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ANALYSIS OF CABLE STAYED BRIDGES

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Submitted in fulfilment of the requirements
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in
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By
TEJ PRAKASH AGRAWAL

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DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF ROORKEE
RCORKEE U.P. (INDIA)

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CERTIFICATE

Certified that the thesis entitled 'ANALYSIS OF CABLE STAYED BRIDGES' which is being submitted by Sri T.P. Agrawal in fulfilment of the requirements for the award of the degree of Doctor of Philosophy in Civil Engineering of the University of Roorkee, Roorkee, is a record of student's own work carried out by him under our supervision and guidance. The matter embodied in this thesis has not been submitted for the award of any other degree.

This is further certified that he has worked for a period of 3 years and 3 months from August 1973 to October 1976 for preparing this thesis.

Premkrishna

(Prem Krishna)
Professor of Civil Engineering
University of Roorkee, Roorkee,
India.

A S Arya

(A.S.Arya)
Professor
Department of Earthquake Engg.,
University of Roorkee, Roorkee,
India.

University of Roorkee, Roorkee
Certified that the attached Thesis/Dissertation has been accepted for the award of Degree of Doctor of Philosophy / Master of Engineering Civil vide notification No. Ex/30|G191 (Degree) dated 18.4.80

Assistant Registrar (Exam.)

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ABSTRACT

The available work on cable stayed bridges gives very little information about the non-linear and the three-dimensional behaviour of these bridges. There are also no suitable guidelines for the designer to select the dimensions and the sectional properties of the elements of the bridge. The work contained in this thesis is a step towards providing some of the above information.

The investigations embodied in this thesis include linear as well as large deflection analyses of certain chosen cable bridge systems. The analytical results are validated by verification with the results of load tests on a bridge model. The work also includes a study of bridge behaviour under certain idealised loading cases and the influence of variation in the stiffness and geometrical parameters of the system.

The thesis proceeds to present charts based on the above study, which should be helpful to the designer in choosing design parameters. Another important study contained herein is concerned with the erection stresses of a radiating type cable stayed bridge when constructed by the double cantilever method. A comparison of the results as obtained by three-dimensional and two-dimensional linear analyses is made which shows that the values of forces and deflections given by the space frame analysis are smaller for unsymmetrical loads and are almost equal for symmetrical loads.

The effect of large deflections on the behaviour of the bridge is investigated. A non-linear plane frame analysis of the radiating, harp and star arrangements is carried out, in which the effect of variation in the girder flexibility and the support conditions at the tower base is also investigated. Nonlinear space frame analysis is carried out for the harp and radiating arrangements. It is found that the nonlinearity due to large deflections is generally very small. The nonlinear values of the forces and deflections are smaller than the linear values. As may be expected, the nonlinearity increases with the flexibility of the structure.

Investigations into the behaviour of cable stayed bridges during erection are carried out in two parts. In the first part, the study is concerned with the effect of variation in panel lengths on the girder moments. Two bridges each with 36 cables are analysed, the panel lengths being equal in one case but unequal in the other. This study indicates that in the initial stages of erection, the girder moments, cable tensions and the vertical deflections are smaller in the bridge with unequal panel lengths but in the later stages, these forces and deflections become larger than those in the bridge with equal panel lengths. In the second part of the study a sequence is established to reduce

the girder moments by prestressing the cables as the erection proceeds. The analysis of bridges with 12, 20, 28 and 36 cables shows that the girder moments during erection are reduced considerably. The final girder moments in all the above cases become very close to those in a continuous beam with non-deflecting supports.

In the chapter on parametric study, the effects of various parameters viz. stiffness of cables, girders and towers; number of cables; length of the central panel; tower height to total span ratio, and side to main span ratio, on the cable tensions and tower and girder moments are investigated by varying one or more parameters at a time. The main results of this study are the following:

(a) The flexural rigidity of the towers does not affect appreciably the behaviour of the bridge,

(b) An increase in the flexural rigidity of the girders leads to increase in girder moments but decrease in girder deflections and cable tensions.

(c) With an increase in cable rigidity the cable tensions increase while the girder moments and deflections decrease.

(d) The effect of change in the length of the central panel on the cable tensions and the hogging girder moments is small. The positive girder moments are much smaller in bridges with smaller central panel length as compared to those with larger length of the central panel.

(e) The sagging girder moments and the cable tensions decrease with an increase in the tower height. The effect of the tower height on the hogging girder moments is small.

A comparison of the behaviour of the harp and radiating arrangements shows that the cable tensions, girder moments and deflections are larger in the case of harp arrangement as compared to radiating arrangement. Consequently the cost of cables and girders as well as their combined cost is higher in the harp arrangement as compared to the radiating arrangement.

Load tests are carried out on a perspex model of the proposed second Hooghly bridge. These show that the load deflection relationship is, in general, linear. The difference in the theoretical and experimental values of the deflection is less than 5 per cent, the theoretical values being smaller. The cable tensions show wider variation.

Finally design curves incorporating the variation of the various parameters are plotted for the maximum tension in the cables, and the positive and negative girder moments.

The radiating arrangements with 12, 20, 28 and 36 cables are considered. The side to main span ratios are taken as 0.35, 0.40, 0.45 and 0.50 and the tower height to the total span ratios as 0.075, 0.100 and 0.125. A stiffness parameter relating the extensional stiffness of cables to the flexural stiffness of girders is introduced and two values of this parameter, viz. 83000 and 62000 are used in drawing the design charts.

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NOTATIONS

A	Cross sectional area of the member
A_c	Total area of cables in one plane
E_c	Modulus of Elasticity of cable
E_G	Modulus of Elasticity of the girder
G	Modulus of rigidity
h_t	Height of the tower
I	Moment of inertia of member
I_G	Moment of inertia of the girder
I_x or J	Torsional constant (Polar moment of inertia)
I_y	Moment of inertia for bending in XZ plane
I_z	Moment of inertia for bending in XY plane.
L_r	Length of the side span
L_M	Length of the main span
L_T	Total length of the bridge
l_1	Length of one panel in the side span
l_2	Length of one panel in the main span
l_c	Length of the central panel
n	Number of cables
w	Intensity of the uniformity distributed load over the bridge
λ	A stiffness parameter

$$\left(\lambda = \frac{E_c \, A_c \, L_T^{\,2}}{E_G \, I_G} \right)$$

Note:- Other notations are explained wherever they are used.

