledge and experience with such methods.

Various methods of post-tensioning are successfully applied for strengthening of existing short and medium span bridges. Some bridges successfully strengthened by post-tensioning are discussed later in this chapter.

Compartively less research has been done toward developing economical methods for increasing load carrying capacity of plate girder bridges by post-tensioning.

The purpose of post-tensioning steel structures is different than that of concrete. Concrete is a structural material that has a high compressive strength but relatively low value in tension. Post-tensioning concrete is therefore done to induce compressive stresses to the member either by axially or combined axial and bending components in such a way that when design load is applied, the entire element remains in compression or sometimes in tension up to allowable value, which is small as compared compression.

But this is not the case in steel because it has equal value in tension and compression. The purpose of post-tensioning steel is to try to reduce the tensile stress caused by design loads. In other words, it is to induce a stress distribution in steel members similar but opposite in nature to those of design loads. The theoretical background of post-tensioning techniques applied to plate girder is explained in Chapter 3.

1.3.2.1 Advantages

a) The strengthening operation generally does not affect the flow of traffic and thus it is of great advantage over the conventional methods.

b) The post-tensioning cables can be placed and anchored in any configuration making the post-tensioning particularly suitable for strengthening existing bridges.

c) It does not require heavy formwork and jacking to lift the girders to relieve the stresses, as required in the case of conventional methods.

d) It eliminates the excessive field welding and splices which is very expensive and complicated to perform in the field.

e) It is structurally more efficient as compared to other methods.

1.3.2.2 Disadvantages

a) It can affect the lateral instability of the post-tensioned members.

b) It requires additional computations as compared with the conventional methods, however, it is always the case of advanced structural design.

c) It requires special arrangements for anchoring of tendons.

1.4 EXISTING BRIDGES STRENGTHENED BY POST-TENSIONING

Three bridges strengthened by post-tensioning are discussed in brief as follows:

1.4.4 Lewis, Yukon River Bridge [1]

The Lewis, Yukon river bridge is located at 897.5m of Alaska highway approximately 20 miles south of the city of Whitehorse. The bridge is composed of two 250 ft. skewed through Warren truss spans, was fabricated in USA in 1920 and was erected in 1955. The bridge was damaged by vehicle.The damage locations are shown in Fig. 1.1.

The fabricated bottom chord of truss was strengthened by providing four 1.25 inch diameter high strength bars of strength 150 ksi. Fig. 1.2 shows the exact location of tensioning bars.

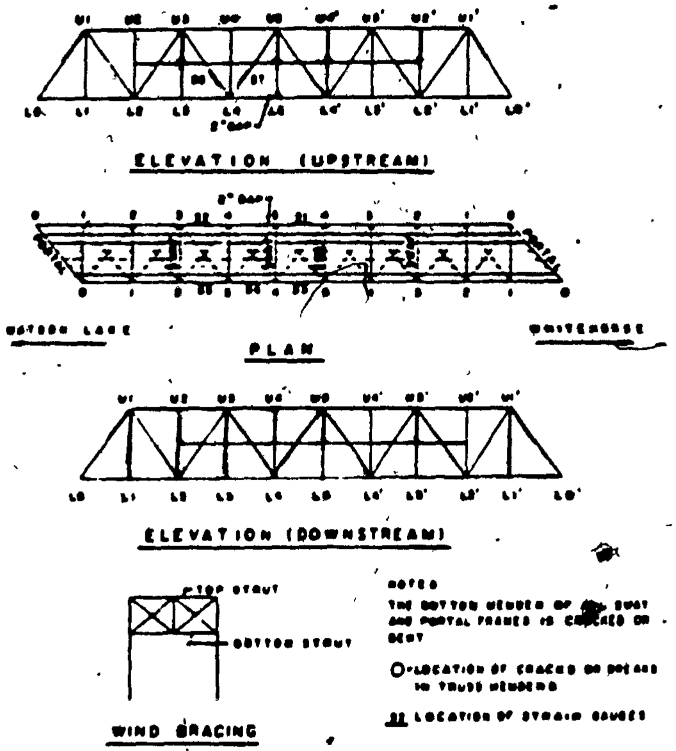


Fig. 1.1 Location of Damage and Strain Gauges.

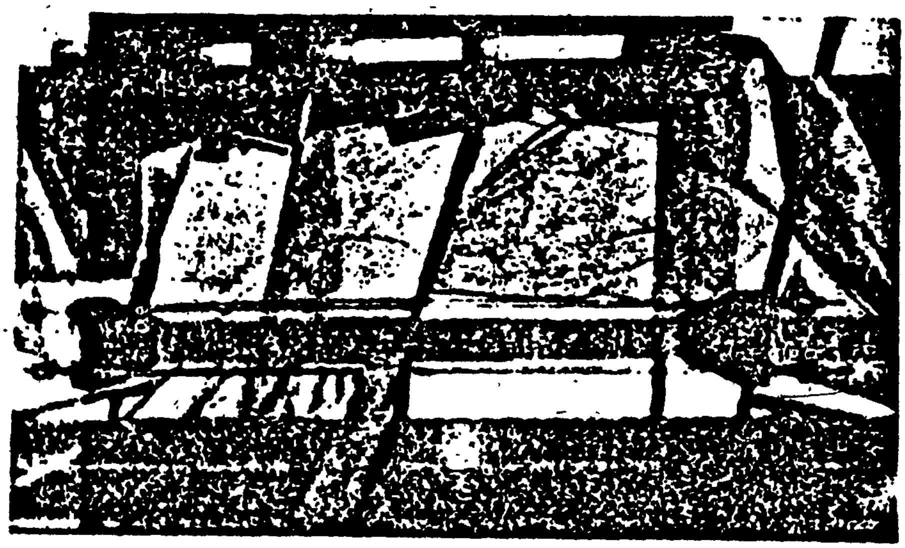


Fig. 1.2 Brackets and Dywidag Bars Used for Jacking.

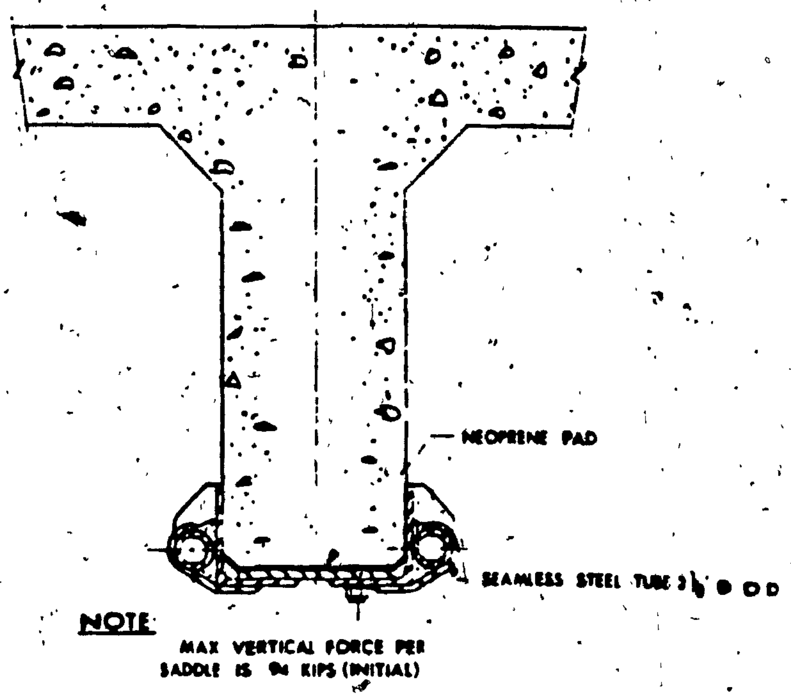


Fig. 1.3 Special Saddle Transmit Vertical Load To Mid-Span of Bridge Beams.

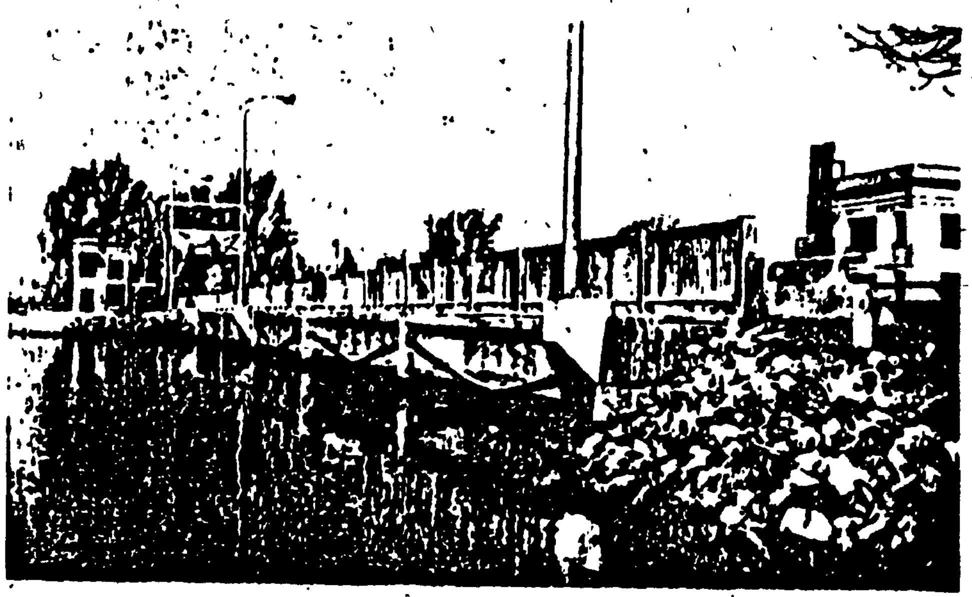


Fig. 1.4 View of Rehabilitated Bridge Over Welland Canal.

The jacking of fractured chord is an innovative application of post-tensioning method. The rehabilitation done in this project has been proven to be feasible, economical and expedient.

1.4.2 Welland Canal Bridge [9]

The bridge under consideration is a concrete bridge over the Welland canal in Ontario. The bridge was built in 1920 and following an inspection in 1965, some cracks and spalling of concrete from beams require immediate strengthening. Converting the existing simple spans to continuous structure by means of external post-tensioning was considered best for rehabilitation of this bridge. The bridge after rehabilitation is shown in Fig. 1.3. The external post-tensioning was done using Freyssinet cables of 0.915 inches diameter. The saddles were designed to hold the cables at appropriate locations to transmit load to main carrying beams.

The results obtained by strengthening Welland canal bridge by post-tensioning were excellent. The time consumed and cost was less than the other proposed strengthening techniques.

1.4.3 Champlain Bridge

Champlain bridge is located at Montreal, Quebec. The main spans are of trusses and the approaching spans are of pre-stressed concrete supporting the deck. The external concrete beams due to atmospheric action are so much deteriorated that the pre-stressing wires can be seen clearly. So the repair (now underway) is being done by external post-tensioning to restore the same pre-stress to the existing beams. Figs. 1.5 to 1.10 exhibit the rehabilitation phase which are now in progress.

Fig. 1.5 Anchor Block Unit.

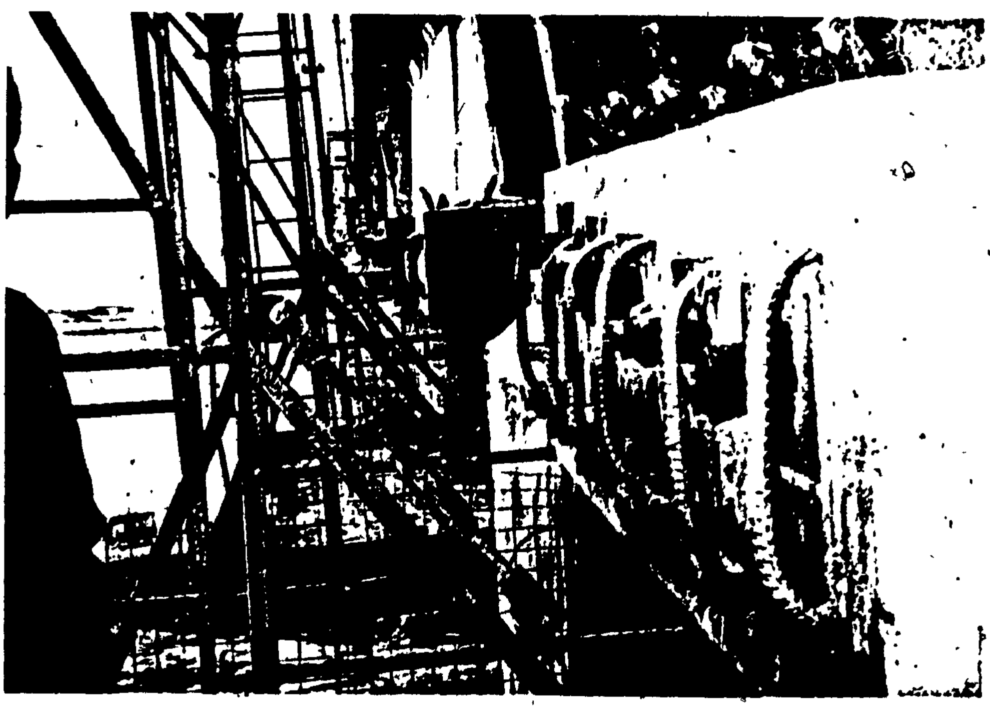


Fig. 1.6 Details of Anchor Block Reinforcement.

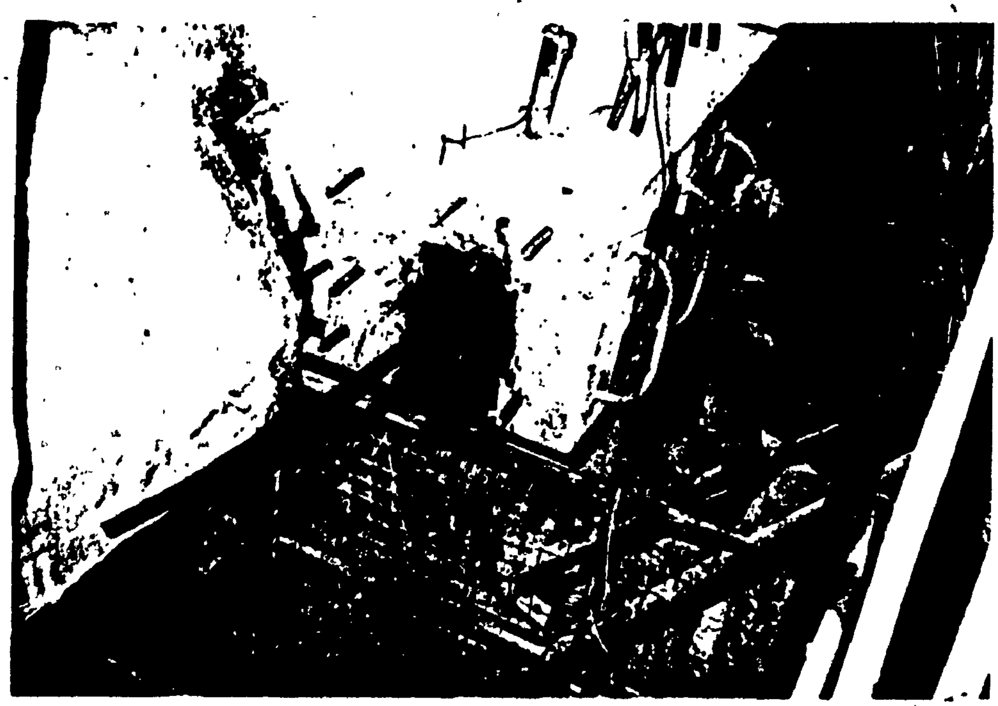


Fig. 1.7 Anchor Plate Details.

Fig. 1.8 Longitudinal View of Post-tensioned Beam.

Fig. 1.9 Longitudinal View of Supporting System for Cables.

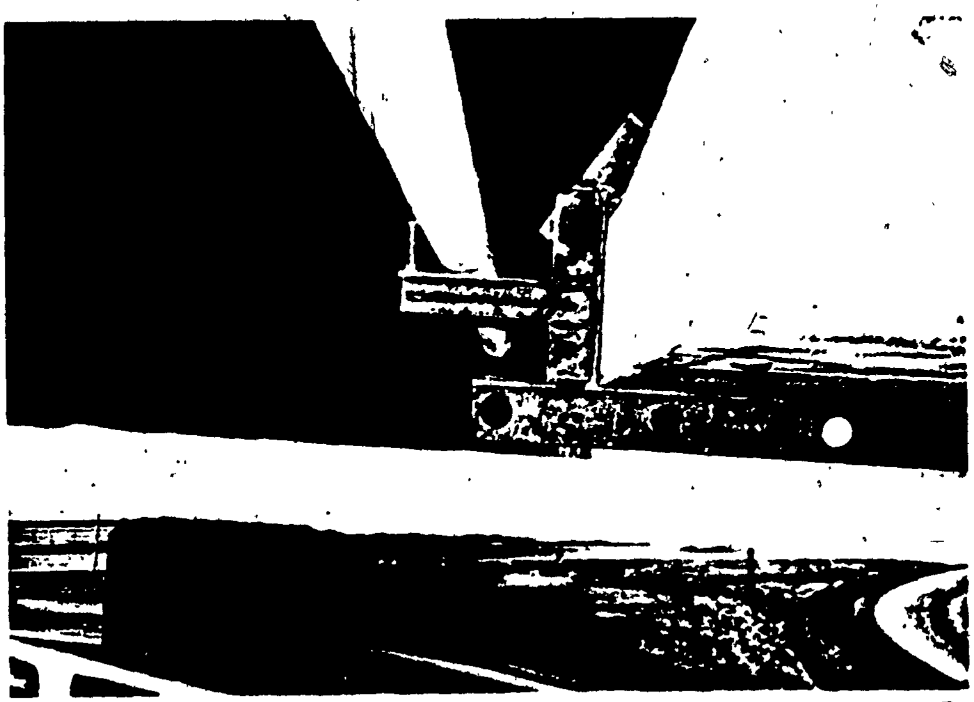


Fig. 1.10 Details of Supporting Unit

CHAPTER 2

2.0 INSPECTION AND ASSESSMENT OF BRIDGES BEFORE STRENGTHENING

2.1 INTRODUCTION

The inspection and assessment of bridge structure before applying any strengthening technique is necessary. All members should be inspected carefully by detailed visual studies. An accurate assessment of safe load carrying capacity of any existing structure is a complex and difficult task. It is even more complex in the case of bridge structure that generally suffer adverse environments of dynamic loading and climatic conditions. Problem is even worse if information such as design details, contract documents, construction records, past performance of the structure and maintenance history is not available or lacking.

The evaluation of the strength of existing bridge member has to be calculated prior to rehabilitation of the structure. Existing standards and guidelines do not provide satisfactory results [10]to undertake realistic assessment and evaluation of some common type of bridges existing throughout Canada. Different evaluation techniques give different results. The intention of this thesis is not to deal with rating of bridge, but some important steps should be followed before strengthening.

2.2 INSPECTION

Various publications already exist regarding the strategy, technique, procedure and documentation of inspection for various types of bridges. However, from structural considerations, the inspection procedure should be specifically directed as follows [10] :

a) Stability :- Undermining of foundation alignment and distortion of compression members, bearing characteristics of main load carrying members, main bracing elements transferring longitudinal and lateral forces to foundations etc. This type of inspection requires visual inspection, except, when detailed investigation are desired.

b) Deterioration :- Loss of strength of bridge components due to deterioration requires a detailed visual study and assessment of loss of material at critical locations of the structure such as flange at midspan of flexural ~~steel~~ members, ends of coverplates, stiffeners, web at support, concrete at both top and bottom of deck slab, pier caps, abutement seats, bearing, loss of materials at bent bases and loosening of connections etc.

c) Durability :- Non-durability may affect long term perfomance of the structure if left unchecked. The cracks which can occur due to these causes can generate deflections and vibrations which will affect the behaviour of the structure.