CHAPTER I

STATEMENT OF PROBLEM AND SCOPE OF STUDY

1.1 GENERAL

The development of the modern form of cable stayed bridges has taken place during the last two decades, although a wooden structure, very similar to present day cable stayed bridges, was proposed as early as in 1784 by a German Carpenter, C.J. Loscher.

Probably due to failure of some early cable bridges, their development was neglected. The modern cable stayed bridge was rediscovered by Dischinger in 1955 when the Stromsund bridge was completed. Germans are the pioneers in this field. The statistics available show that more than one third of total bridges of this kind in the world have been built in Germany alone.

Since 1955 many investigators have taken interest in the field of research and development of cable bridges. The detailed study of the existing literature on this type of system reveals that sufficient information is not available for their efficient, economical and rational design. For the purpose of analysis, so far, the system is assumed to be a planar structure and the effect of lateral loads due to wind has not been considered rigorously. Little information is available regarding the behaviour of these bridges considering large deformations. Similarly not much has been reported by way of experimental verification of the theoretical results. It would be seen that there are many areas of investigations which have been covered only partly, or, not at all, and thus further investigations are called for. This investigation will essentially be another step towards a more rational analysis of cable stayed bridges. The problem is of interest in India due to the proposal of a bridge of this kind across the river Hooghly at Calcutta.

1.2 PROBLEM STATEMENT AND OBJECTIVES

The problem proposed for investigation is mainly divided into five parts as follows:

1.2.1 Behaviour under Lateral Loads

Lateral loads, caused due to wind and earthquake, act in the horizontal direction perpendicular to the longitudinal axis of the bridge. The analysis of cable stayed bridges for lateral loads involves three dimensional behaviour of the structure. The stiffness matrix method shall be employed by treating the structure as a space frame. Sufficient number of typical cases will be analysed to show the extent of space action.

1.2.2 Effect of Large Deflections

Present methods of design of cable stayed bridges assume the deflections of the towers and girders to be small, but in some cases these deflections may be large enough to cause a nonlinear behaviour of the structure and the results obtained by linear analysis may be in error. Hence it is proposed to carry out a nonlinear elastic analysis to account for the large deflections so as to evaluate the difference in stress resultants as obtained from linear elastic analysis.

1.2.3 Stresses during Erection

The analysis of stresses and deflections in the various components of the bridge in different stages of construction is important particularly in prestressed structures. The same is carried out herein by assuming that the construction will be done by double cantilever method starting from each tower on both sides of it simultaneously.

The study will include various cases of radiating type double plane bridges with different number of cables. It is also proposed to determine the cable spacing to obtain the most economical arrangement as far as stresses during erection stages are concerned.

1.2.4 Preparation of Design Curves

It is proposed to prepare a set of design curves, suitable for design office use, showing the effects of different design parameters. The design parameters to be considered in this case will be the same as in the case of stresses during erection stages. The analysis will be carried out by the stiffness matrix method, treating the bridge as an equivalent plane frame.

1.2.5 Model Study

In order to justify the validity of the assumptions made in theoretical analysis, the theoretical results are proposed to be compared with those obtained experimentally. For this purpose the tests would be carried out on a model of the proposed Hooghly bridge.

1.3 SCOPE OF STUDY

Two different approaches are used for the analysis of cable stayed bridges. One approach assumes the structure to be a plane frame while the other takes into account the space action when subjected to lateral loads due to wind or earthquakes. In both these analyses, the stiffness matrix method of analysis is used and the formulation is extended to account for the behaviour under large deformations. For this purpose Newton-Raphson method of iteration is adopted. Separate computer programmes are developed for the plane frame and the space frame analyses. The programme for linear elastic analysis, assuming the structure to be a plane frame, is further modified to generate the coordinates of the nodal points for different erection stages and for preparation of design charts.

The linear analysis, treating the structure to be a plane frame, is used to obtain stresses in the erection stages and preparation of design charts for a radiating type bridge. The following parameter values are used for computing results for design charts.

Total number of cables	12,20,28,36
Side/main span ratio	0.35, 0.40, 0.45, 0.50
Tower height/bridge length ratio	0.075, 0.100, 0.125
Dimensionless parameter λ	62000, 83000

The linear and nonlinear elastic analyses, treating the structure as a space frame, are used to get the effect of space action and the analysis for lateral loads.

The linear and non-linear results for the radiating, harp and star arrangements of cables are compared. The towers are taken as fixed or hinged at the base. The girders are assumed as on roller supports at the towers.

These bridges, in general, are designed for equivalent uniformly distributed loads. The dead and vehicular loads act in vertical direction, while the loads due to wind and earthquake act in lateral direction. The cable stayed systems considered here are analysed for dead loads as well as superimposed loads acting on part or whole of the span.

1.4 OUTLINE OF THESIS

The statement of the problem and the scope of study have been given in this chapter already. A brief historical review and the state of art are included in the second chapter. The methods of linear analysis and the nonlinear analysis of cable stayed bridges are given in third and fourth chapter respectively. The details of the analysis covering the stages of erection are presented in the fifth chapter. The results of parametric study including the effect of various parameters on the behaviour of the cable stayed bridges are presented in the sixth chapter. The experimental work on a model bridge is described in the seventh chapter. The conclusions obtained from the various investigations conducted herein are presented in detail in the eighth chapter.

CHAPTER II

CABLE STAYED GIRDER BRIDGES-STATE OF THE ART

2.1 HISTORICAL REVIEW

The evolution of 'Cable-Stayed Bridges' or 'Tied Cantilever Bridges' has taken place in the last two decades, although a similar concept was proposed as early as 1784 by C.J. Loscher, a German carpenter in Friborg. Around 1840, Hatley an Englishman, built a bridge with stays arranged as in a harp (62,196). Many early bridges were constructed with chains or round bars as the stays. In 1873 the Albert bridge over river Thames was constructed in which all inclined tie members converged to the top of the tower (90). This also reflects the same idea. Unfortunately, due to lack of technical knowledge and methods of analysis of such bridges (a highly indeterminate structure) and non-availability of appropriate construction technology, some of the early bridges failed.

In 1938 a German Engineer F.Dischinger rediscovered cable stayed bridges. Following his lead, the modern cable-stayed bridges have been developed after 1955. Stromsund bridge(216) and North bridge at Dusseldorf (27) are the examples of bridges of this type.

It may be said that the development of cable stayed bridges has originated in West Germany as a result of large scale destruction of bridges over Rhine river and innumerable other bridges during World War II. This required to economise bridge construction techniques. About fifty bridges of this type have been constructed all over the world and almost one third of these are in Germany the distribution is given in Table 2.1.

Table 2.1

Present Distribution of Stayed Bridges over the World(151)

Country	No.of Bridges	Country	No. of bridges
Germany	17	Holland	2
United States	8	USSR	2
Japan	7	Austria	1
Canada	4	Denmark	1
Great Britain	3	India	1
Italy	3	Libya	1
Argentina	2	Sweden	1
Australia	2	Venezuela	1
France	2	Zambia	1

In India two bridges of this type were proposed, one across river Ganga near Allahabad and the other across Hooghly near Calcutta. Due to certain difficulties Allahabad bridge was finally replaced by an ordinary balanced cantilever bridge. Details of the proposed design for the second Hooghly bridge are shown in Fig.2.1. The clear span between the two towers is 457.2 meters. There are four pendulum type supports and two main towers. The centre to centre distance between the supports is 91.44 m. The total width of the deck is 34.5 m while the distance between centres of box girders is 27.5 m and there are six traffic lanes. The tower has three cross braces - one below the deck level and two above it. The total height of tower is 105.3 m.

2.2 STRUCTURE ARRANGEMENTS

Many factors affect the choice of the structural arrangement for a bridge. Some of the important factors are, the soil available in the river bed, the economical span length, the width of the navigation space required etc. The quality of the structural arrangement also depends upon the ingenuity of the designer.

A cable stayed bridge can have various alternatives in the transverse and the longitudinal arrangements which are discussed hereafter.

2.2.1 Cable Arrangements

A large number of variations are possible in the arrangement of cables. Many factors such as, main span

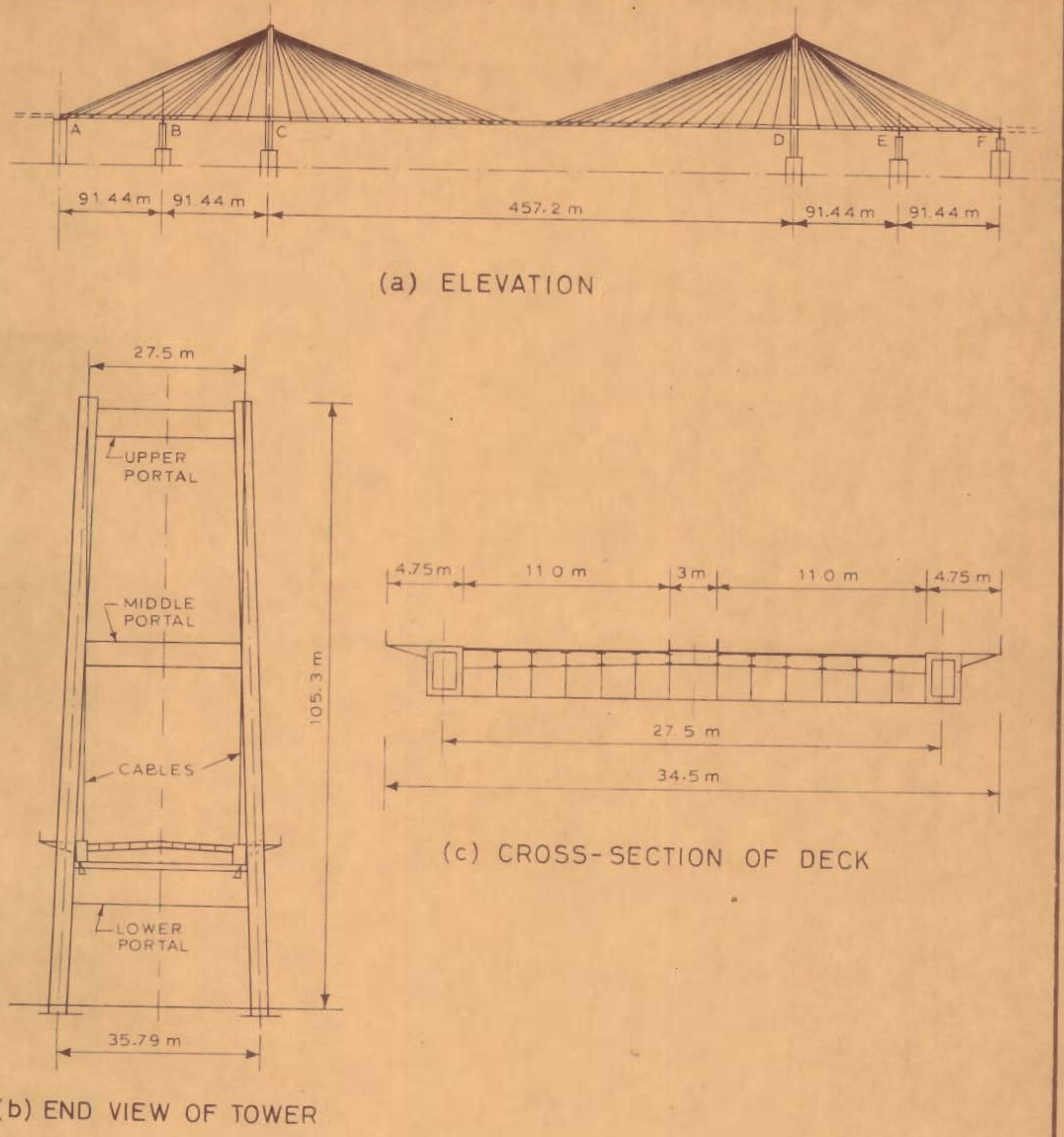


Fig. 2.1 Proposed Second Hooghly Bridge.

length, width of the roadway, vehicular and wind loads etc. affect the choice of both longitudinal and transverse cable arrangements. Various types of transverse and longitudinal arrangements are discussed in the following paragraphs.

2.2.1.1 Transverse Cable Arrangements

Figure 2.2 shows the various transverse arrangements of cables in the stays. Cable stayed bridges with cables only in one plane are suitable for smaller spans with narrow roadways such as pedestrian bridges. In this case the towers and the ties are provided in the median strip of the roadway (Fig.2.2(a)). As the loads are to be carried by the cables in one vertical plane only, the size of cables becomes larger. The half roadway cantilevers from the central longitudinal girder and large amount of torsional moment may develop in the main girder. In some cases the stays are provided at one side of the roadway (Fig.2.2(b)).