2.3 CAUSES OF DETERIORATIONS

A qualitative diagnosis of possible causes may be helpful in planning an effective program for strength evaluation and repair. Failure to understand the cause of any defects may lead to inaccurate assessment. Spalling of concrete sections subjected to de-icing chemicals, due to inadequate cover to rebars, vibrations due to loss of bond between deck slab and supporting units, damage to protruding steel sections due to impact of moving vehicles are readily identifiable causes which can affect the behaviour of the members. Fatigue crack, tearing in steel members etc., can be related to poor detailing and construction practices as well as overloading of bridges.

2.4 EVALUATION OF STRENGTH

Evaluating the true load carrying capacity of an existing structure, whether deteriorated or not, is a major engineeing problem involving material technology in addition to structural design.

Cases where some members have deteriorated and distortion to a degree that strength is obviously inadequate, offers no problem. Problem arises only at instances where there is some question as to the adequacy of strength. It is an important step, before the implementation of any rehabilitation method, to evaluate each member correctly.

The evaluation can be carried out in three different ways as follows:

a) Analytical study :- The existing bridge structure should be analysed according to the codes of practice existing in each country for maintenance and inspection of bridges. If the bridge capacity is adequate to carry maximum load requirements, no further consideration need to be given other than normal inspection and regulation of load limits. But if the bridge capacity is inadequate then detailed analysis by refined analysis techniques should be done. The refined analysis may not be justified in design of a new bridge (as small saving of material as compared to engineering labor put for analysis), but for repair purposes, some reserve of strength can mean the difference between repair and no repair.

b) Laboratory testing of materials :- To obtain realistic assessment of properties of existing materials, as affected by decaying problems of corrosion, salt penetration etc., test coupons and cores would have to be taken from various parts of the structure for laboratory tests. Laboratory tests form an important part of assessment and evaluation program. They not only do evaluation of properties of materials, but also assist in implementation of proper strengthening procedures. For example, if ordinary welding techniques are attempted as means of repair of non-weldable type of steel members, the rehabilitated units may reveal in short time signs of damage due to fracture. Moreover, the theoretical computations are made on the basis of an idealized behaviour of the material and it is essential to verify that such behaviour exists in materials subjected to long term conditions of weather and loading cycles. Test samples should be representative of structure and be so chosen from consideration of safety of individual member or structure as a whole due to removal of the material. CSA and ASTM provide some guidelines with regard to sampling procedure.

The laboratory tests should reveal the important properties of steel such as chemical and mechanical properties, welding properties, behaviour of existing material against brittle fracture and fatigue. Similarly, tests for the deck slab should reveal the compressive strength, chemical contents and properties of reinforcing bars.

c) Field tests:- The analytical approach may not provide all answer, in such cases, the field testing may provide reasonable results. Jorgenson and Larson[10] indicated that a reinforced bridge designed for HS 20 based on working strength design (WSD), requires eight HS 20 trucks to provide any permanent deflection and twenty HS 20 trucks to cause collapse, however, Kinner and Barton [10] show comparable results between tests and analytical work carried out using the finite element method.

So the conclusion is that the more refined the analysis technique is, the better is the correlation between test results and analytical value. Unless the analytical work can accurately reflect the prototype unit, load test will tend to show much greater strength than computed.

2.5 REMARKS

a) The inspection of bridge superstructure is done by detailed visual studies. The corroded members should be checked in detail.

b) The adequacy of strength near anchorage points should be checked in detail, because there is high concentration of stresses due to post-tensioning force.

c) The samples taken for laboratory testing should be taken from the members which are corroded and from the least stresses zone.

d) The analysis of structure should be done using refined techniques because inherent non-homogenuity of materials accentuated by varying degree of deterioration, in different parts of member, can be considered.

e) The non-linear analysis should be done where distortions and deformations of the existing members are permanent and require preservation of existing structures. The final evaluation of the member strength should be based on both analytical and field tests.

f) While calculating the adequacy of strength for existing members of the structure, allowance should be made for deterioration by introducing deterioration factor, as described in details in reference [10].

CHAPTER 3

3.0 METHOD OF STRUCTURAL ANALYSIS

3.1 INTRODUCTION

The analysis of simple span and continuous span plate girder is explained in this chapter. The influence coefficient method has been considered as most efficient method to solve such types of problems, because the results can be easily programmed for personal computer.

3.2 LITERATURE REVIEW

After extensive literature review, it has been observed that the analysis of simple span and continuous span plate girder bridges, by influence coefficients, has not been done in details. However, for simple span composite bridges, finite element analysis has been recently done[4]. The influence coefficient method has been applied to simple span as well as continuous span beam bridges in this chapter.

3.3 INFLUENCE COEFFICIENT METHOD

The influence coefficient method[7] has been derived from strain energy approach, which, in turn, is defined as the work done per unit volume. So strain energy of the body will be the volume integral of strain energy density;

$$u = \int_S w \, dv \tag{3.1}$$

where,

$$w = \frac{1}{2} \int_S P(\eta, \xi) \, \varepsilon(\eta, \xi) \, dv$$

Here, (η,ξ) = direction coordinates (x,y,z)

Putting equations (3.2) into (3.1),

$$u = \frac{1}{2}\int_S P(\eta,\xi)\, e(\eta,\xi)\, dv \qquad (3.3)$$

The different cases of loads applied are interpreted in terms of strain energy as follows:

a) Direct energy :- In case of normal cross-sectional area A under the action of direct axial force N (both may vary along its length, the stress is given by

$$p_{xx} = \frac{N}{A} \qquad (3.4)$$

$$\text{Strain produced } \varepsilon = \frac{N}{AE} \qquad (3.5)$$

$$\text{Since volume } V = \int_x A\, dx \qquad (3.6)$$

Putting the values of equations (3.4), (3.5) and (3.6) in (3.3) yields,

$$U_{\text{direct}} = \int_X \frac{N^2}{2AE}\, dx \qquad (3.7)$$

b) Flexural energy:- In case of member of moment of inertia I under moment M, both of which may vary along its length.

$$p_{xx} = \frac{My}{I} \qquad (3.8)$$

$$\varepsilon_{xx} = \frac{My}{EI} \qquad (3.9)$$

$$\int_I \frac{y^2}{I}\, dv = x\int dx \qquad (3.10)$$

Putting equations (3.8), (3.9) and (3.10) in (3.3) yields

$$u\,(\text{bending}) = \int_X \frac{M^2}{2EI}\,dx \tag{3.11}$$

c) Shear energy :- Assume a member of cross-section A is acted upon by a shear force S parallel to the direction of the coordinate Y and assume also that the shear stress distribution is uniform across the section,

$$p(x,y) = \frac{S}{A} \tag{3.12}$$

$$\varepsilon\,(x,y) = \frac{S}{CA} \tag{3.13}$$

Thus as in the case of direct thrust, we may write,

$$u' = \int_x \frac{S^2}{2CA} \tag{3.14}$$

But the shear stress is not uniform along the length of the beam, so it can be written;

$$u\,(\text{shear}) = K \int_x \frac{S^2}{2CA}\,dx \tag{3.15}$$

In the present discussion, torsion is not taken into consideration. In summary, the direct, bending and shear stresses are put in terms of strain energy as follows :

$$u(\text{direct}) = \int_x \frac{N^2}{2AE}\,dx \tag{3.16a}$$

$$u(\text{bending}) = \int_x \frac{M^2}{2EI}\,dx \tag{3.16b}$$

$$u(\text{shear}) = K \int_x \frac{S^2}{2CA}\,dx \tag{3.16c}$$

The above basic theory has been applied for the analysis of simple span and continuous span post-tensioned plate girder bridges. The following steps are followed for the analysis of pre-stressed beam:

a) Structure is made determinate by putting required releases.

b) The moment diagram due to applied load for released structure is drawn.

c) Applying unit load or unit moment at each release, the corresponding diagrams are drawn. All other applied loads are removed while drawing unit moment or load diagrams. The inderterminate moment is given by

$$M = m_o + m_1 x_1 + m_2 x_2 + \dots\dots + m_n x_n \tag{3.17}$$

According to the theory of least work

$$\frac{\delta u}{\delta x_1} = 0 \tag{3.18}$$

Putting the value of indeterminate moment in equation (3.11) and differentiating with respect to x_1 x_2, to x_n, the following results are obtained

$$\frac{\delta u}{\delta x_1} = \int_x \frac{(m_o + m_1 x_1 + \dots\dots + m_n x_n)^2}{2EI} dx \tag{3.19}$$

Simplifying,

$$\int_x \frac{m_o m_1}{EI} dx + x\int_x \frac{m_1^2}{EI} dx + \dots\dots + x_n \int_x \frac{m_1 m_n}{EI} dx \tag{3.20}$$

For convenience, the terms in equation (3,20) are denoted as follows:

$$\int_x \frac{m_o\, m_1}{EI}\, dx \qquad = u_1 \tag{3.21a}$$

$$\int_x \frac{m_1^2}{EI}\, dx \qquad = f_{11} \tag{3.21b}$$

$$\int_x \frac{m_1\, m_2}{EI}\, dx \qquad = f_{12} \tag{3.21c}$$

Putting the values of these notations in equation (3.20),gives

$$x_1 f_{11} + x_2 f_{12} + \ldots\ldots\ldots\ldots\ldots\ldots + x_n f_{1n} = -u_1 \tag{3.22}$$

So number of equations as per redundants can be formed. In general, the coefficient $f_{11}, f_{12}, \ldots$ f_{1n}, considering bending energy only;

$$f_{ij} = \int_x \frac{m_i\, m_j}{EI}\, dx \tag{3.23a}$$

$$u_i = \int_x \frac{m_o\, m_i}{EI}\, dx \tag{3.23b}$$

But in post-tensioned beam, the direct energy is equally important. Working on the same lines as for bending energy, the total influence coefficient are given by

$$f_{ij} = \int_x \frac{m_i\, m_j}{EI}\, dx + \int_x \frac{n_i\, n_j}{EA}\, dx + \int_x \frac{n_i\, n_j}{E_c A_c}\, dx \tag{3.24a}$$

$$u_i = \int_x \frac{m_o\, m_i}{EI}\, dx + \int_x \frac{n_o\, n_i}{EA}\, dx + \int_x \frac{n_o\, n_j}{E_c A_c}\, dx \tag{3.24b}$$

where,

$\int_x \frac{m_i m_j}{EI} dx$ = Influence due to energy under bending

$\int_x \frac{n_i n_j}{EA} dx$ = Influence due to axial thrust

$\int_x \frac{n_i n_j}{E_c A_c} dx$ = Influence due to cable itself

The terms $\int_x \frac{n_o n_i}{EA} dx$ and $\int_x \frac{n_o n_i}{A_c E_c} dx$ are negligible

So,

$$u_i = \int_x \frac{m_o m_i}{EI} dx \tag{3.25}$$

$$f_{ij} = \int_x \frac{m_i m_j}{EI} dx + \int_x \frac{n_i n_j}{EA} dx + \int_x \frac{n_i n_j}{E_c A_c} dx \tag{3.26}$$

3.4 APPLICATION OF INFLUENCE COEFFICIENT METHOD

a) Simple Span Straight Cables :- The general arrangement of the beams is shown in Fig. 3.1.

As there is only one redundant x_1,

$$f_{11} x_1 = -u_1 \tag{3.27}$$

$$f_{11} = \int_x \frac{m_1^2}{EI} dx + \int_x \frac{n_B^2}{EA} dx + \int_x \frac{n_c^2}{E_c A_c} dx \tag{3.28}$$

$$u_1 = \int_x \frac{m_o m_1}{EI} dx \tag{3.29}$$

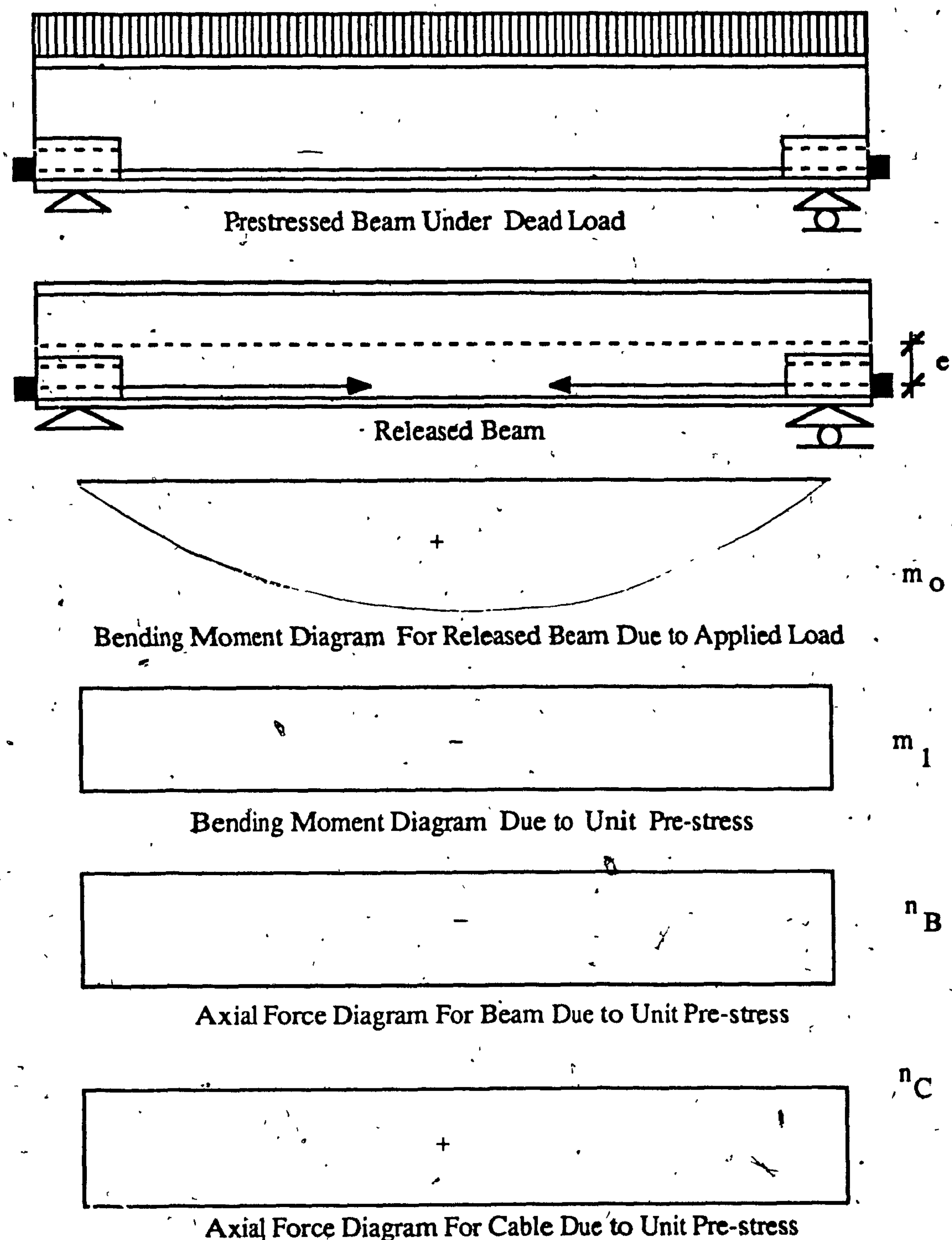


Fig. 3.1 Simple Span With Straight Cables Under Dead Load.

When the values of these coefficients have been found, they have the following values

$$f_{11} = \left[\frac{e^2 L}{EI} + \frac{L}{EA} + \frac{L}{E_c A_c} \right] \tag{3.30a}$$

$$u_1 = - \frac{q L^3 e}{12 EI} \tag{3.30b}$$

By putting the values of f_{11} and u_1 in equation (3.27), x_1 can be determined. The redundant is the increment of post-tensioned force in the cable.

b) Simple Span Drapped Cables : - The loaded and pre-stressed beam is shown in Fig. 3.2 .

For drapped cables, the values of influence coefficients are :

$$f_{11} = \frac{2ae_1(e_1 - e) + e^2(3L - 4a)}{3EI} + \frac{L}{EA} + \frac{L}{E_c A_c}\left[1 + \frac{2a}{\cos^3 \alpha}(1 - \cos^3 \alpha)\right] \tag{3.31a}$$

$$u_1 = -\frac{q}{12}\left\{ e\left[L^3 - a^2(2L - a)\right] - a^2 e_1 (2L - a) \right\} \tag{3.31b}$$

The increment of pre-stressing force is,

$$x_1 = \frac{-u_1}{f_{11}} \tag{3.32}$$

Finally, the stresses in beam due to applied load and post-tensioning are calculated using the following basic equation,

$$f = \pm \frac{M(DL + LL)y}{I} - \frac{(P + \Delta P)}{A} \pm \frac{(P + \Delta P)ey}{I} \tag{3.33}$$

c) Continuous Beam : - Assume the two span continuous beam loaded and post-tensioned as shown in Fig. 3.3. The beam is made determinate by putting the required number of releases. The values of the coefficients are obtained in similar manner as for simple span analysis.

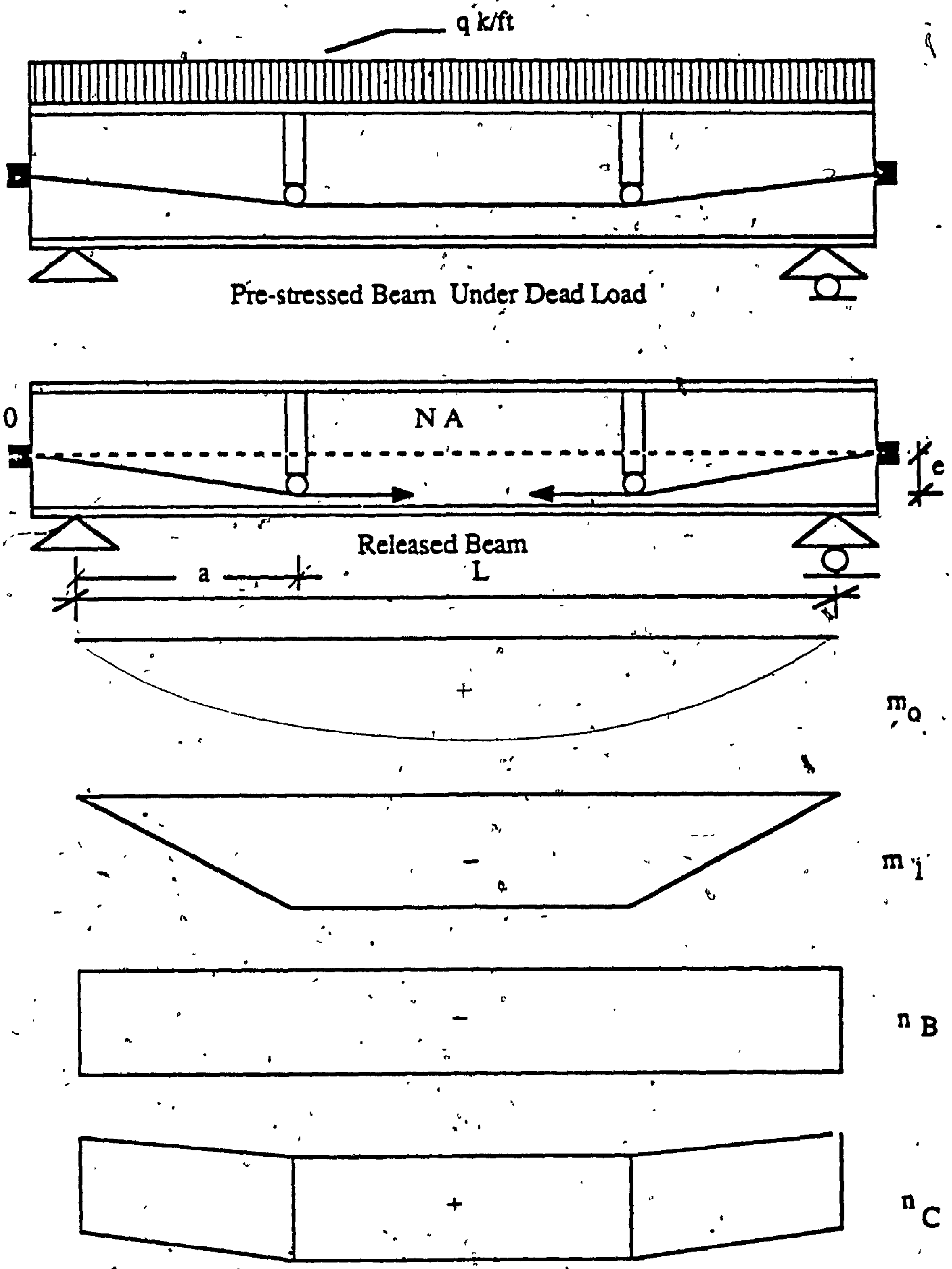


Fig.3.2 Simple Span With Drapped Cables Under Dead Load.