The two-plane systems of cable stays are suitable for large span bridges and several traffic lanes. In this case two or more longitudinal girders are provided with cable stays at the sides (Fig.2.2(c)). Since the cable forces are carried in two planes, the size of the cables will be small and the torsional moment is absent as the cross beams are simply supported over the main beams.

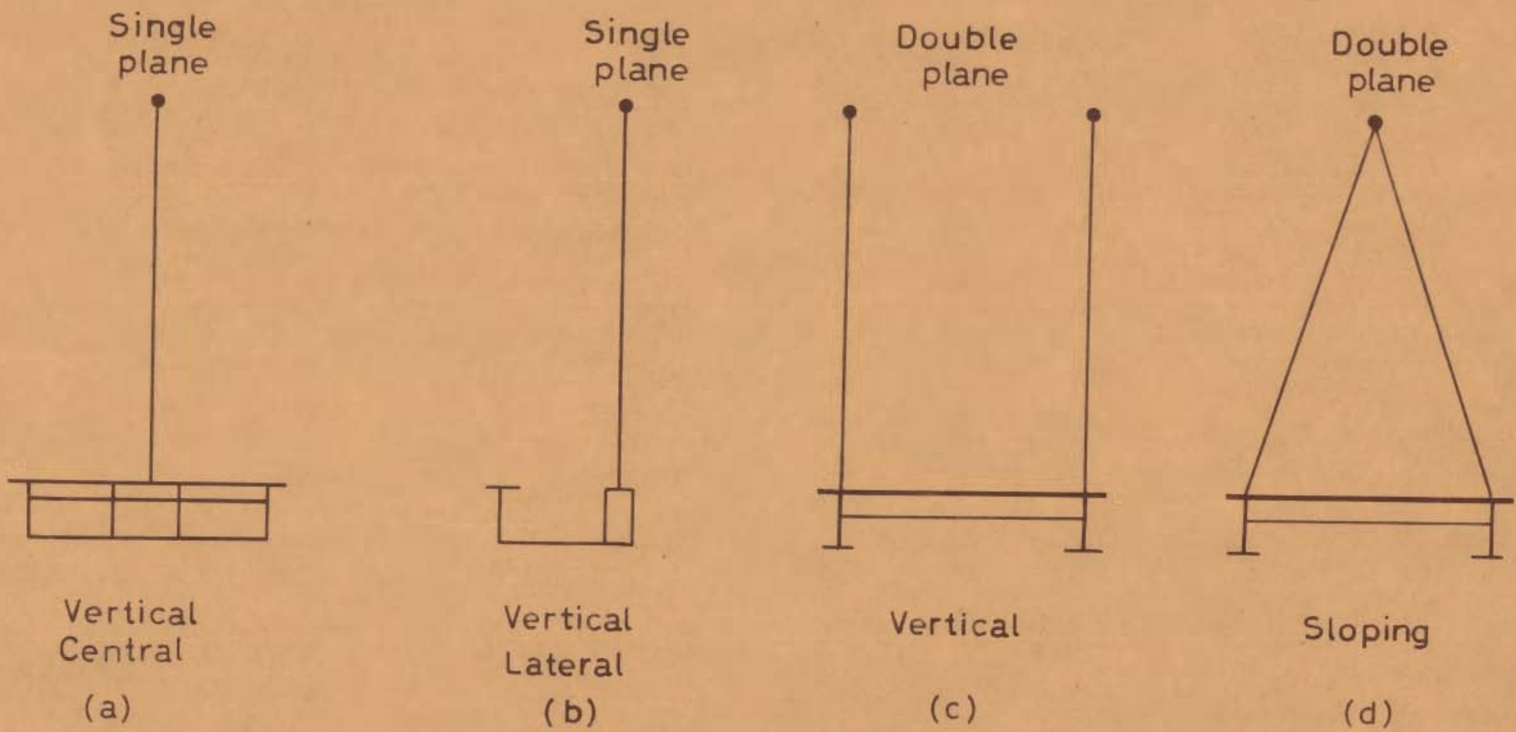


Fig. 2.2 Transverse Cable Arrangement

The cable arrangement with ties in two planes inclined towards each other is also used (Fig.2.2(d)). In this arrangement the towers are of A-type frames and it is structurally better with regards to aerodynamic stability.

2.2.1.2 Longitudinal Cable Arrangements

Cable stayed bridges with one cable on either side of the tower are suitable for short spans. Bridges with multiple ties are more efficient for larger spans.

The types of cable arrangement mainly used are the

(i) Radiating type,
(ii) Harp
(iii) Star, and
(iv) Fan.

In the radiating type of cable arrangement, all the cables converge to the tower tops (Fig.2.3(a)) while in the harp type, the cables are parallel to each other and are distributed along the height of the tower (Fig.2.3(b)). In the star arrangement, the cables are distributed over the entire height of tower and converge to a single point on the deck (Fig.2.3(c)), while in the fan arrangement groups of cables are connected at various points of the tower (Fig.2.3(d)).

In the radiating arrangement, cable forces are higher and bending moments in girders are smaller. In the

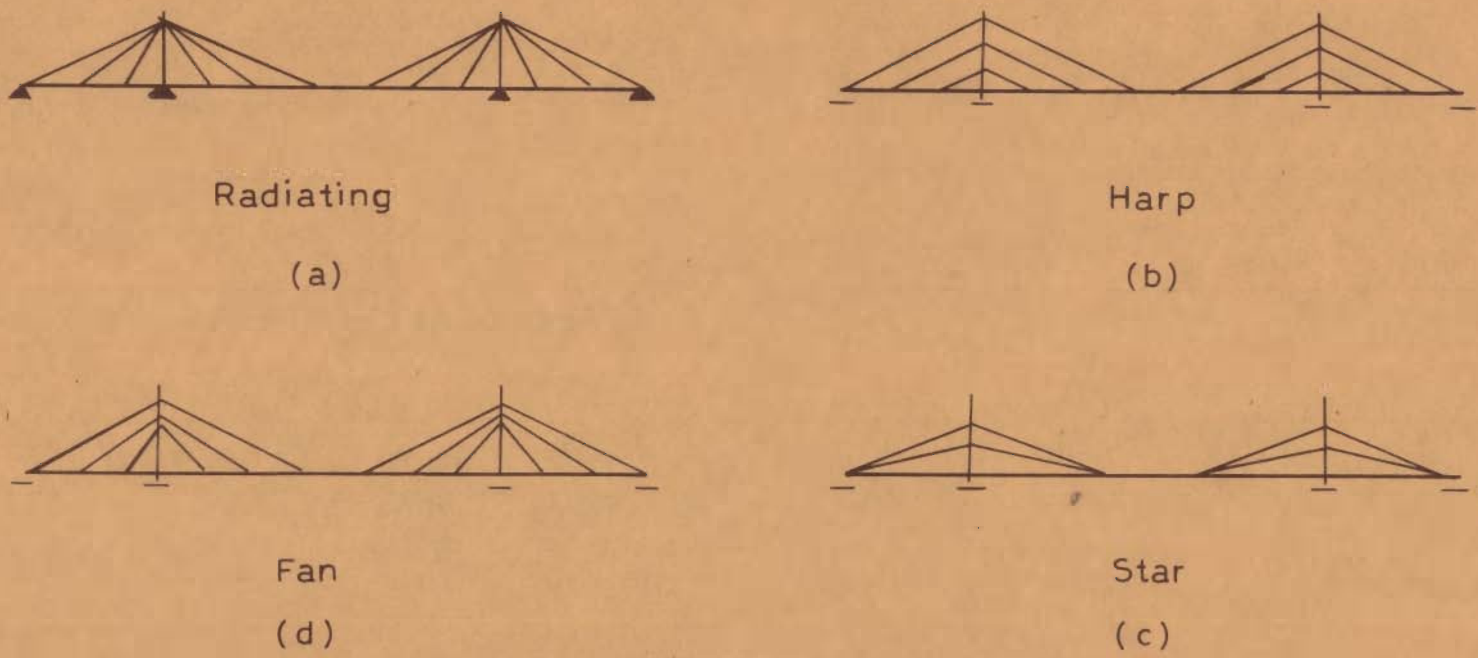


Fig. 2.3 Longitudinal Cable Configuration.

harp arrangement, cable forces are smaller and girder moments are larger. The star and fan arrangements are unlikely to be economical on account of the possibility of larger girder moments as compared to the former two types. The latter, however, have a pleasing aesthetic appearance.

2.2.2 Towers

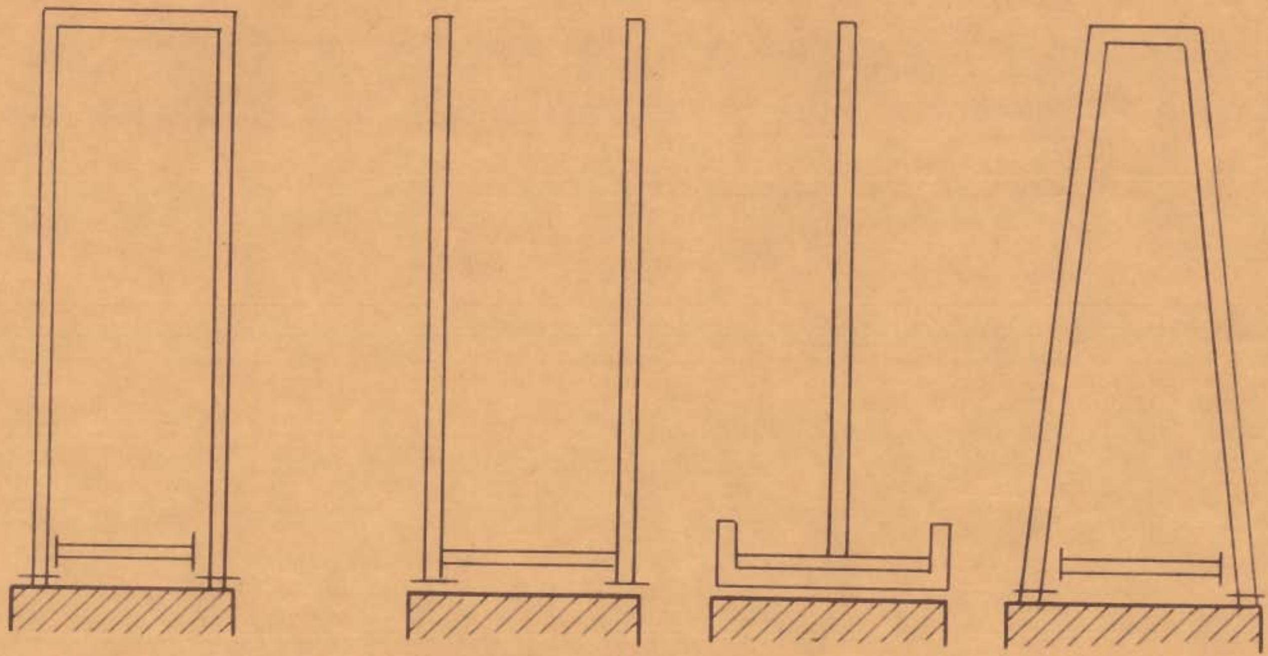
The load from the deck is transferred to the towers through cables as well as directly from the girders and large compressive forces are developed. Towers can have any of the following forms:

(i) Rectangular portal frame either fixed to the superstructure or to pier (Fig.2.4(a)).

(ii) The top bracing in towers of type 1, may or may not be provided as large lateral displacements of tower tops are restricted by tension in cables.

(iii) Single towers are either fixed to the girder or to the pier-the former type is shown in Fig.2.4(b). In case of the axial girder system the tower is generally fixed to the girder.

(iv) The towers can be in the form of trapezoidal portal frame as shown in Fig.2.4(c).



(i) With top Bracing (ii) Without top Bracing

(a) Rectangular Portal Frame Towers (b) Single Tower (c) Trapezoidal portal Frame.

Fig.2.4 Various Types of Towers.

The side elevation of towers may be a single column or an A-frame. The single tower works well at places where a sound foundation is available. In case of piers on alluvial soils a slight settlement of foundation may cause large girder moments and can even reverse the nature of these moments. All these drawbacks are fully corrected in the structural arrangement shown in Fig.2.5.

2.2.3 Deck

Various types of bridge decks have been employed in cable stayed bridges. The types of bridge deck cross sections which have been employed are illustrated in Fig.2.6. The usual form is one having two main I-shaped girders or multiple I-shaped girders, all of which have low torsional rigidity. Also there is single rectangular or trapezoidal box girders which may be either single or multi-celled. These box girders may be with or without side cantilevers. In some cases twin rectangular or twin trapezoidal box girders have been used.