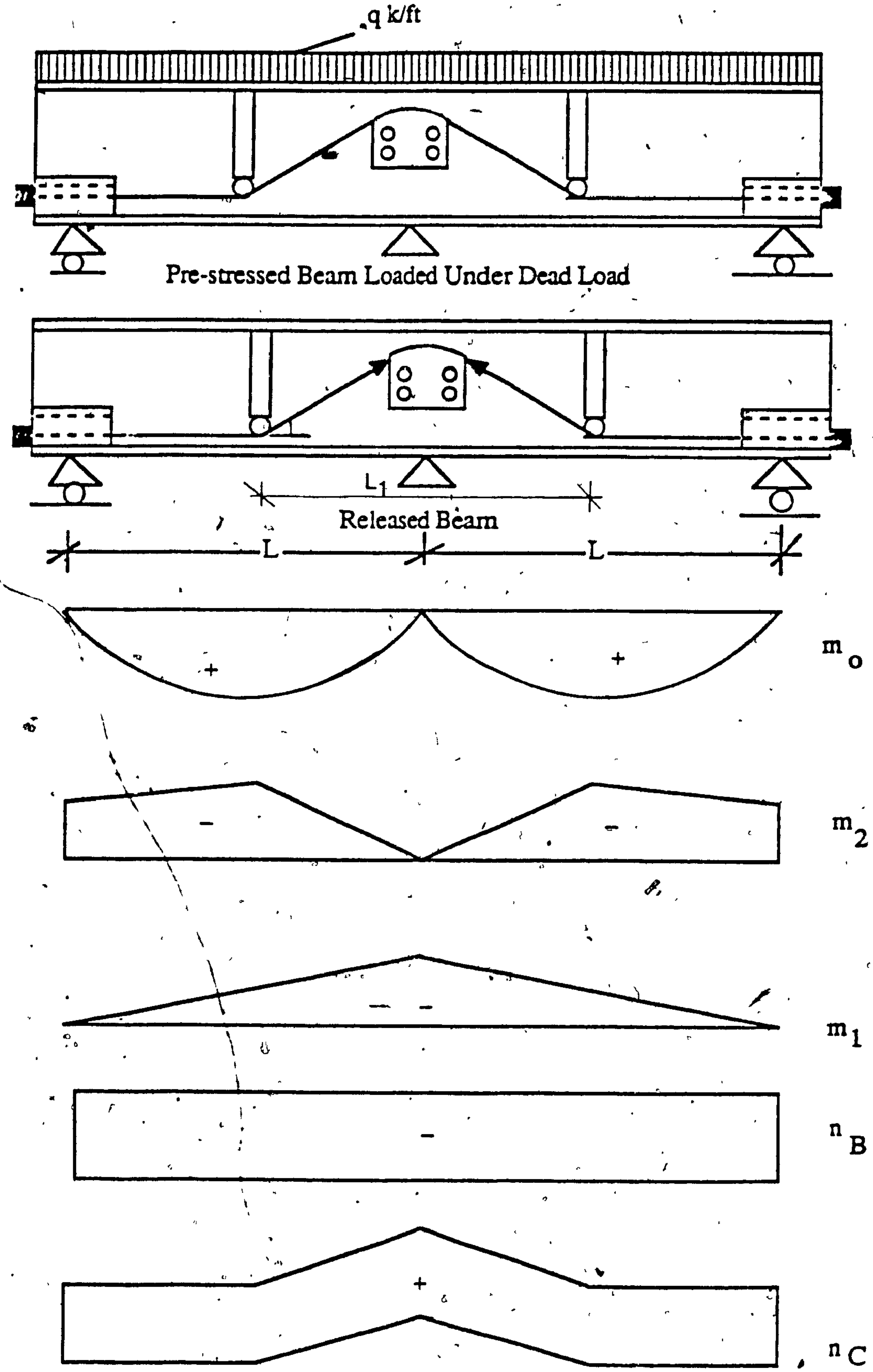


Fig. 3.3 Continuous Pre-stressed Beam With Dead Load.

Writing the influence coefficient equations,

$$x_1 f_{11} + x_2 f_{12} = -u_1 \tag{3.34}$$

$$x_1 f_{21} + x_2 f_{22} = -u_2 \tag{3.35}$$

The coefficients f_{11}, f_{12}, f_{21}, f_{22}, u_1 and u_2 are calculated using equations (3.25) and (3.26).

The values of these coefficients for the case shown in Fig. 3.3 are:

$$f_{11} = \frac{2L}{3EI} \tag{3.36}$$

$$f_{22} = \frac{2L}{3EI} \left\{ e \left(1 - \frac{L_1}{2L} \right) \left[2e + e_1 \left(1 - \frac{L_1}{2L} \right) \right] + \left[e + e_1 \left(1 - \frac{L_1}{2L} \right) \right]^2 \right\} + \frac{2L}{EA}$$

$$+ \frac{1}{E_c A_c} \left[2L + \frac{L_1}{\cos^3 \alpha} \left(1 - \cos^3 \alpha \right) \right] \tag{3.37}$$

$$f_{12} = \frac{L}{3EI} \left\{ e \left(1 - \frac{L_1}{2L} \right)^2 + \left(2 - \frac{L_1}{2L} \right) \left[e + e_1 \left(1 - \frac{L_1}{2L} \right) \right] \right\} \tag{3.38}$$

$$u_1 = - \frac{qL^3}{12EI} \tag{3.39}$$

$$u_2 = - \frac{q}{6EIL_1} \left\{ L^3 \left[eL + e_1 \left(L - \frac{L_1}{2} \right) \right] - (e + e_1) \left(L - \frac{L_1}{2} \right)^3 \left(L + \frac{L_1}{2} \right) \right\} \tag{3.40}$$

Similarly, the influence coefficients are derived for any cable configuration and any type of loading. Once the influence coefficients are known, the values of the redundants are calculated using equations (3.34) and (3.35).

CHAPTER 4

4.0 EXPERIMENTAL PROGRAM

4.1 EXPERIMENTAL PLANNING

To check the theoretical results, the experimental observations are necessary. As indicated earlier, the theory has been applied to simple span and continuous span bridges; so it became necessary to test two models and to observe the behaviour of models under post-tensioning.

Different types of materials for models were studied and finally the steel was considered the best for models, because it is the same material as the prototype. The sizes of models were so selected so that it is convenient to test in the laboratory.

4.2 SIMILITUDE CONDITIONS

The similitude conditions were applied in order to have relationship between model and prototype so that the behaviour and response of the prototype could be predicted from tests on model beams.

In static loading conditions, the following similitude conditions should be satisfied:

a) All corresponding linear dimensions and linear deflections must be in constant ratio.

b) Corresponding external loads, internal elastic forces must be in constant ratio.

These ratios are called scale factors. By applying these scale factors to dimensions, properties and loads of the prototype, the corresponding dimensions, properties and loads for the model are determined. By the application of the principles of structural mechanics, the conditions of similitude are derived and they are expressed mathematically in terms of parameters and scale factors involved, as given below. The model dimensions are separated from the prototype by putting dash on the letter.

Similitude conditions for :

i) Length ;

$$L = K\bar{L} \tag{4.1}$$

where, K is the scale reduction factor for length.

ii) Moment of inertia;

$$I = K^4 \bar{I} Z \tag{4.2}$$

where, Z is the dimensional slicing factor introduced in order to meet the requirement of moment of inertia.

iii) Cross-sectional area;

$$A = \bar{A} K^2 Z \tag{4.3}$$

iv) Sectional modulus;

$$S = \bar{S} K^3 Z \tag{4.4}$$

v) Depth;

$$d = \bar{d} K \tag{4.5}$$

vi) Deflection ;

$$\Delta = \bar{\Delta} K \tag{4.6}$$

vii) Strain ;

$$\varepsilon = \bar{\varepsilon} \tag{4.7}$$

viii) Concentrated load;

$$Q = \bar{Q} K^2 Z \tag{4.8}$$

ix) Uniformly distributed load ;

$$w = \bar{w} KZ \tag{4.9}$$

x) Bending moment;

$$M = \bar{M}K^3Z \tag{4.10}$$

4.3 SECTIONAL PROPERTIES

As explained earlier, the similitude conditions were applied to the prototype plate girder. The model beam and the prototype plate girder are shown in Figs. 4.1 and 4.2 respectively. The calculations for simple span and continuous span bridge design is given in Appendix 1. By applying similitude conditions, the beam model cross-section is obtained as shown in Fig. 4.3 for continuous girder. Similarly, the calculations for application of similitude conditions to prototype to obtain the model beam is shown in Appendix 2.

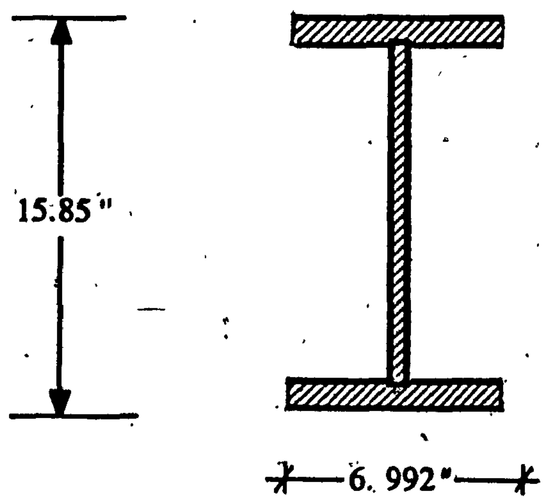


Fig. 4.1 Simple and Continuous Span Model Beam Section.

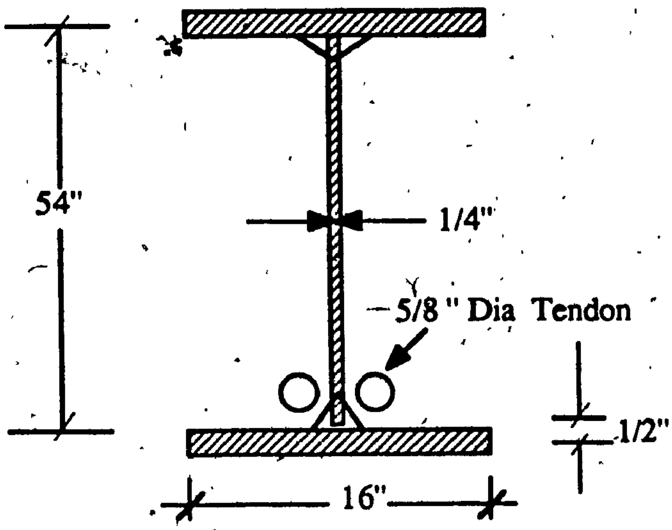


Fig. 4.2 Simple Span Prototype Girder Section

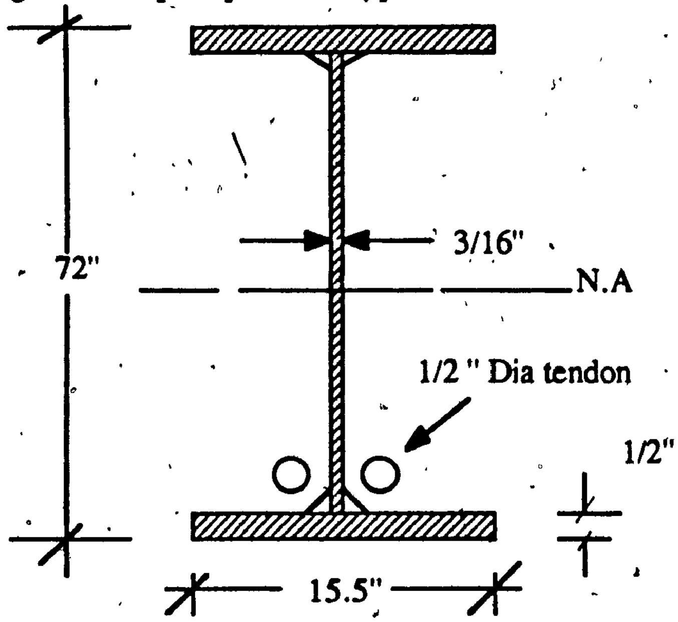


Fig. 4.3 Continuous Span Prototype Girder Section

4.4 PREDICTION OF STRESSES IN PROTOTYPE

The strains measured in the model beam are related to the actual bridge girder. First of all bending strains should be separated from the axial strains because of different correction factors to be applied to individual strain (axial and bending strain). Fig. 4.4 shows the procedure of splitting the axial strain form bending strain.

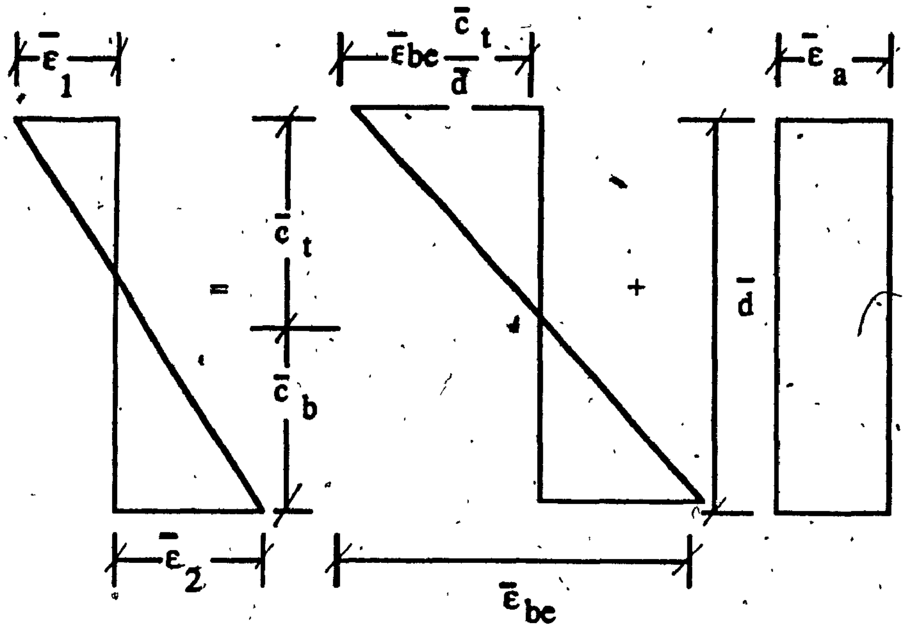


Fig. 4.4 Splitting of Bending and Axial Strains.

From the figure shown above,

$$\bar{\varepsilon}_1 = \frac{\bar{c}_t}{\bar{d}}\bar{\varepsilon}_{be} + \bar{\varepsilon}_a \tag{4.11}$$

$$-\bar{\varepsilon}_2 = -\frac{\bar{c}_b}{\bar{d}}\bar{\varepsilon}_{be} + \bar{\varepsilon}_a \tag{4.12}$$

Solving (4.11) and (4.12) yields,

$$\bar{\varepsilon}_{be} = \bar{\varepsilon}_1 + \bar{\varepsilon}_2 \tag{4.13}$$

$$\bar{\varepsilon}_a = \frac{\bar{c}_b}{\bar{d}}\bar{\varepsilon}_1 - \frac{\bar{c}_t}{\bar{d}}\bar{\varepsilon}_2 \tag{4.14}$$

where, $\bar{\varepsilon}_1$ and $\bar{\varepsilon}_2$ are taken as absolute values regardless of those which are due to compression or tension.

Some corrections are to be made because the exact size of the beam, as required by the similitude conditions is not available. So some distortion of the model exists and corrections are necessary to be applied.

4.5 CORRECTION FOR BENDING AND AXIAL STRAINS

4.5.1 Corrections for bending strain:

a) Correction for depth :- The correction for depth is determined by establishing the relation between strain in ideal homologus cross-section and strain in actual cross-section of the model as shown in Fig. 4.5 .

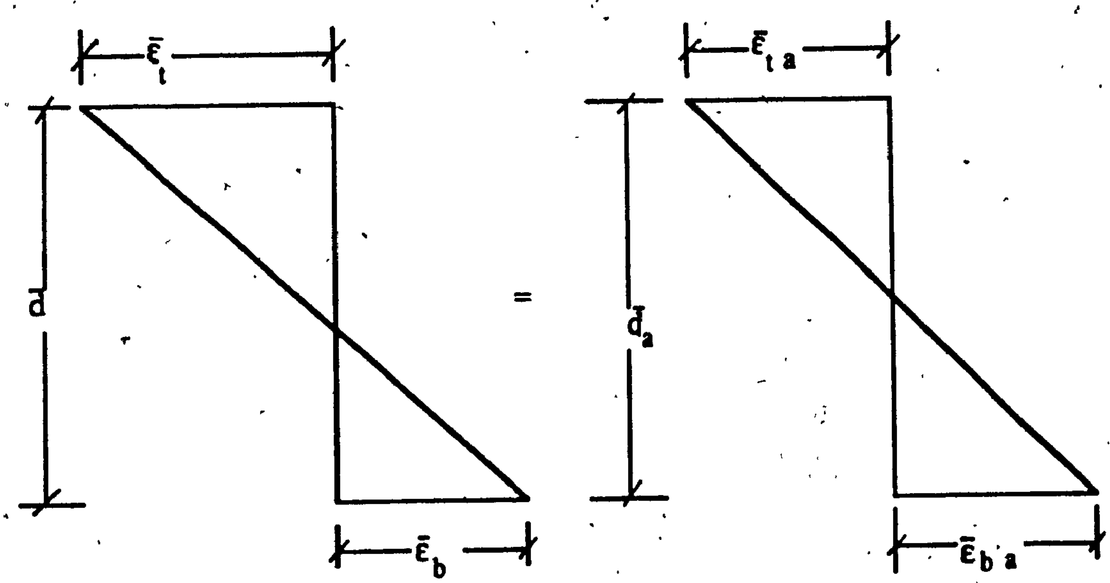


Fig. 4.5 Correction For Depth.

The depth correction is,

$$\bar{\varepsilon}_t + \bar{\varepsilon}_b = \frac{\bar{d}}{\bar{d}_a}[\bar{\varepsilon}_{ta} + \bar{\varepsilon}_{ba}] \tag{4.15}$$

The cross-section factor is thus the ratio of depth of ideal cross-section depth to actual cross-section depth.

b) Correction for shift of neutral axis:- The ideal location of neutral axis and actual location of the neutral is different as shown in Fig. 4.6 .