The current trend is to provide orthotropic plate decks with box girders, especially the trapezoidal box girder with side cantilevers. The latter arrangement is superior to other types of sections in regards to aerodynamic stability. A further advantage of box shaped girders is that a larger cross sectional area is provided in the bottom flange as compared to the I-shaped girder configuration. This means that a shallower construction is possible

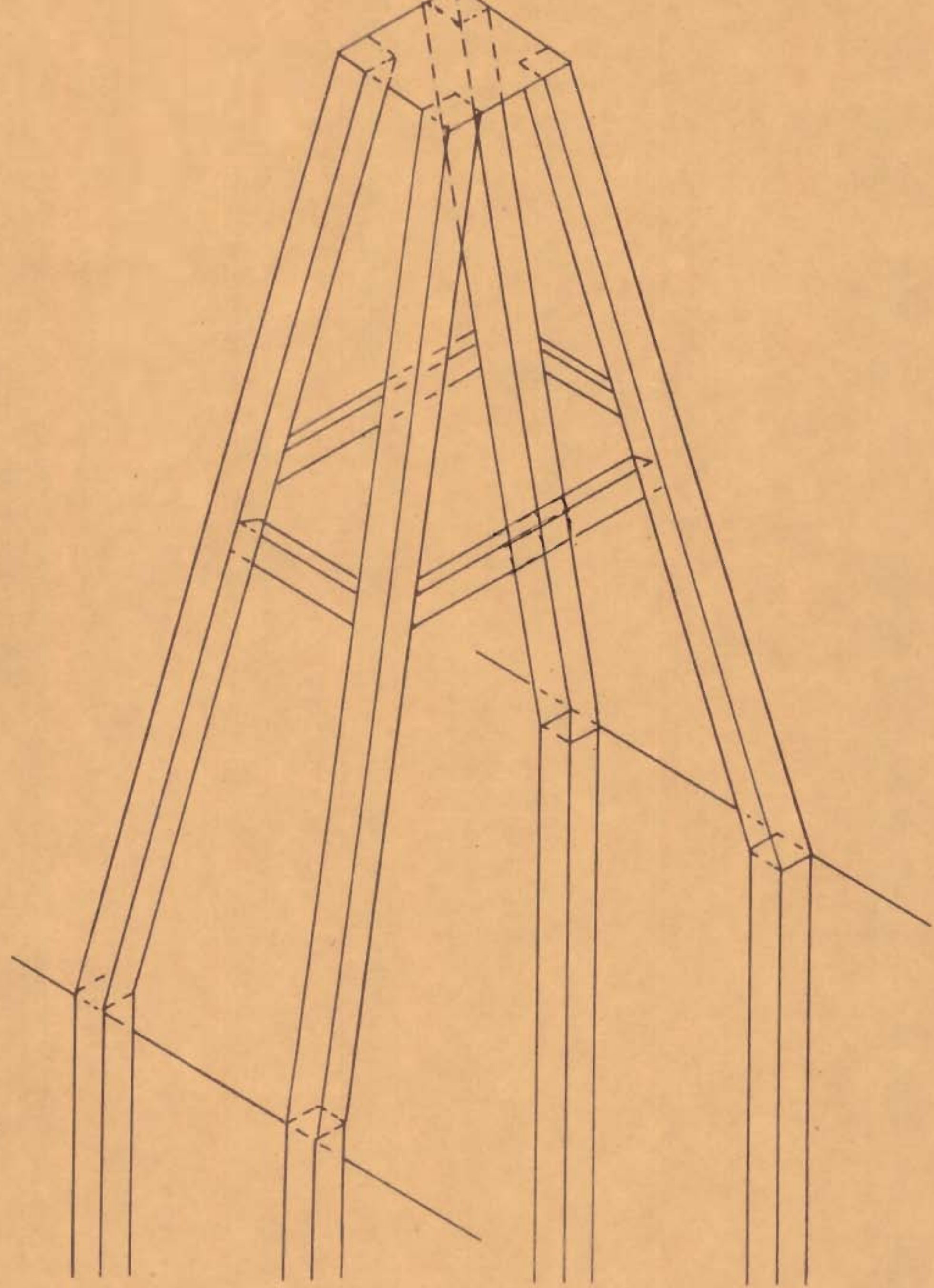


Fig. 2.5 A-Frame Tower.

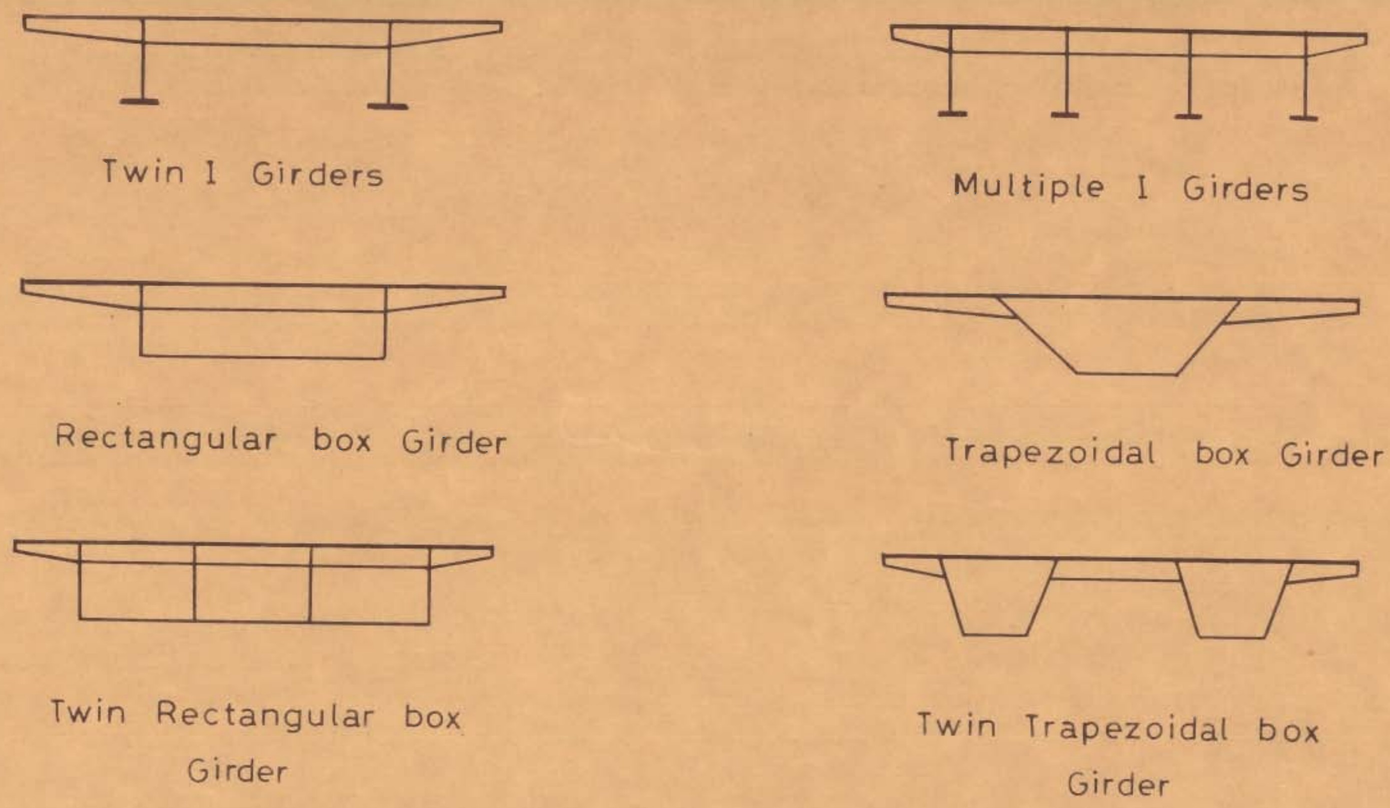


Fig. 2.6 Bridge Deck Cross – Sections.

for box girders leading to a reduction in wind load. Figure 2.7 illustrates the variation in plate girder depth compared to box girder for spans ranging from 30 m to 50 m. The average reduction in depth is approximately 16% or in the proximity of 300 mm for a span of 50 m. A box girder section is to be preferred for cable stayed bridges, especially for a single vertical plane transverse cable arrangement, because of better torsional stiffness. Unsymmetrical live loading and wind forces can produce large torsional forces. The box section has better torsional properties and thus can reduce torsional stresses and rotations in the deck more efficiently.

2.3 DISTINCTIVE FEATURES

The economic analysis of various types of bridges indicates that cable stayed type are suitable in the medium span range and fill the gap between the girder bridge and the suspension bridge. Recent cost studies have shown that cable stayed bridges are found to be in close competition with the suspension bridges even for medium to large span ranges. For long spans, suspension bridges are clearly superior. A comparison of the behaviour of these two bridge forms is given in this section.

(i) In cable stayed bridges the cables are straight as compared to those in suspension bridges, hence the non-linearity due to catenary action is

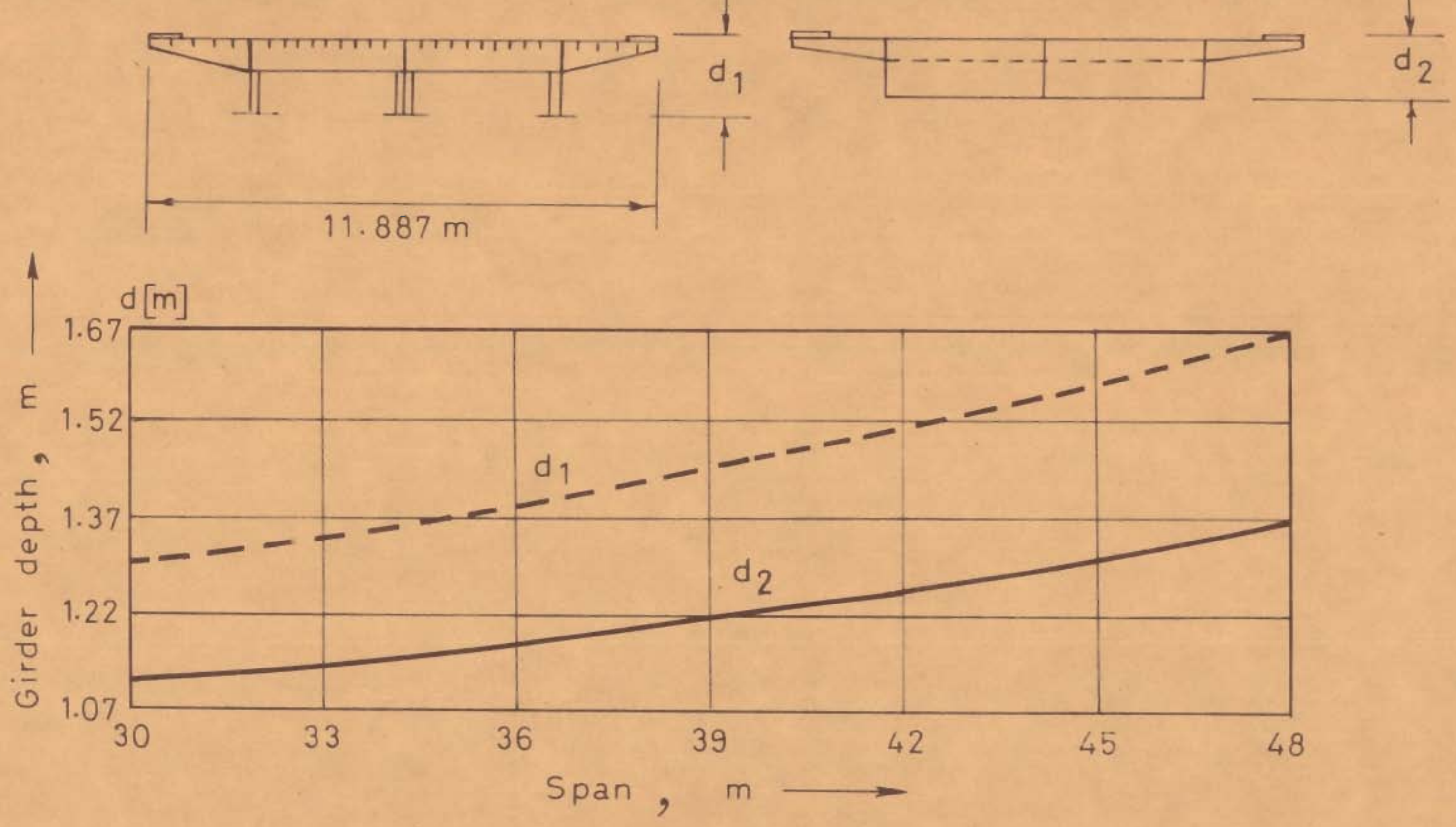


Fig. 2.7 Depth Comparison for Plate and Box Girders.

much smaller.