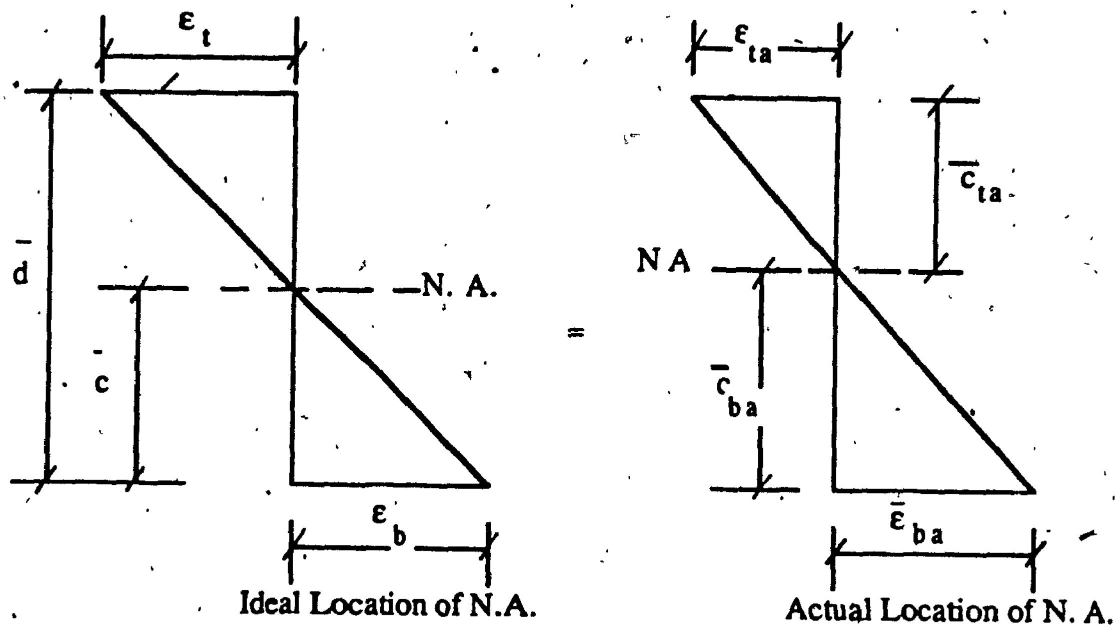


Fig. 4.6 Correction For Shift of Neutral Axis.

So correction for actual model due to the shift of nuetral axis (n.a.) is,

$$\bar{\varepsilon}_t + \bar{\varepsilon}_b = \bar{\varepsilon}_{ta} + \bar{\varepsilon}_{ba} \tag{4.16a}$$

$$\bar{\varepsilon}_t = \frac{\bar{c}}{\bar{c}_a}\bar{\varepsilon}_{ta} \tag{4.16b}$$

and

$$\bar{\varepsilon}_t = \frac{\bar{c}}{\bar{c}_a}\bar{\varepsilon}_{ta} \qquad (4.16c)$$

c) Correction due to moment of inertia:- The moment of inertia of actual model may not be as the ideal model, so correction should be applied and correction in this case is

$$\frac{\bar{I}_a}{\bar{I}} \qquad (4.16d)$$

where, I_a = actual moment of inertia; and I = Calculated ideal moment of inertia.

All the correction factors are evaluated in Appendix 2.

Combining all the corrections and strain in the model and prototype should be equal:

$$\varepsilon_p = [\frac{\bar{d}}{\bar{d}_a}\frac{\bar{c}}{\bar{c}_a}\frac{\bar{I}_a}{\bar{I}}]\bar{\varepsilon} \qquad (4.17a)$$

In the present model beam , I_a= I (Appendix 2), Hence

$$\varepsilon_p = [\frac{\bar{d}}{\bar{d}_a}\frac{\bar{c}}{\bar{c}_a}]\bar{\varepsilon} \qquad (4.17b)$$

The term [$\bar{d}/\bar{d}_a$ $\bar{c}/\bar{c}_a$] is denoted by C_k. In equation(4.17) ; ε_p= strain in prototype plate girder.

Therefore, bending stress in prototype plate girder is

$$f = \varepsilon_p E = C_k \bar{\varepsilon} E \qquad (4.18)$$

4.5.2 Correction for axial strain:

The axial strain correction for prototype should be made as follows:

$$\varepsilon_{pa} = \frac{\bar{A}_a}{\bar{A}} \qquad (4.19)$$

where, A_a = actual area of cross-section of model beam; and A= homologous ideal cross-sectional area. Therefore, axial stress is predicted with the expression,

$$f = E\,\varepsilon_{pa} \tag{4.20a}$$

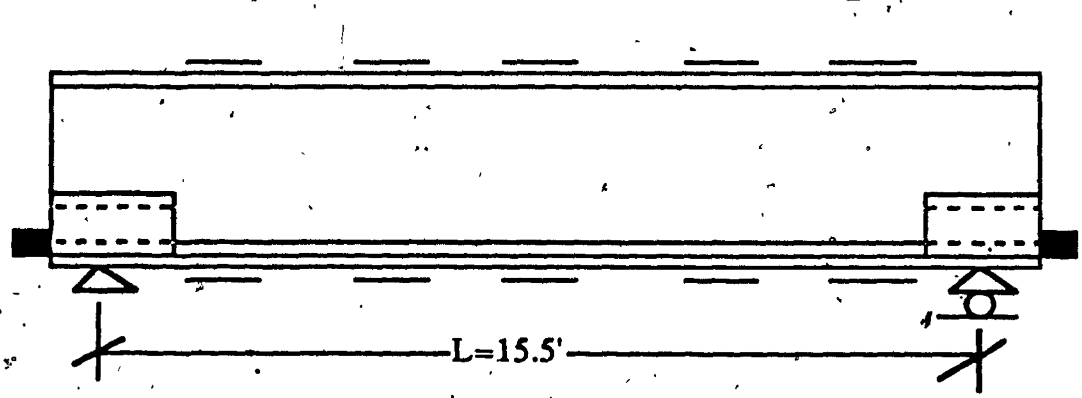
or,

$$f = E\frac{\bar{A}_a}{\bar{A}}\varepsilon Y \tag{4.20b}$$

4.6 INSTRUMENTATION FOR MODEL BEAM:

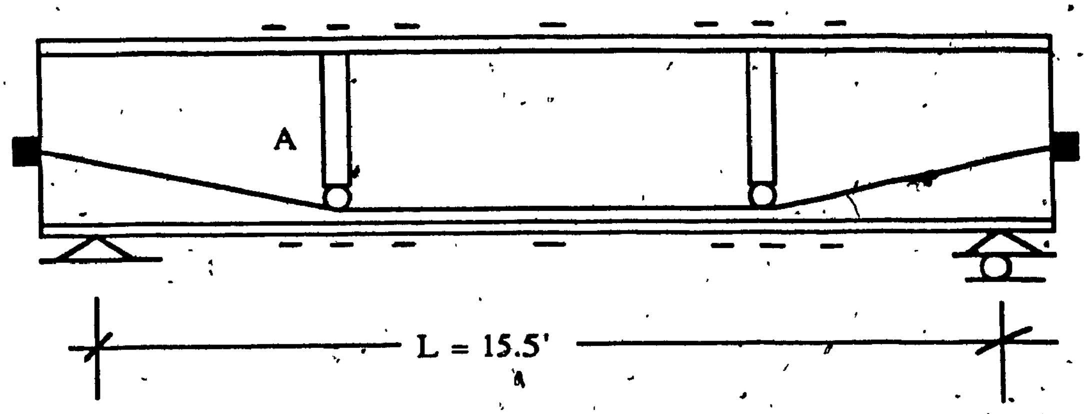
The two model beams were tested and instrumentation for each is shown as follows:

a) For simple span beam :- Fig. 4.7 and Fig. 4.8 show the instrumentation for simple span beam.



Legend ----- — Strain Gauge Location

Fig. 4.7 Straight Tendon (Cable) Configuration.



Legend — Strain Gauge Location

Fig. 4.8 Instrumentation for Drapped Cable Configuration.

b) For continuous span beam:- The instrumentation for continuous beam is shown as follows:

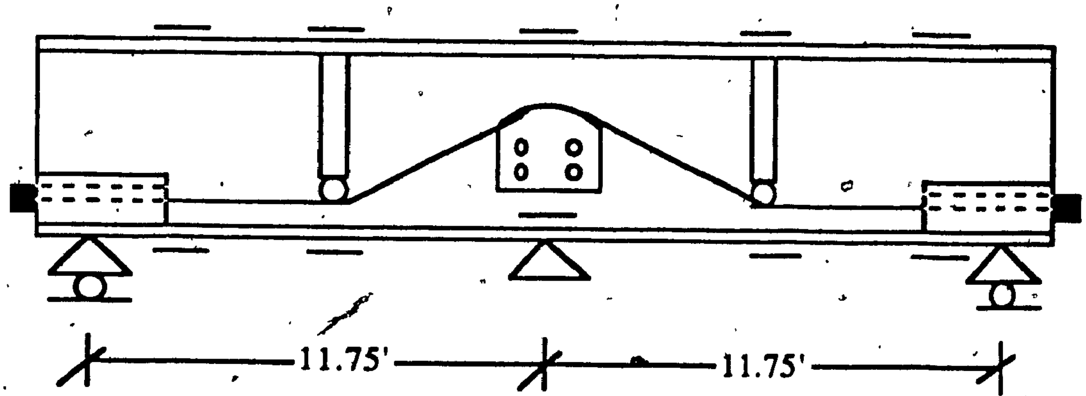


Fig. 4.9 Instrumentation for Continuous Model Beam.

4.7 GENERAL EXPERIMENTAL ARRANGEMENT

Figs. 4.10 to 4.17 exhibits the different views of tested beam in the Structures Laboratory.

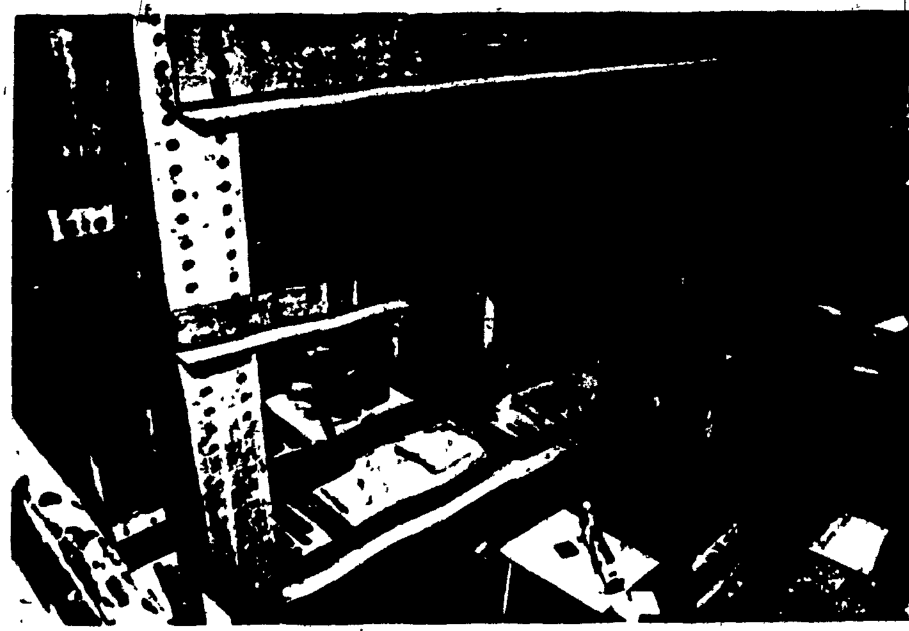


Fig. 4.10 Top View of Loaded Beam.

Fig. 4.11 Pre-stressing Jack in Position.

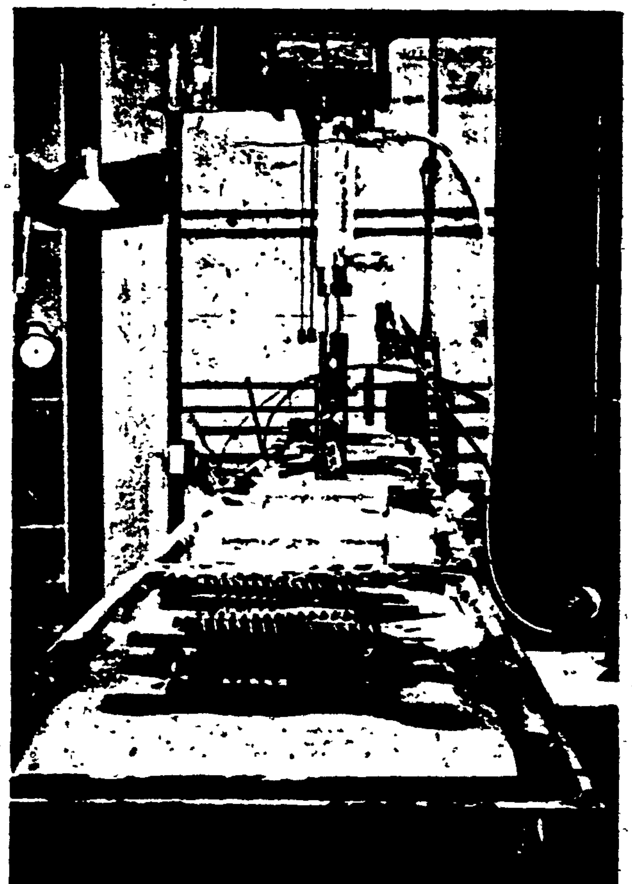


Fig. 4.12 Beam Loaded With Dead Load and Live Load.

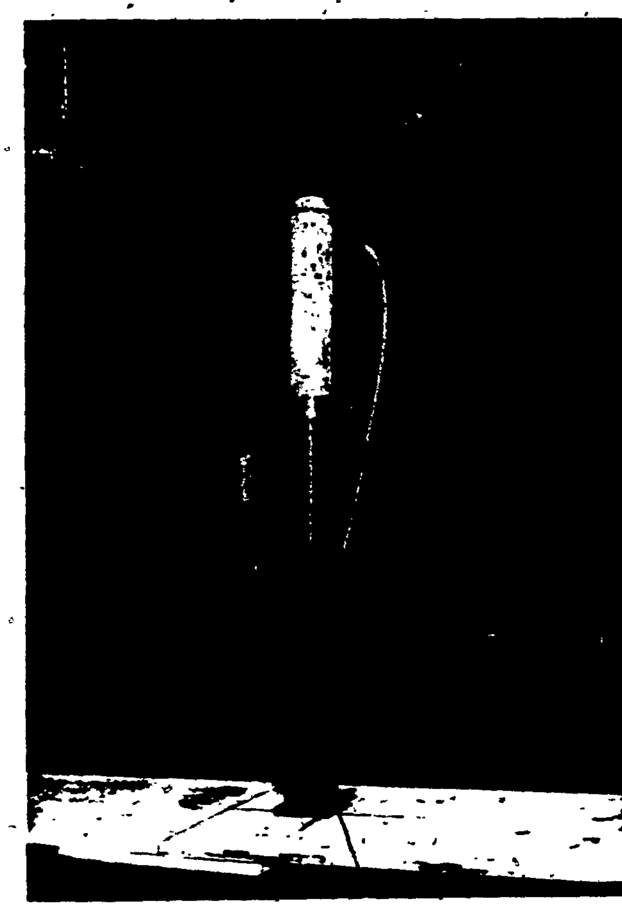


Fig. 4.13 Application of Truck Load by Jack.

Fig. 4.14 Anchor Unit for Drapped Cables.

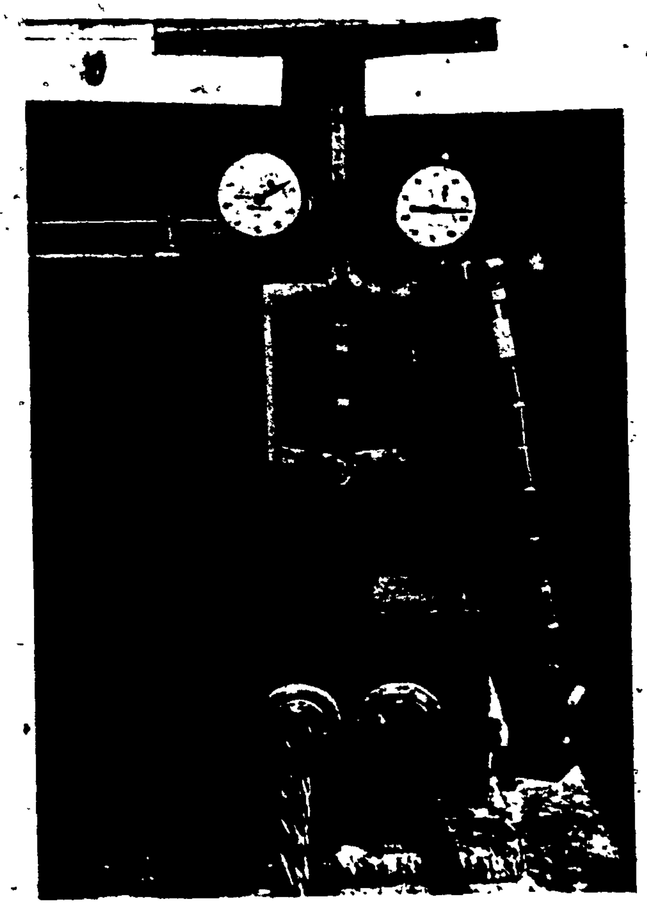


Fig. 4.15 Front View of Anchor Block for Drapped Cables.

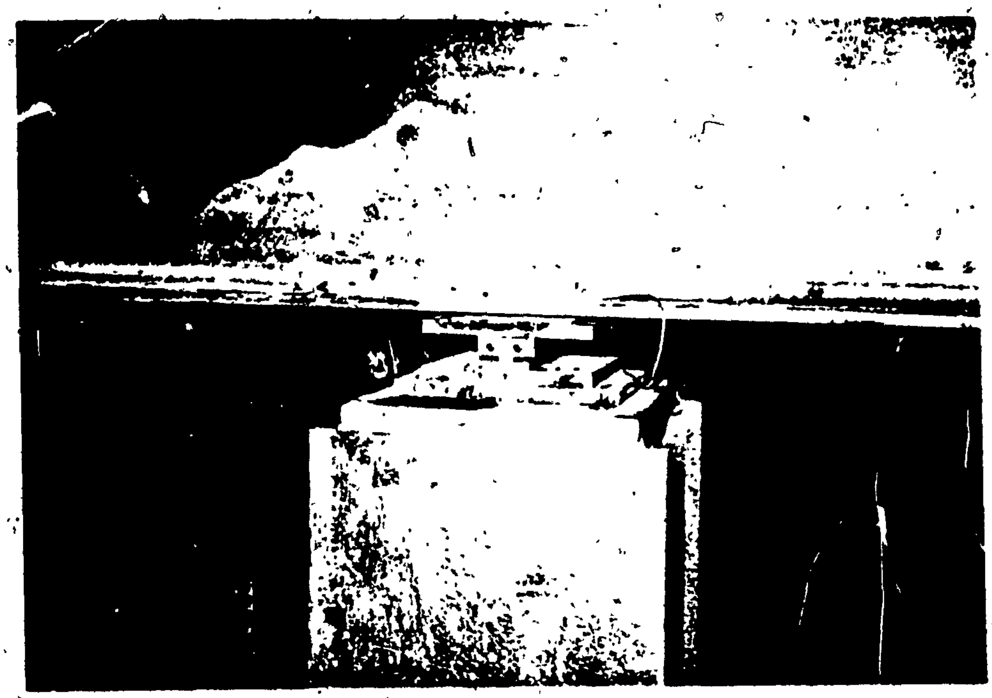


Fig. 4.16 View of Central Support (Saddle) for Continuous Beam.

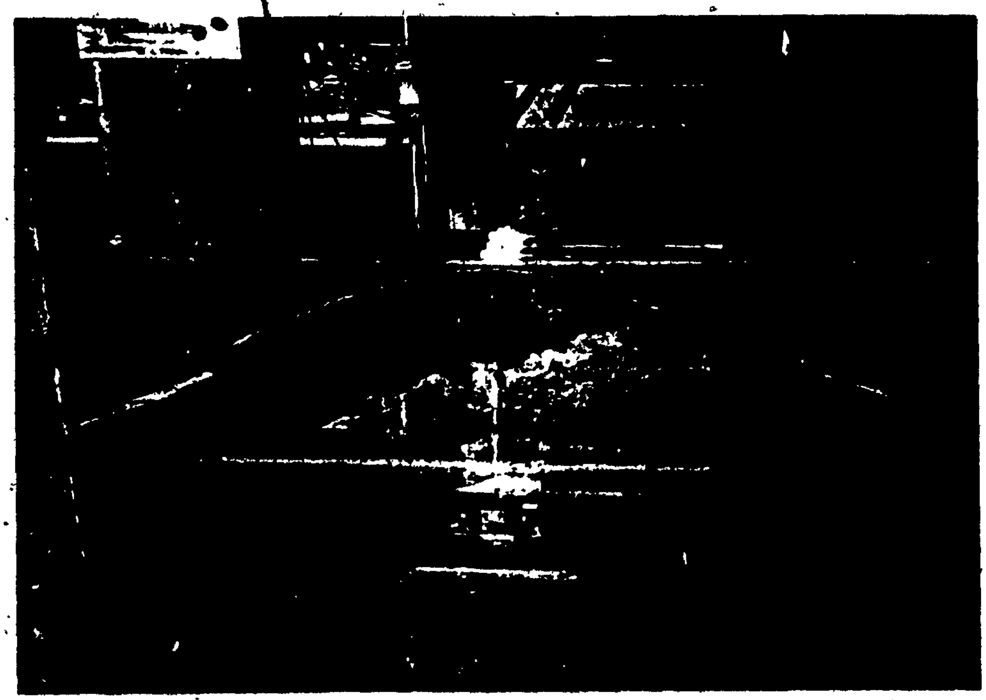


Fig. 4.17 Details of Central Support (Saddle) for Continuous Beam.

CHAPTER 5

5.0 TEST RESULTS

The analytical work has been checked by experimental observations as described in this chapter. The two tests were performed, one for simple span model beam and the other for continuous model beam. The details of the tests and test results are as follows:

5.1 SIMPLE SPAN MODEL BEAM:

For simple span model beam two types of cable configuration were tested as follows:

5.1.1 Straight Cable Configuration:- The straight cable configuration for simple span beam under dead load and live load is shown in Fig. 3.1 and Fig. 5.1, respectively.

From the theory, the coefficients f_{11D} and u_{1D}, for uniformly distributed dead load is as follows:

$$f_{11D} = \left[\frac{e^2 L}{EI} + \frac{L}{EA} + \frac{L}{E_c A_c} \right] \tag{5.1}$$

$$u_{1D} = \frac{1}{12EI} q L^3 e \tag{5.2}$$

The live load also acts on the bridge, so these coefficients similarly are calculated by exact integration of the moment diagrams as given below;

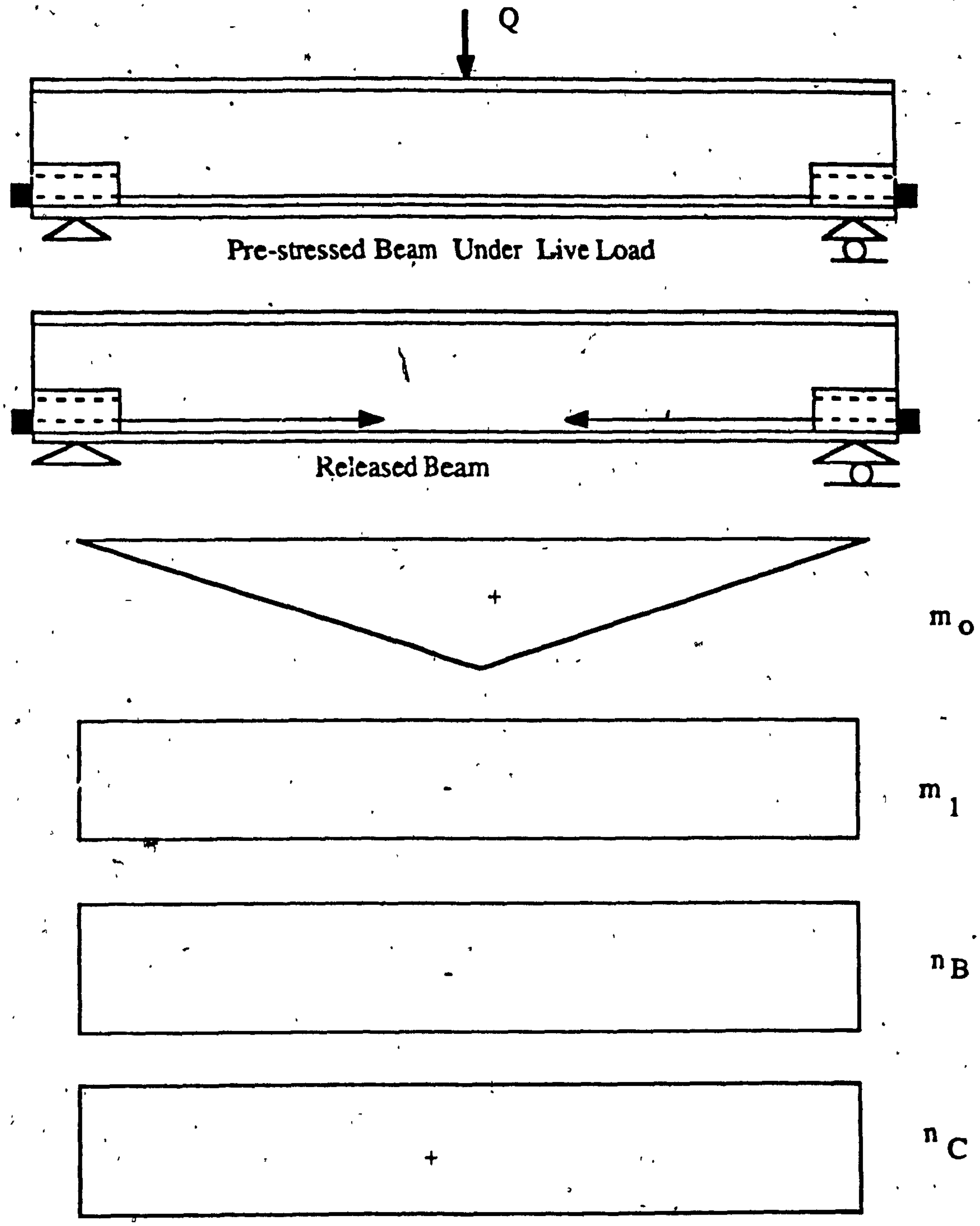


Fig. 5.1 Simple Span Beam with Straight Cables Under Live Load.

$$f_{11L} = \left[\frac{e^2 L}{EI} + \frac{L}{EA} + \frac{L}{E_c A_c} \right] \tag{5.3}$$

$$u_{1L} = - \frac{PLe}{8} \tag{5.4}$$

where, P is the resultant of truck load which is positioned on the beam so that the bending moment is maximum. The increment in post-tensioning force due to uniform superimposed load and due to truck load is calculated separately and added as follows;

For dead load,

$$\Delta P_{DL} = - \frac{u_{1D}}{f_{11D}} \tag{5.5}$$

For live load,

$$\Delta P_{LL} = - \frac{u_{1L}}{f_{11L}} \tag{5.6}$$

The theoretical stresses in the girder are calculated by using the general formula:

$$f = \pm \frac{M(DL + LL)y}{I} - \frac{(P + \Delta P)}{A} \pm \frac{(P + \Delta P)ey}{I} \tag{5.7}$$

The net values of the stresses are tabulated. comparison of the theoretical results and experimental observations for straight tendons and drapped tendons under dead load and live load (truck load) are made. The strains recorded on distorted models are changed to true model strains by applying corrections. The correction factor for bending strain is Y[d/d_a c/c_a] and correction factor for axial strain is Y[A_a/A] . These corrections are explained in the previous chapters. The values of the correction factors are given in Appendix 2.

The location of various cross-sections under experimental observations are shown in Fig. 5.3 and the stresses at respective sections are tabulated following Fig. 5.3.

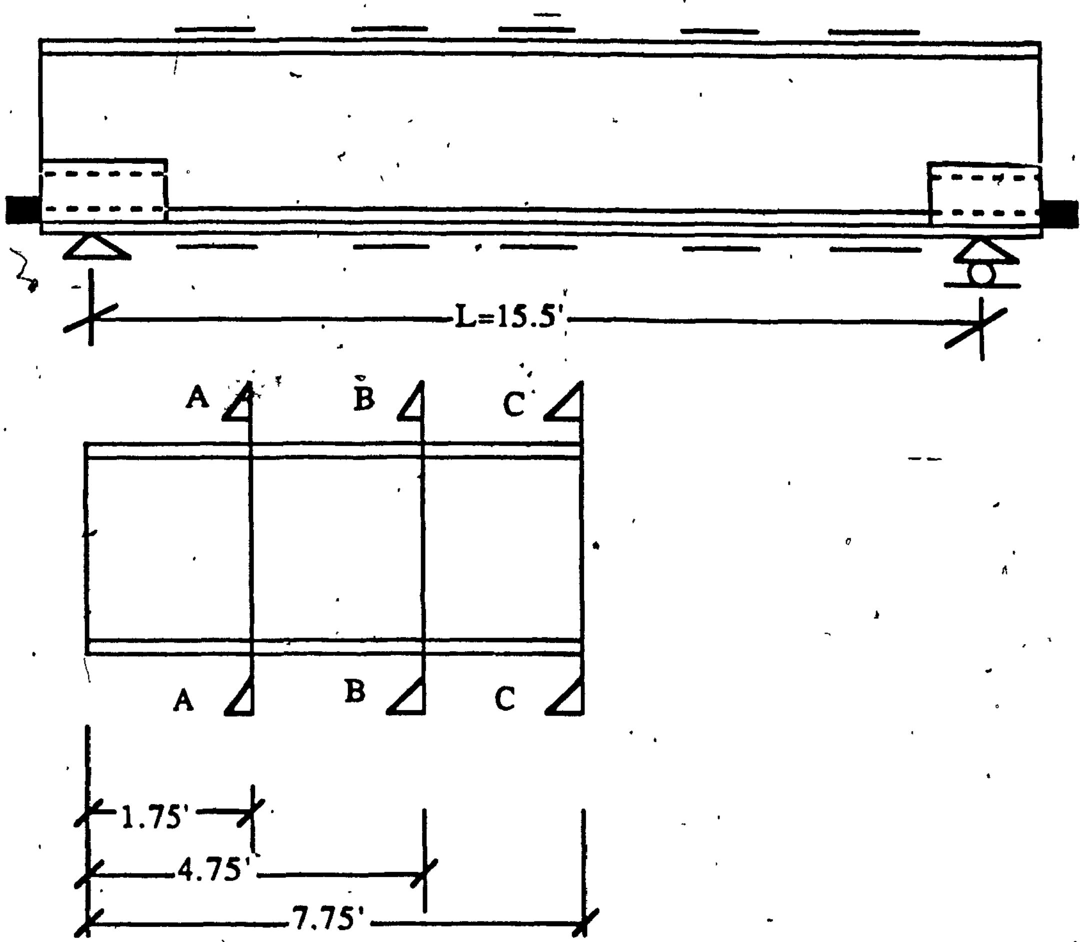


Fig. 5.2 Locations of Cross-sections Under Experimental Study.

Table 5.1 Stresses at Cross-section A-A

Pre-stressing Force (kips)	Stress at Top (ksi)		% Deviations	Stress at Bottom (ksi)		% Deviations
	Theoretical	Experimental		Theoretical	Experimental	
8.362	-3.983	-4.054	1.78	+1.460	+1.437	1.58
23.746	-3.613	-3.770	4.35	-3.353	-3.450	2.89
39.130	-3.243	-3.156	2.68	-8.160	-8.270	1.35
54.515	-2.873	-2.780	3.23	-12.980	-12.970	0.07
69.900	-2.510	-2.514	0.16	-17.796	-16.710	6.10
85.285	-2.133	-2.183	2.54	-22.593	-21.763	3.67

Table 5.2 Stresses at Cross-section B-B

Pre-stressing Force (kips)	Stress at Top (ksi)		% Dev-iations	Stress at Bottom (ksi)		% Dev-iations
	Theoretical	Experimental		Theoretical	Experimental	
8.362	-8.621	-8.822	2.33	+6.101	+6.070	0.49
23.746	-8.251	-8.142	1.32	+1.291	+1.270	1.63
39.130	-7.881	-7.868	0.16	-3.159	-3.500	0.54
54.515	-7.513	-7.770	3.42	-8.333	-8.532	2.39
69.900	-7.148	-7.068	1.12	-13.152	-13.200	0.36
85.285	-6.771	-6.767	0.06	-17.949	-18.450	2.79

Table 5.3 Stresses at Cross-section C-C

Pre-stressing Force (kips)	Stress at Top (ksi)		% Dev-iations	Stress at Bottom (ksi)		% Dev-iations
	Theoretical	Experimental		Theoretical	Experimental	
8.362	-11.181	-11.710	4.79	+8.655	+8.905	2.89
23.746	-10.800	-11.210	3.80	+3.835	+3.915	2.09
39.130	-10.431	-10.910	4.59	-0.975	-0.915	6.15
54.515	-10.060	-10.360	2.98	-5.785	-5.375	7.08
69.900	-9.700	-10.011	3.21	-10.605	-10.095	4.81
85.285	-9.320	-9.356	0.39	-15.405	-15.565	1.04

5.1.2 Drapped Cable Configuration:

The cables are drapped as shown in Fig. 3.2 and Fig. 5.3 . The pulleys were provided to eliminate the friction at the point of drap. The increment of pre-stressing force for drapped cable configuration under dead load and live load is calculated as follows:

$$\Delta P = \frac{-u_1}{f_{11}} \tag{5.8}$$

where for dead load and live load,

$$f_{11D} = f_{11L} = \frac{2a\,e_1\,(e_1 - e) + e^2(3L - 4a)}{3EI} + \frac{L}{EA} + \frac{1}{E_c A_c}\left[L + \frac{2a}{\cos^3\alpha}\{1 - \cos^3\alpha\}\right] \tag{5.9}$$

For dead load,

$$u_{1D} = -\frac{q}{12EI}\{e[L^3 - a^2(2L - a)] - a^2 e_1 (2L - a)\} \tag{5.10}$$

For live load,

$$u_{1L} = \frac{P}{6EI}\left[\frac{3}{4}L^2 e - a^2(e + e_1)\right] \tag{5.11}$$

For dead load, the increment of pre-stressing force is;

$$\Delta P_{LL} = \frac{-u_{1D}}{f_{11D}} \tag{5.12}$$

Similarly, for live load, the increment of pre-stressing force is;

$$\Delta P_{LL} = \frac{-u_{1L}}{f_{11L}} \tag{5.13}$$

The experimental observations were taken at different sections of the model beam and the results were compared with the theoretical stresses on the prototype plate girder. The cross-section under study are shown in Fig. 5.4.

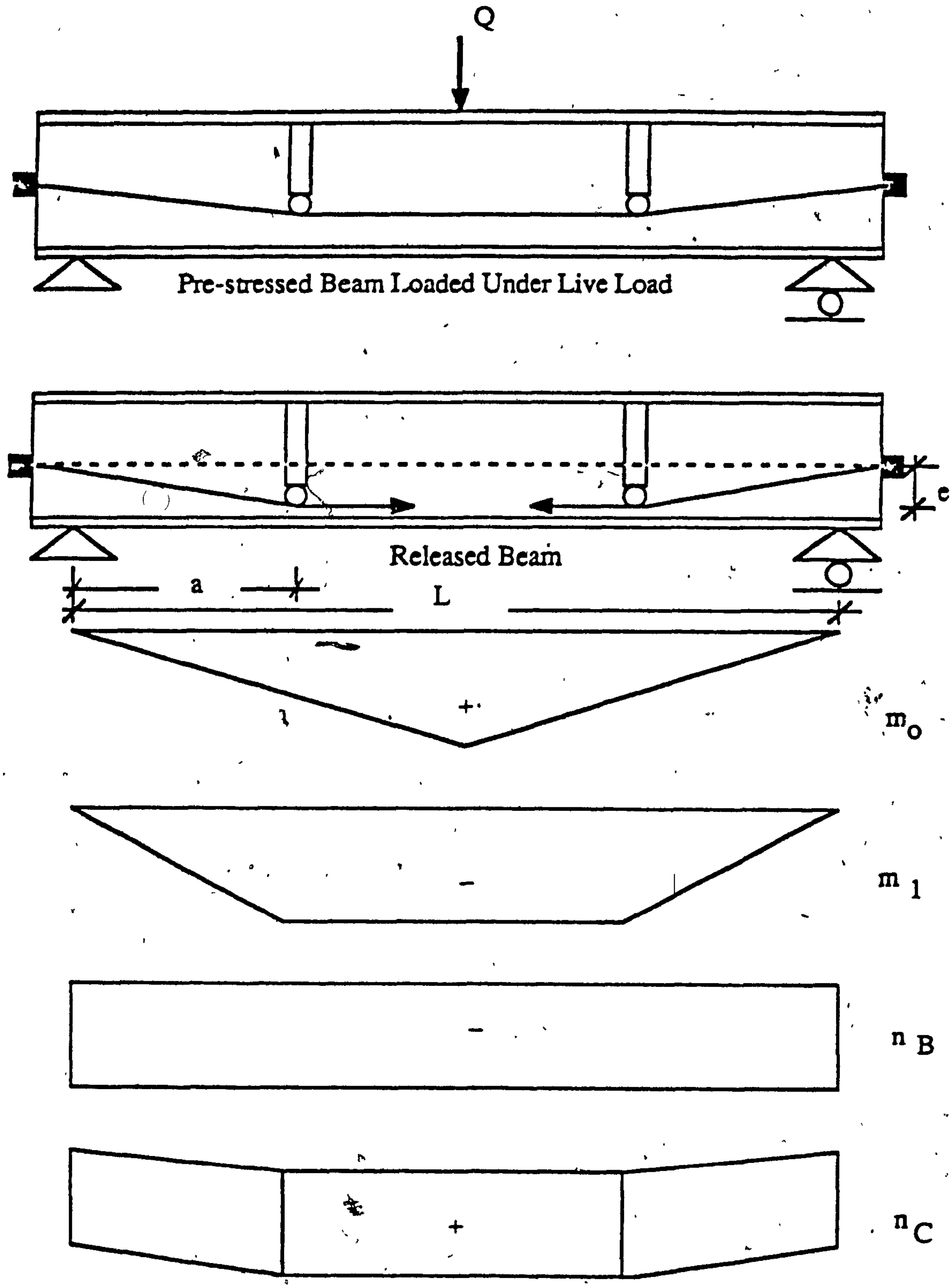
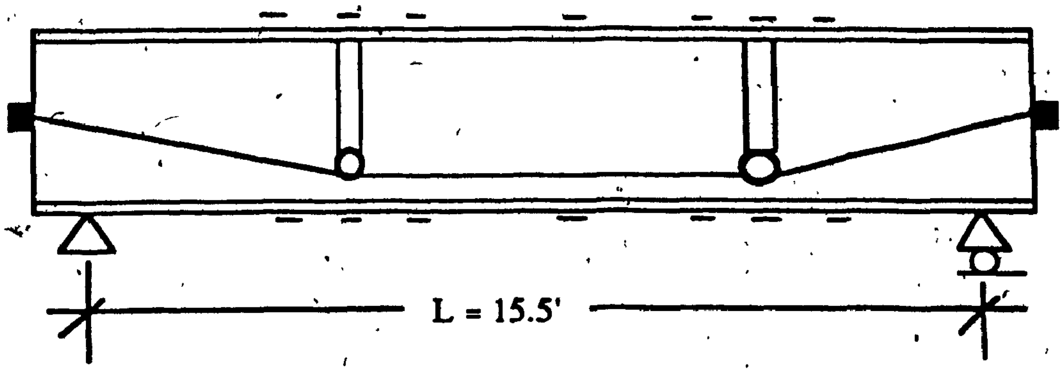


Fig. 5.3 Simple Span beam with Drapped Cables under Live Load.



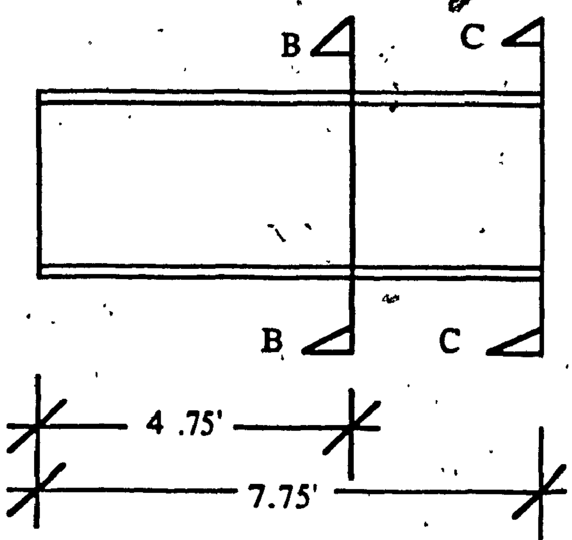


Figure 5.4 Location of Cross-sections under Study.

Table 5.4 Stresses at Section B-B .

Pre-stressing Force (kips)	Stress at Top (ksi)		% Deviations	Stress at Bottom (ksi)		% Deviations
	Theoretical	Experimental		Theoretical	Experimental	
8.223	-8.62	-9.06	5.10	+6.16	+5.70	7.46
23.607	-8.25	-8.85	7.27	+1.34	+0.94	29.90
38.992	-7.88	-7.71	2.16	-3.47	-4.35	25.30
54.377	-7.52	-7.07	5.98	-8.28	-9.77	17.99
69.762	-7.15	-7.02	1.81	-13.10	-15.60	19.08
85.146	-6.78	-6.52	3.83	-17.91	-21.20	18.37

Table 5.5 Stresses at Section C-C

Pre-stressing Force (kips)	Stress at Top (ksi)		% Deviations	Stress at Bottom (ksi)		% Deviations
	Theoretical	Experimental		Theoretical	Experimental	
8.223	-11.18	-10.84	3.08	+8.72	+8.94	2.52
23.607	-10.82	-10.92	0.92	+3.91	+3.95	0.90
38.992	-10.45	-10.97	4.97	-0.90	-0.89	1.10
54.377	-10.08	-10.09	0.10	-5.71	-5.39	5.50
69.762	-9.71	-9.89	1.65	-10.54	-9.75	7.49
85.146	-9.34	-9.77	4.60	-15.34	-16.18	5.48

Referring to Table 5.4, it can be understood that the experimental stresses at bottom for section B-B are 15 percent more than expected. This is due to local action of concentrated force acting at point of drap as shown in Fig. 5.5 . The area of distribution of concentrated stress at pulley point was small and this resulted in local stresses. However, the local effect has been noted at the top flange but it is negligible, because the stress produced by the vertical component of pre-stressing force was distributed to the web by the welds of the stiffner.

The problem of stress distribution in beam at the pulley is of great practical interest. It was shown theoretically before that the local stresses exist near the point of application of concentrated load. In this very case, the application of various forces are shown in Fig. 5.5 .

The theoretical formula for stresses in the x, y directions are given below [5] as,

$$\sigma_x = \frac{-Q}{4\pi t}\left[\, (1-\mu)\frac{y}{x^2+y^2} - (1+\mu)\,\frac{2x^2 y}{x^2+y^2{}^{2}} \right. \tag{5.14}$$

$$\sigma_y = \frac{Q}{4\pi t}\left[\, (3+\mu)\,\frac{y}{x^2+y^2} - (1+\mu)\,\frac{2x^2 y}{(x^2+y^2)^2} \,\right] \tag{5.15}$$

and,

$$\tau_{xy} = \frac{Q}{4\pi t}\left[\, (1-\mu)\,\frac{x}{(x^2+y^2)} + (1+\mu)\,\frac{2xy^2}{(x^2+y^2)^2} \right. \tag{5.16}$$

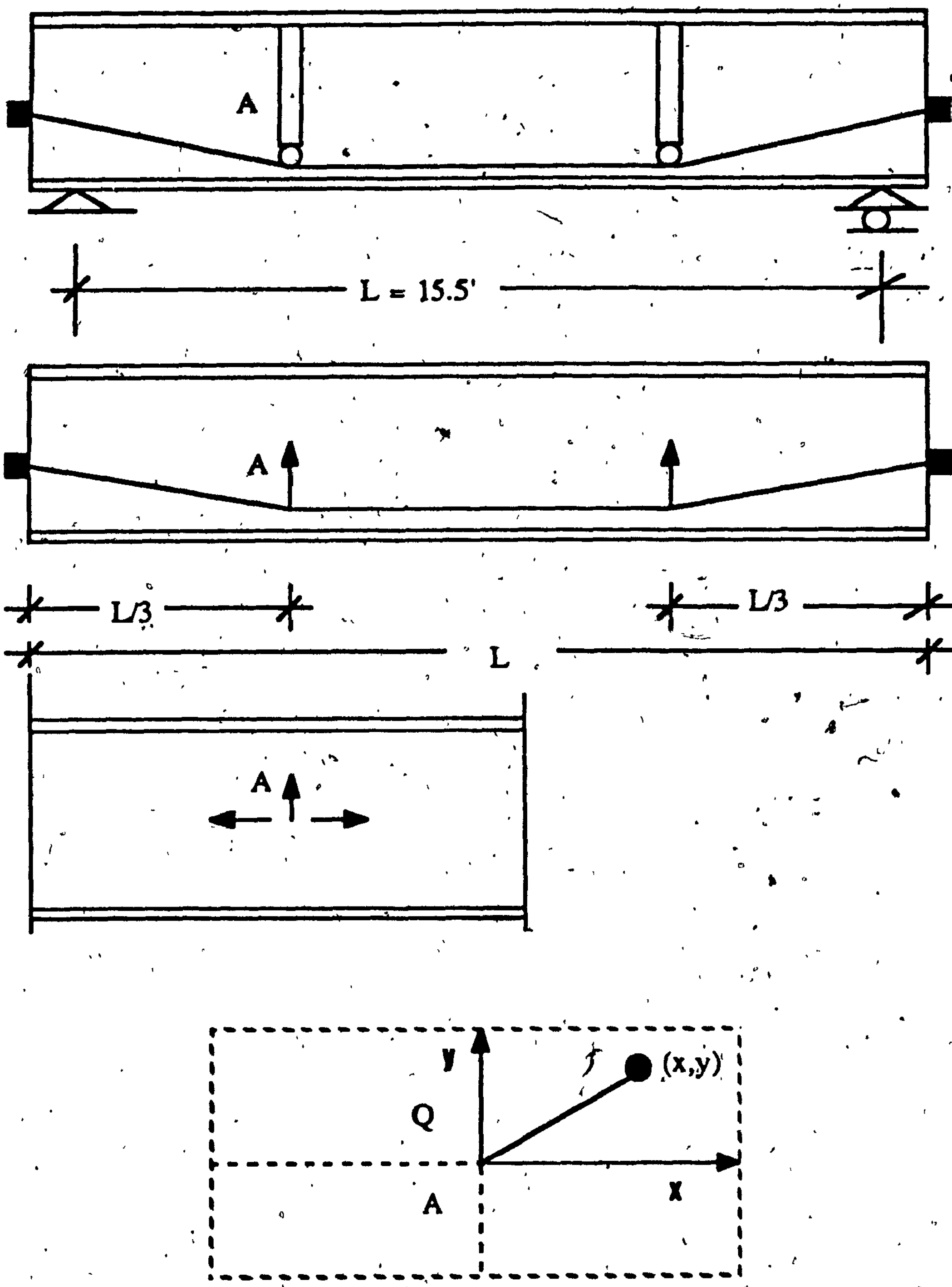


Fig. 5.5 Local Stresses At The Pulley Points.

The coordinates x and y give the position of concentrated load. As discussed earlier, the deviations of stresses at pulley points are 15 percent more than expected. If the theoretical values are increased by the local stress values as given by the sets of equations (5.14 to 5.16), it can be seen that the stresses are close to the experimental stresses. The stresses after applying correction are tabulated in Table 5.6 as follows :

Table 5.6 Stresses at Section B-B after taking the Local affect

Pre-stressing Force (kips)	Stress at Top (ksi)		% Dev-iations	Stress at Bottom (ksi)		% Dev-iations
	Theoretical	Experimental		Theoretical	Experimental	
8.223	-8.62	-9.06	5.10	+5.967	+5.70	4.47
23.607	-8.25	-8.85	7.27	+0.900	+0.94	4.30
38.992	-7.88	-7.71	2.16	-4.437	-4.35	1.96
54.377	-7.52	-7.07	5.98	-9.634	-9.77	1.41
69.762	-7.15	-7.02	1.81	-14.840	-15.60	5.12
85.146	-6.78	-6.52	3.83	-20.037	-21.20	5.80

5.2 CONTINUOUS BEAM:

The post-tensioned continuous beam under various loading is shown as in Figs. 3.3, 5.6 and 5.7 . The analysis is similar as for the simple span beam, with the difference that the redundants in the continuous beam are more, which are found by the following simultaneous equations .

$$x_1 f_{11} + x_2 f_{12} = -u_1 \qquad (5.17)$$

$$x f_{21} + x_2 f_{22} = -u_2 \qquad (5.18)$$

The values of f_{11}, f_{12}, f_{21}, and f_{22} are calculated according to the scheme of pre-stressing. For dead load, as shown in Fig. 3.3, the values of the coefficients are

$$f_{11} = \frac{2l}{3EI} \tag{5.19}$$

$$f_{22} = \frac{2L}{3EI}\left[e\left(1 - \frac{L_1}{2L}\right)\left\{ 2e + e_1\left(1 - \frac{L_1}{2L}\right)\right\} + \left\{ e + e_1\left(1 - \frac{L_1}{2L}\right)\right\}^2\right] + \frac{2L}{EA}$$

$$+ \frac{1}{E_c A_c}\left[2L + \frac{L_1}{\cos^3\alpha}\left(1 - \cos^3\alpha\right)\right] \tag{5.20}$$

$$f_{12} = f_{21} = \frac{L}{3EI}\left\{ e\left(1 - \frac{L_1}{2L}\right)^2 + \left(2 - \frac{L_1}{2L}\right)\left[e + e_1\left(1 - \frac{L_1}{2L}\right)\right]\right\} \tag{5.21}$$

$$u_{1D} = \frac{ql^2}{12EI} \tag{5.22}$$

$$u_{2D} = -\frac{q}{6EI\,L_1}\left\{ L^3\left[eL + e_1\left(L - \frac{L_1}{2}\right)\right] - (e_1 + e)\left(1 - \frac{L_1}{2}\right)^3\left(1 + \frac{L_1}{2}\right)\right\} \tag{5.23}$$

For the truck load for continuous beam, the values for f_{11}, f_{22}, f_{12} and f_{21} are same as for uniformly distributed load, the values for u_{1L} and u_{2L} are changed and are given as,

$$u_{1L} = -\frac{Ql^2}{8EI} \tag{5.24}$$

$$u_{2L} = \frac{Q}{24EI}\left[3(2e + e_1)L^2 - (e + e_1)L_1^2\right] \tag{5.25}$$

The total increment of pre-stressing force should be taken by adding the effect of superimposed dead load and live load. The beam is tested for maximum negative moment at the central support. The different moment diagrams are shown in Fig. 5.6 and Fig. 5.7 for truck load positioning for maximum positive and negative moment respectively.

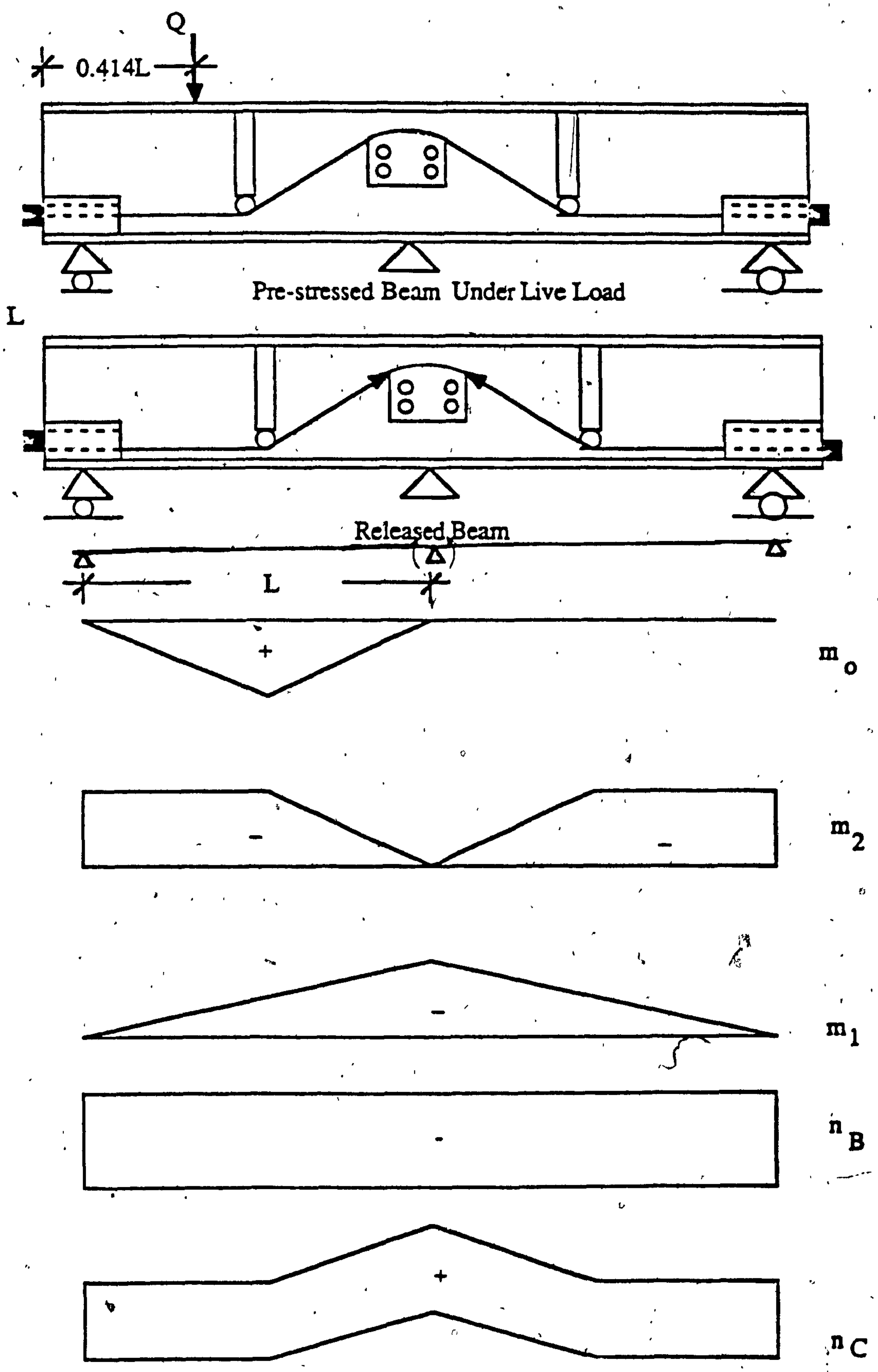


Fig. 5.6 Continuous Pre-stressed Beam under Live Load for Maximum Positive Moment

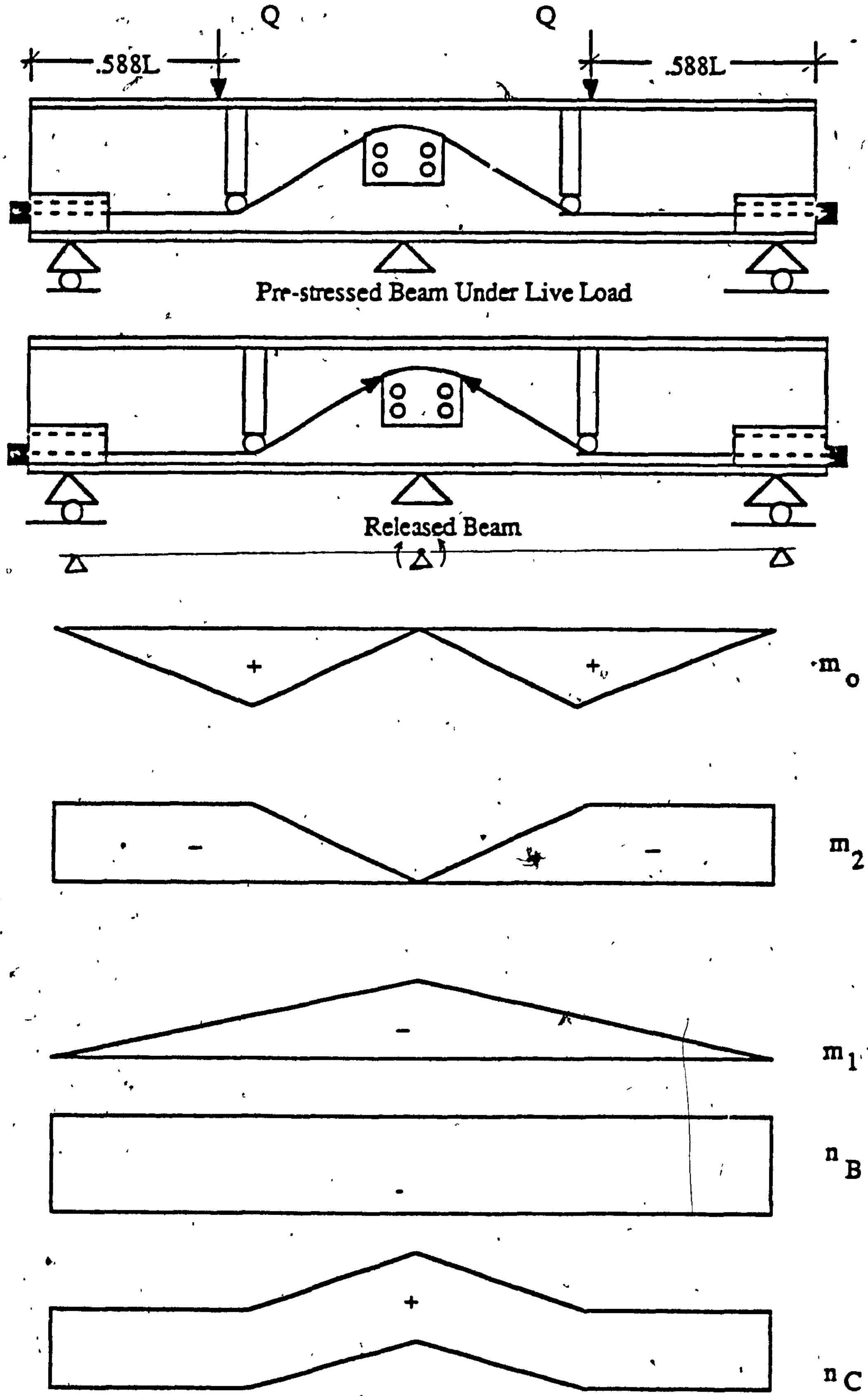


Fig. 5.7 Continuous Pre-stressed Beam under Live Load for Maximum Negative Moment.

The theoretical values for all the coefficients are calculated . The values of x_1 and x_2 are then calcualted by solving simultaneous equations (5.22) and (5.23). The redundant denotes the intermediate moment at support and x_2 denotes the increment of pre-stressing force in the tendon. The stresses are similarly calculated using the following basic equation,

$$f = \pm \frac{M(DL + LL)Y}{I} - \frac{(P + \Delta P)}{A} \pm \frac{(P + \Delta P)ey}{I} \tag{5.26}$$

The theoretical and experimental values at different sections as shown in Fig. 5.11 have been compared.

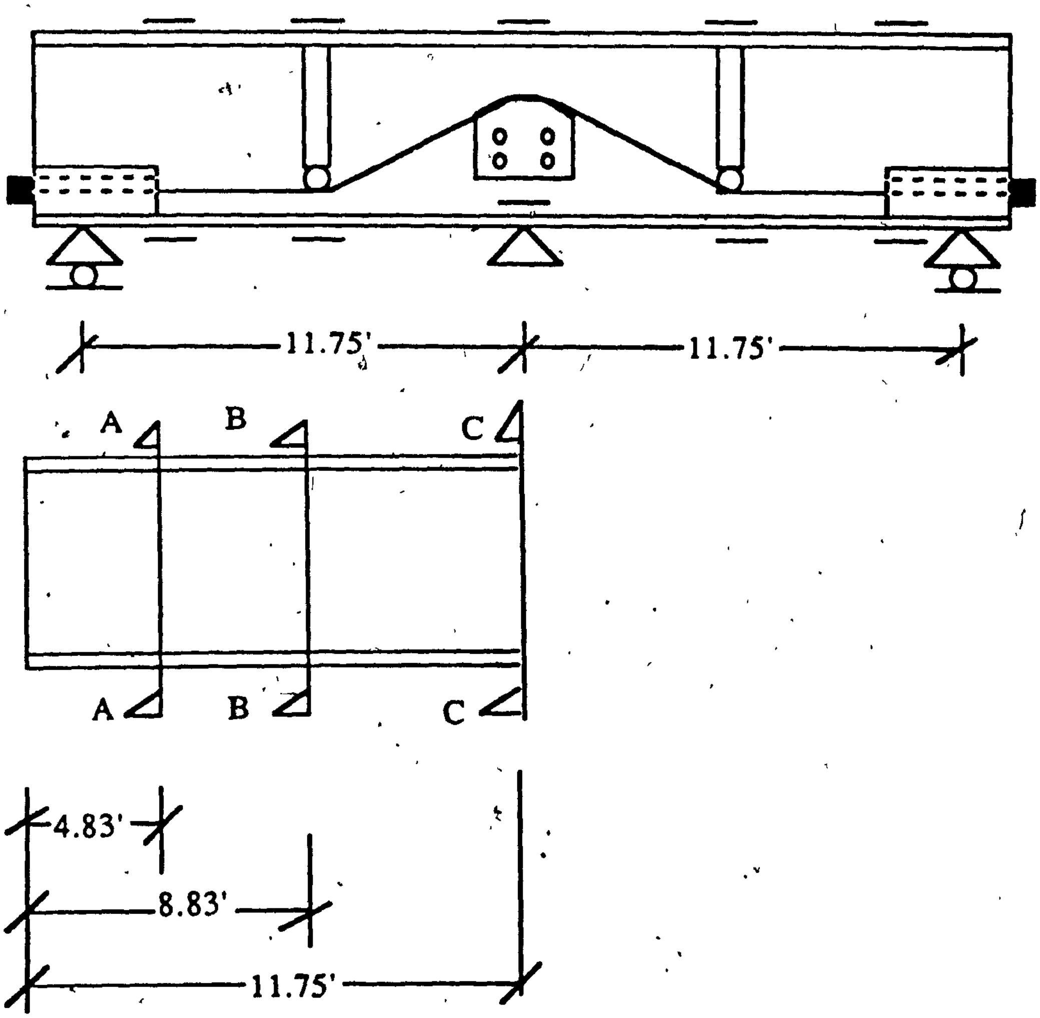


Fig. 5.8 Location of Cross-sections On The Model Under Study

The stresses at section AA ,BB and CC are shown in the Tables 5.7, 5.8 and 5.9 as follows:

Table 5.7 Stresses at Section A - A

Pre-stressing Force (kips)	Stress at Top (ksi)		% Deviations	Stress at Bottom (ksi)		% Deviations
	Theoretical	Experimental		Theoretical	Experimental	
18.597	-10.236	-10.125	1.08	+6.362	+6.250	1.76
35.263	-9.696	-9.120	5.94	+2.350	+2.380	1.28
51.930	-9.151	-8.880	2.96	-1.661	-1.800	8.36
68.596	-8.617	-8.850	2.70	-5.673	-5.550	2.17

Table 5.8 Stresses at Section B - B

Pre-stressing Force (kips)	Stress at Top (ksi)		% Deviations	Stress at Bottom (ksi)		% Deviations
	Theoretical	Experimental		Theoretical	Experimental	
18.597	+0.602	+0.625	7.97	-4.476	-5.640	26.01
35.263	+1.142	+1.250	9.46	-8.489	-10.286	21.18
51.930	+1.681	+1.799	7.02	-12.499	-15.105	20.85
68.596	+2.221	+2.354	5.99	-16.511	-20.550	18.99

Table 5.9, Stresses at Section B - B Taking The Effect of Local Stresses.

Pre-stressing Force (kips)	Stress at Top (ksi)		% Dev-iations	Stress at Bottom (ksi)		% Dev-iations
	Theoretical	Experimental		Theoretical	Experimental	
18.599	+0.602	+0.625	7.97	-6.001	-5.640	6.02
35.263	+1.142	+1.250	9.46	-11.380	-10.286	9.61
51.930	+1.681	+1.799	7.02	-16.759	-15.105	9.87
68.596	+2.221	+2.354	5.99	-22.138	-20.550	7.72

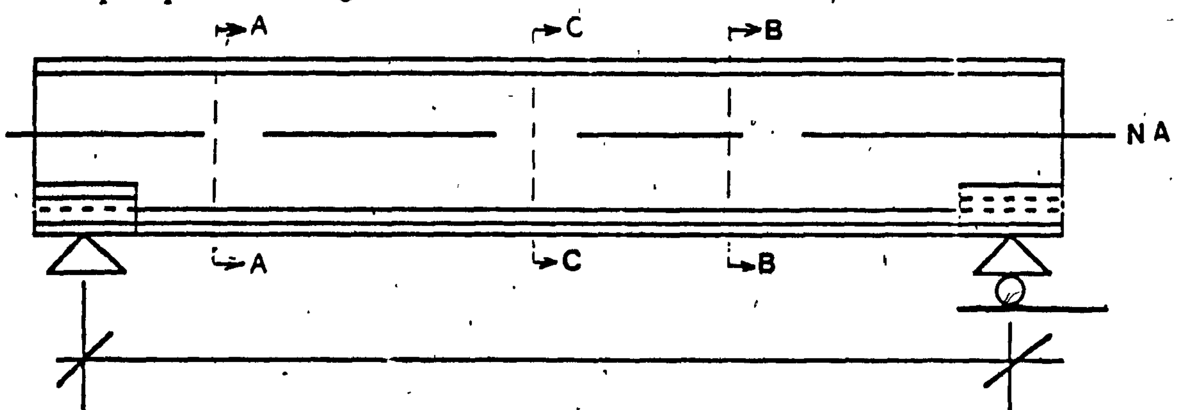
Table 5.10 Stresses at Section C - C.

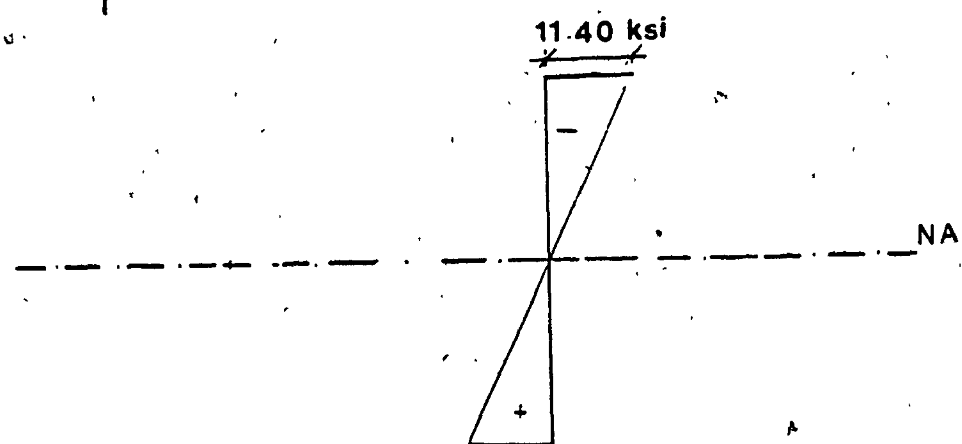
Pre-stressing Force (kips)	Stress at Bottom (ksi)		% Dev-iations	Stress at Top (ksi)		% Dev-iations
	Theoretical	Experimental		Theoretical	Experimental	
18.597	-12.940	-13.000	0.46	+8.952	+8.950	0.02
35.263	-12.401	-11.990	3.31	+4.941	+4.925	0.32
51.930	-11.860	-11.550	2.61	+0.930	+0.935	0.54
68.596	-11.320	-10.750	5.04	-3.080	-3.180	3.25

5.3 PERCENTAGE GAIN IN STRENGTH

The percentage gain in strength for simple-span and continuous span beam is shown as follows;

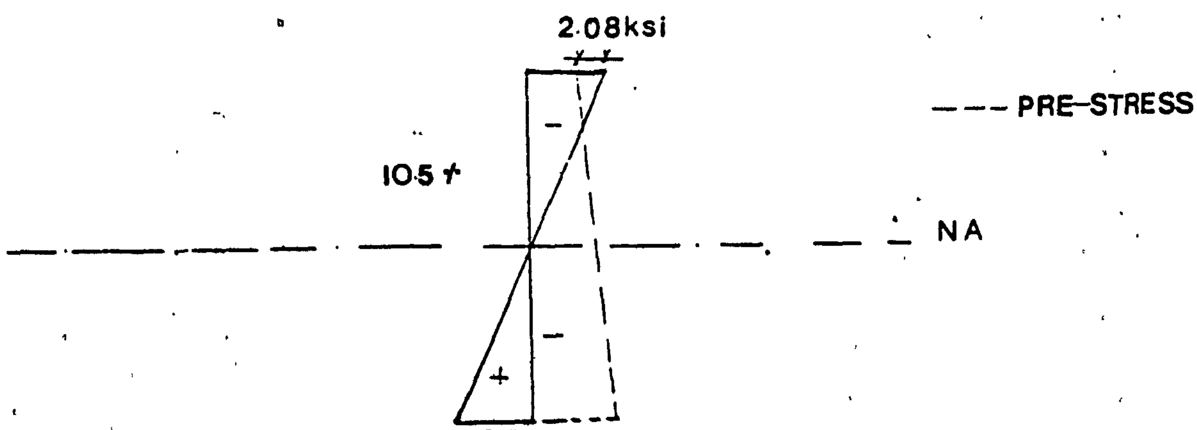
5.3.1 Simple Span with Straight Cables





C-C

(DL + LL) Stress Diagram

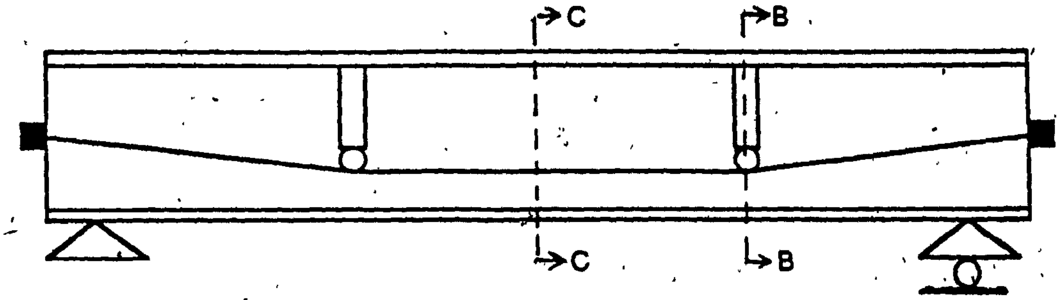


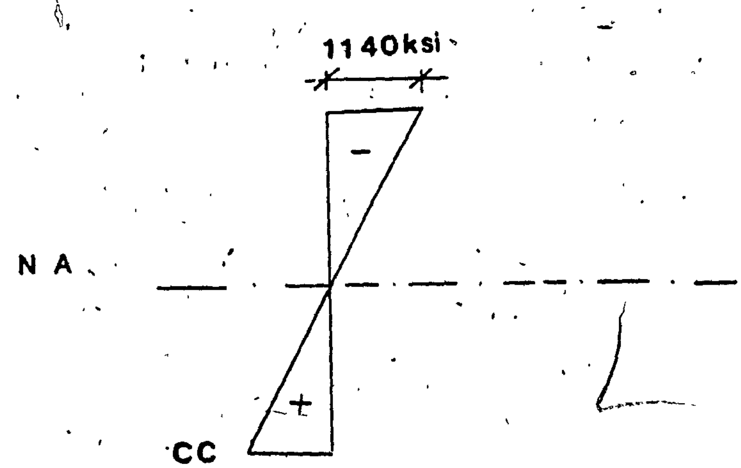
CC

(DL + LL + PR) Stress Diagram

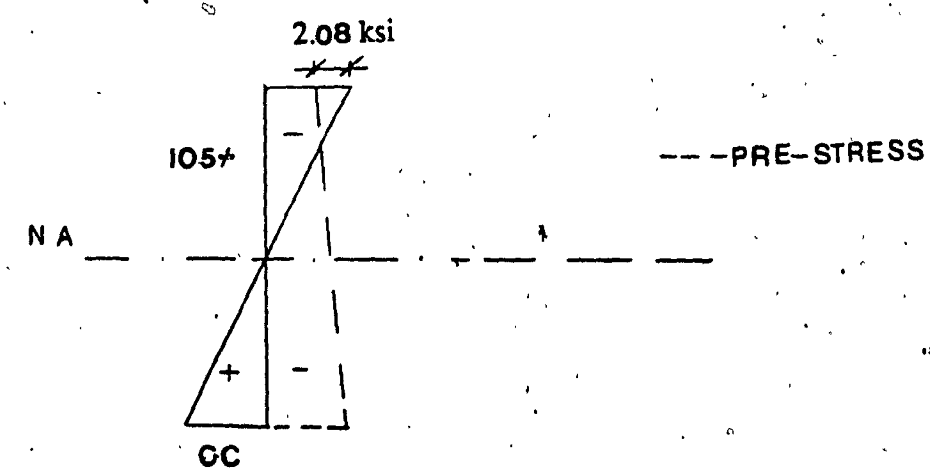
Fig.5.9 Percntage Gain In Strength For Simple Span With Straight Cables

5.3.2 Simple Span With Drapped Cables





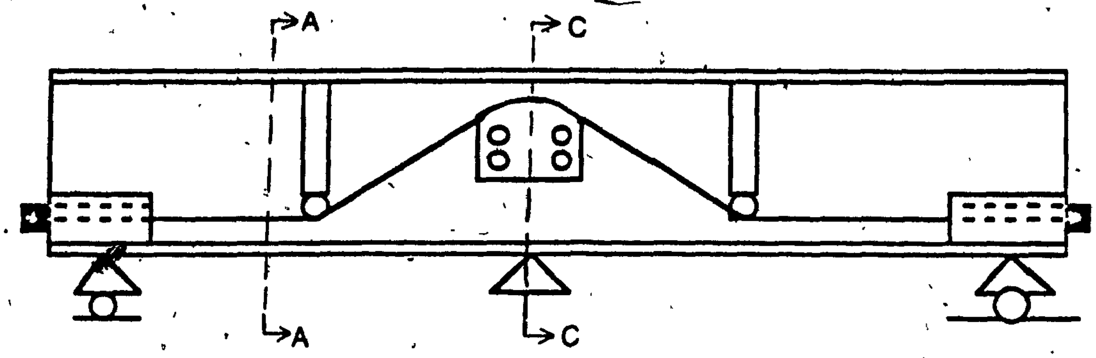
(DL + LL) Stress Diagram

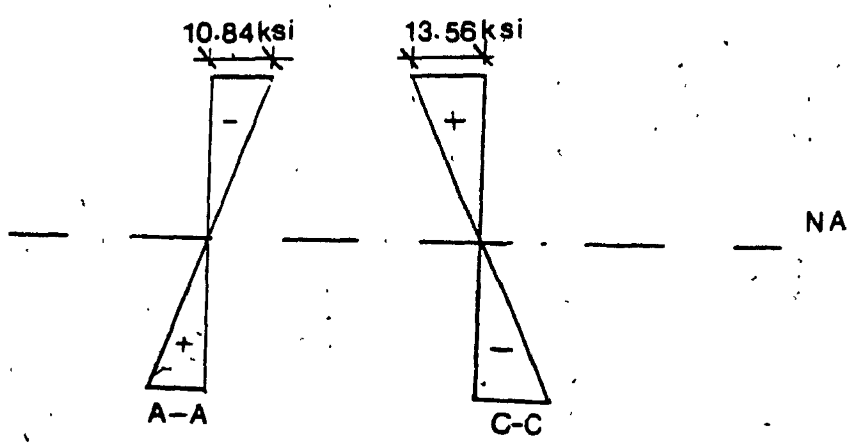


(DL + LL + PR) Stress Diagram

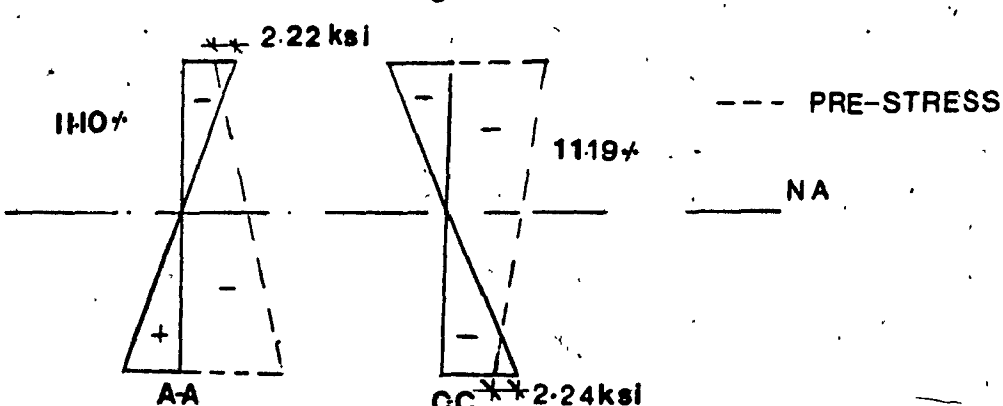
Fig.5.10 Percentage Gain In Strength For Simple Span With Drapped Cables

5.3.3 Continuous Span Beam





(DL + LL) Stress Diagrams



(DL + LL + PR) Stress Diagrams

Fig.5.11 Percentage Gain In Strength For Continuous Beam

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

After analytical and experimental investigations of the problem it is concluded that plate girder bridges can be strengthened up to 10 to 14 percent of the allowble capacity. The gain in strength depends upon the position of the cables with respect to kern of the section. The plate girder bridges can strengthened more by increasing the eccentricity. The other conclusions for each test beam are as follows;

6.1 SIMPLE SPAN BEAM

6.1.1 Straight Cable Configuration:

a) The cable need not be taken to the end of the beam (Fig. 6.1). The cable length depends upon the type of loading.

b) If the cables are to be taken to the end, the following configurations as shown in Fig. 6.2 should be used.

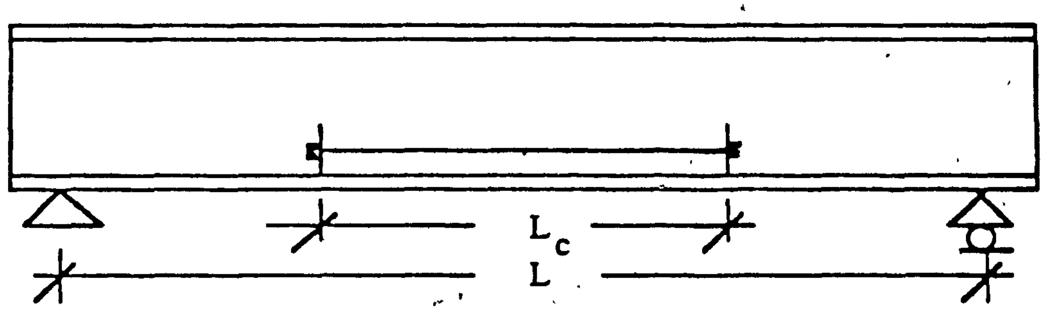


Fig. 6.1 Provision of Shorter Cables for Simple Span Beam.

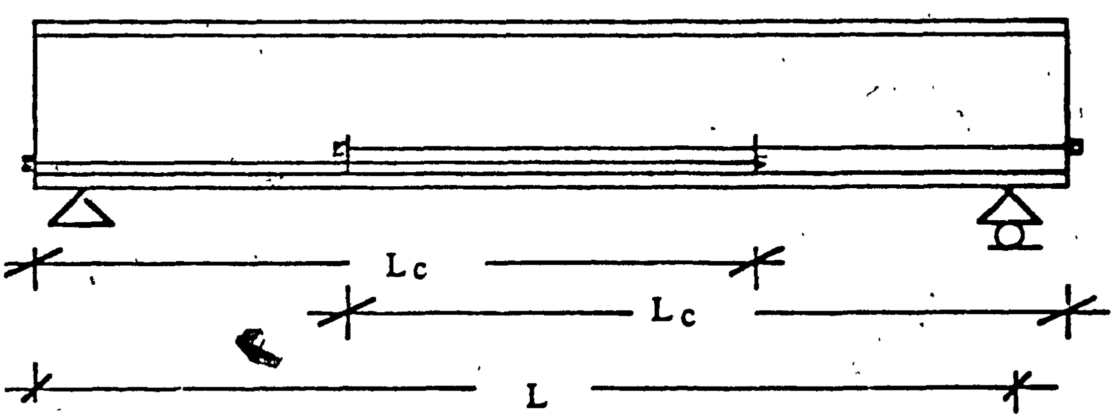


Fig. 6.2 Provision of Two Cables for Simple Span Beams.

c) Straight cables give maximum pre-stressing increment force under applied load.

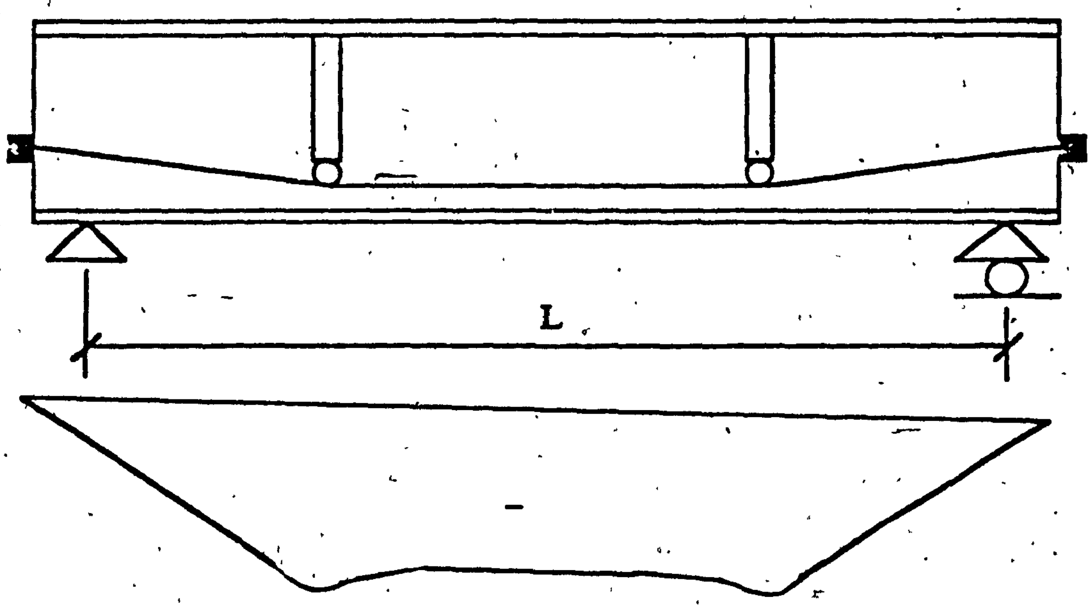
d) The straight cables are recommended for use in the strengthening of existing bridges.

6.1.2 Drapped Cable Configuration :

Drapped cable configuration as shown, follows the bending moment diagram for dead load shape is the best configuration but for the following points are of interest :

a) The increment of pre-stressing force is less than for the straight cable configuration by about 20 percent meaning that the pre-stressing force requirement is more in drapped cable configuration.

b) The bending moment due to the pre-stressing force does not follow the same configuration as required. It follows somewhat the shape shown in Fig. 6.3.



Lower Flange Stress Diagram

Fig. 6.3 Lower Flange Stress Diagram for Drapped Cable Configuration.

The stresses at point of pulley axle are about 15 percent more than expected. This is due to local action as explained below. It is recommended that if drapped cable shape is to be used, then the pulley or saddle should have enough surface area so that it can distribute the forces evenly. In addition, so that it does not produce great local forces as in case of pulleys. However, the local effect has been noted to be less at the top than at the bottom because the stiffeners were welded to the web and the stresses are transferred to the beam.

6.2 CONTINUOUS BEAM

a) The increment of pre-stressing force is more than that for the simple span beam meaning that the less pre-stressing force is required.

b) The losses due to friction are more at angles of bent. More pre-stressing force is required to overcome the frictional losses. However, wax and steel strips were provided to minimize the friction loss as shown in Fig. 6.4

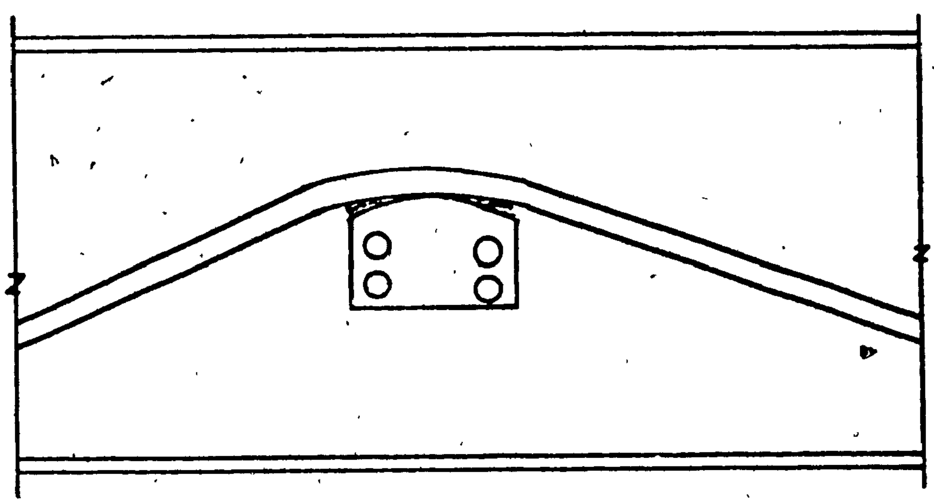


Fig. 6.4 Wax, Steel Stirrup Arrangement at Central Support for Continuous Beam.

c) The cable arrangement shown in Fig. 6.5 has been recommended for continuous beam.

d) The releive of compressive stresses at central support are less as compared to the relieve of tensile stresses. High pre-stress forces are required to strengthen the section at the central support.

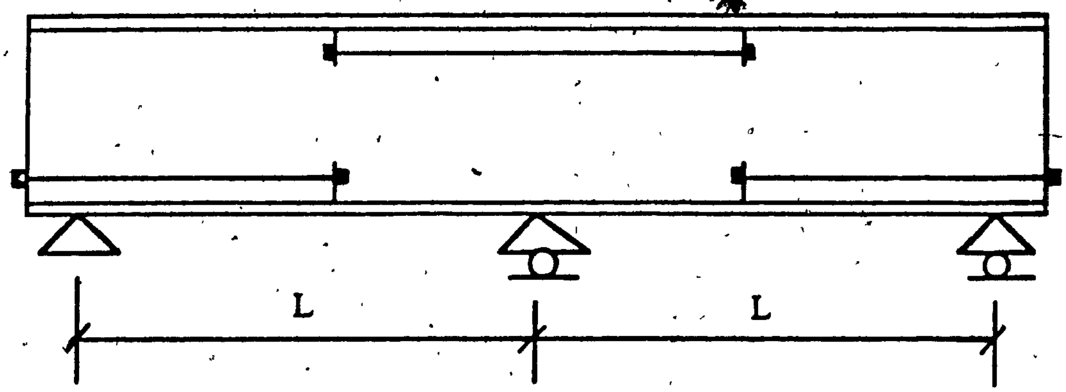


Fig. 6.5 Recommended Cable Configuration for Continuous Beam.

6.3 RECOMMENDED STRENGTHENING PROCEDURE

a) Calculate dead load and live load stresses.

b) Find the equivalent approximate pre-stressing required to relieve the required stress by the following formular.

$$P = \frac{\sigma_{T\,or\,B}}{\left[\frac{1}{A} + \frac{ey}{I}\right]} \qquad (6.1)$$

c) Calculate the increment of ΔP for the applied loads.

d) Check the stresses in the strengthened girder by the following equation:

$$f = + \frac{M(DL + LL)y}{I} - \frac{(P + \Delta P)}{A} + \frac{(P + \Delta P)ey}{I} \qquad (6.2)$$

e) Check for deflection.

f) Check for shear.

g) Check for buckling.

REFERENCES

1.Beauchamp, J. C., Chan, M. Y. T. , and Pion, R. H., " Repair and Evaluation of a Damaged Truss Bridge - Lewis.Yukon River ", Canadian Society for Civil Engineers, June 1-3, 1984, pp. 494 - 504.

2.Belenya, E. I., " Prestressed load bearing metal structures", translated from Russian by I. V. Savin, Mir Publishers, Moscow, 1977, p. 464.

3. Cheung, M. S. , Beauchamp, J. C., Hendrick, G. J., " Concrete Bridge Decks- Design Construction and Performance", Research Report, Ministry of Highway and Communication of Ontario, 1978.

4. Dunker, K. F., " Strengthening of Simple Span Composite Bridges By Post- tensioning", PhD Thesis, Iowa State University, Iowa, March 1985.

5. Girkmann, K.,"Einfuhrung in Die Elastostatik der Sheiben, Platten, Schalen und Faltwerke", in German, 1959.

6.Kandall, C., "Increasing the Load - Carrying Capacity of Existing Steel Structures", Civil Engineering, ASCE, October, 1968, pp. 48 - 50.

7. Morice, P.B., "Linear Structural Analysis" Hudson, England, 1959.

8. Troitsky, M. S. , Zeilinski, Z. A., and Pimprikar, M. S., "Rehabilitation of Steel Bridges by Pre-stressing Technique", Paper presented at the Annual Conference of the Canadian Society of Civil Engineers, May 27 - 31, 1985, pp. 23 - 45.

9. Verigora, E., Maral, J.R.M, Slater, W.M and Aikon, R. V., "Bridge Rehabilitation and Strengthening by Continuous Post-tensioning", Journal of Pre-stressed Concrete Institute, April, 1969, pp. 88 - 105.

10 " Assessment and Evaluation of Some Common Types of Existing Bridges", Roads and Transportation Association of Canada, 36th Annual Conference, Vancouver, October, 197

11. " Pre-stressed Steel Makes A Bridge", Engineering News Record, November, 1964, pp.24 -25

12. Zielinski, Z. A, "Dwa przyktady zastosowania w Polsce sprezenia do wzmacniania istniejacych konstrukcji (Two examples of uses of post-tensioning of existing structures in Poland)", Inzynieria i Budownictwo, No. 1/1956, (in Polish).

APPENDIX 1

DESIGN OF PROTOTYPE GIRDER (SIMPLE AND CONTINUOUS SPANS)

A1.1 Simple Span:

Simple span bridge 60 ft span, with two lanes was designed according to American Standard Specifications for Bridges. The design criteria followed was that of superimposing all the stresses due to different loads acting on the bridge and then checking the total stresses according to specifications. A concrete deck in conjunction with a plate girder was designed.

The prototype girder in elevation and cross-section is shown in Figs. A1 and A2 respectively.

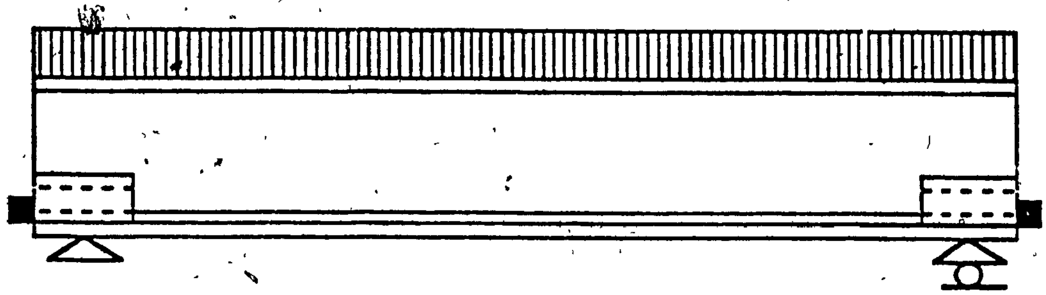


Fig. A1 Elevation of Prototype Girder (Simple Span)

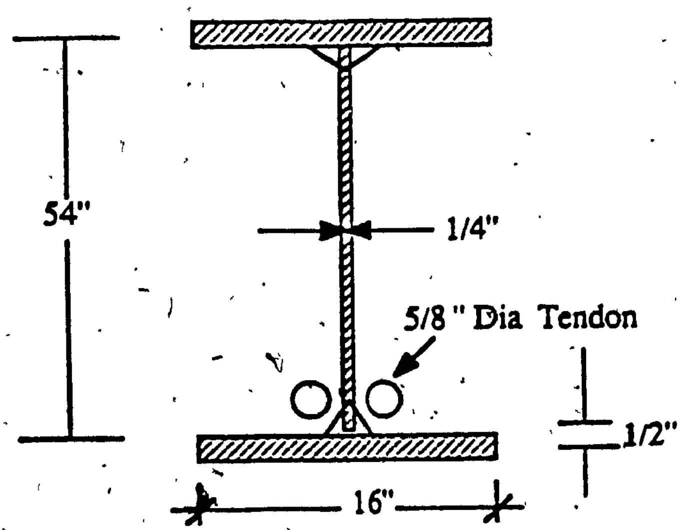


Fig. A2 Sectional View of Prototype Girder (Simple Span)

A1.2 Continuous Span:

The two span continuous bridge, 60 ft each, was designed for maximum negative moment at the central support. The stresses were checked according to specifications. The prototype girder in elevation and cross-sectio is shown in Figs.A3 and A4.

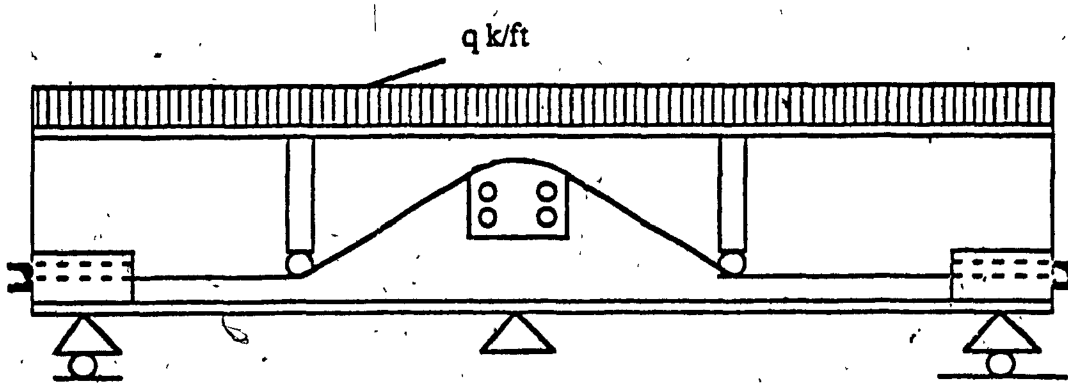


Fig. A3 Elevation of Prototype Girder (Continuous Span)

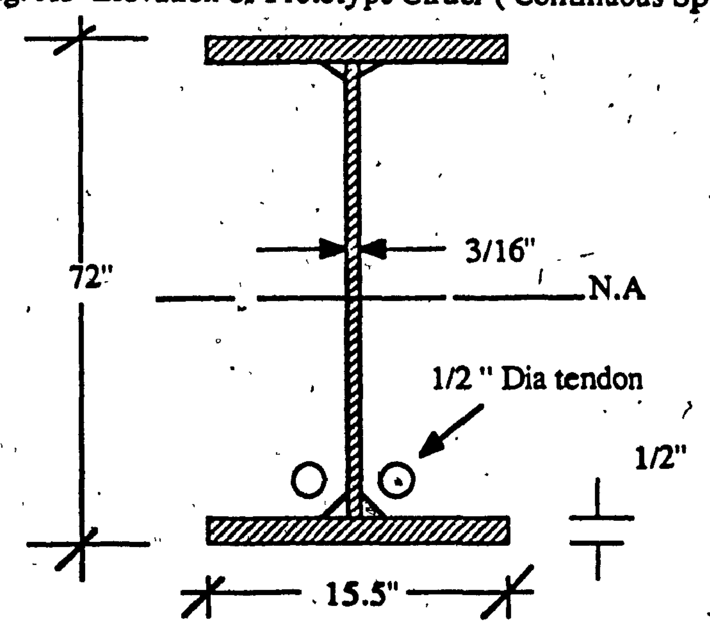


Fig. A4 Sectional View of Prototype Girder (Continuous Span)

APPENDIX 2

CORRECTIONS FOR BENDING AND AXIAL STRAINS

A2.1 Simple Span:

The similitude conditions were applied to prototype girder in order to get the model beam. The values of different reduction factors are as follows:

$K = 3.75$; $Z = 0.287$.

By applying these reduction factors, the dimensions of the model beams were determined. The different corrections were applied to the measured strain on the model beam since it was difficult to obtain the exact section in the market. The following corrections should be applied in order to meet the requirements of model beam used for the tests;

a) Correction for Bending Strain:

i) Correction for depth $= \bar{d}/\bar{d}_a$

where, d = ideal depth of the beam ; and d_a= actual depth of the beam.

Putting the values gives the correction for depth =0.91.

ii) Correction for Shift of Neutral Axis = $\bar{C}/\bar{C}_a$

where, $\bar{C}_a$ = actual position of the neutral axis; and $\bar{C}$ = ideal position of the neutral axis.

Putting the values gives the correction for the shift in neutral axis =0.91.

The correction factor $C = 0.826$ and the loading slicing factor Y is 1.61

b) Correction for Axial Strain:

i) Correction for axial strain = $\bar{A}_a/\bar{A}$

where, Aa = actual cross-sectional area of beam ; and A = ideal cross-sectional area of beam.

Putting the values gives the correction for axial strain of 10.6/11.15 = 0.95 .

A2.2 Continuous Span:

For continuous beam the values for different reduction factors are:

K = 5.0 and Z = 0.091.

By applying the reduction factors, the model beam dimensions were determined. Corrections should be applied to the measured strain as explained earlier. In this case, the various corrections are;

a) Correction for Bending Strain:

i) Correction for depth $=\bar{d}/\bar{d}_a$

Putting the values gives the correction for depth of 0.91.

ii) Correction for shift in neutral axis = $\bar{C}/\bar{C}_a$

This value is obtained as 0.91.

b) Correction for Axial Strain:

i) Correction for axial strain $=\bar{A}_a/\bar{A}$

Putting the values gives the correction for axial strain of 10.6/ 12.66 = 0.837

Correction factor for continuous beam = 0.837.

The load slicing factor is Y=1.319