(ii) The cables are anchored to the deck and it acts as a prestressed member. This is of great advantage specially in concrete decks.

(iii) Presence of cable facilitate erection of the superstructure. The adjustability in cable lengths provides an effective control during construction.

(iv) There is considerable freedom in selection of the structural arrangement particularly when compared with suspension bridges.

(v) Cable-stayed bridges are less effective to support dead loads while more effective in carrying live loads. This leads to restrictions on the economical span range (90) - Refer Art. 2.6.2.

(vi) The natural frequency of vibration of these bridges is in general higher than that of suspension bridges,but lower as compared to conventional girder bridges. Aerodynamic instability has not been a problem in cable-stayed bridges, constructed so far.

2.4 AERODYNAMIC STABILITY

Cable-stayed bridges, being slender are sometimes sensitive to wind oscillations. The manner of vibrations can be broadly classified as (i) flexural or vertical vibrations; (ii) torsional or rotational vibrations; and (iii) the combination or coupling of (i) and (ii). A self induced aerodynamic effect with the combination of the vertical and the torsional vibrations is known as flutter and it is the most dangerous form of vibration which may cause destruction of the structure.

After the failure of Tacoma Narrows bridge, the phenomena of vibrations of suspension bridges have received concerted attention. A popular misconception that wind forces are significant only in long span bridges is obviously incorrect as can be seen from Table 2.2(60) which shows that for spans ranging from 245 feet (74.68 m) to 2800 feet (853.28 m) some bridges have been destroyed or damaged. Some outside this range have had undesirable oscillations (See Table 2.3).

The Golden bridge is a suspension bridge with stiffening trusses. This bridge was subjected to wind vibrations twice. Once the vibrations were with recorded wind velocity of 100 Km/h. In both observations the amplitude of vibrations was estimated at 600 mm (60).

Table 2.2

Suspension Bridges Severely Damaged or Destroyed by Wind

Bridge	Location	Designer	Span (m)	Failure Year
Dryburgh Abbey	Scotland	John and William Smith	79.25	1818
Union	England	Sir Samuel Brown	136.85	1821
Nassau	Germany	Lossen and Wolf	74.67	1834
Brighton Chain Pier	England	Sir Samuel Brown	77.72	1836
Montrose	Scotland	Sir Samuel Brown	131.67	1838
Menai Strains	Wales	Thomas Telferd	176.78	1839
Roche-Bernaard	France	Le Blanc	195.38	1852
Wheeling	U.S.A.	Charles Ellet	307.85	1854
Niagra Lewiston	U.S.A.	Edward Serrell	317.29	1864
Niagra Clifton	U.S.A.	Samuel Keefer	384.05	1889
Tacoma Narrows	U.S.A.	Leon Moisseiff	853.44	1940

Table 2.3

Modern Suspension Bridges which have Oscillated in Wind

Bridge	Year Built in	Span (m)	Type of stiffening provided
Fyksesund (Norway)	1937	228.60	Rolled T-beam
Golden Gate	1937	1280.16	Truss
Thousand Island	1938	243.84	Plate Girder
Deer Isle	1939	329.18	Plate Girder
Bronx White-stone	1939	701.04	Plate Girder

It has been widely accepted that the same theory of vibration analysis, as applied to suspension bridges, may be applied to cable-stayed bridges. The classical theory of vibrations, assumes the structures to be an infinitely thin plate. Although it does not represent the true conditions of the bridge, it has been found that this difference can usually be compensated by modifying the calculated critical wind velocity by suitable form factors (193). The form factor depends mainly on the shape of the cross-section of the bridge and can be determined by appropriate model tests in a wind tunnel. The results of such experiments can be used for preliminary design. The structural damping of bridges has been investigated extensively. Model tests have also been carried out to determine the effect of structural damping on cable stayed and suspension bridges. According to the measurements on the Norderelbe bridge (193) and other bridges, it seems reasonable to assume the logarithmic decrement to be 0.05 to 0.08 for normal cable stayed girder bridges. For calculating natural frequencies, the Rayleigh- Ritz method has been most widely used. The accuracy of the method depends upon proximity of the assumed modes of oscillations to the actual modes.

Indian Standards do not provide any specific recommendations for wind loads on cable stayed girder bridges. However, the recommendations are given for wind loads on other types of steel bridges and other structures in IS:875-1964. According to this code the whole country is divided into three regions. The wind velocities and wind pressures for different heights of the structures in Gangetic plains are given in the following table (Table No.2.4). In the absence of any specific recommendations, the specifications given for other steel bridges may be considered suitable for cable stayed bridges also.

Table 2.4

Wind Velocities and Wind Pressures for Gangetic Plain

Height in m	Wind velocity in km/hr.	Wind Pressure in kg/m^2	Height in m	Wind Velocity in km/hr.	Wind Pressure in km/m^2
0.0	241.40	39.06	30.5	444.18	141.59
3.0	289.68	58.59	38.1	463.49	151.36
6.1	323.48	73.24	45.7	482.80	166.01
9.1	347.62	87.88	53.3	492.46	175.77
12.2	371.76	97.65	60.9	506.94	185.53
15.2	386.24	107.42	76.2	526.25	200.18
18.30	400.72	112.30	91.4	545.56	209.95
21.3	415.21	122.06	106.7	560.05	224.59
24.4	424.86	126.94	121.9	574.53	234.36

Height of the structure is taken as the average height of the exposed surface above the mean retarding surface (ground or bed level or water level).

2,5 ERECTION

It should be pointed out that the importance of the design and planning of the bridges for the erection stages can not be overemphasized. Reasons for such failure may be as follows:

(i) Temperature and wind are two of the most often neglected factors.

(ii) Lower factor of safety is generally permitted for erection stages, and

(iii) During erection the actual loads are considered while under service conditions the loads considered are conservative (19).

The investigation at each stage of erection is necessary to assure safety. Material characteristics such as relaxation of cables creep and shrinkage of concrete, foundation settlement, welding deformations, etc. are generally difficult to be determined exactly during the design. The deflections at every stage of erection must be measured and compared with that provided by the designer. Any discrepancy should be investigated. In case of Stromsund bridge(193) the discrepancy in calculated and in the field measured deflections was due to the fact that actual modulus of elasticity of material was lower than that assumed in the design.

Three general methods have been developed to support the superstructure while it is being built; (i) free cantilevering operation, (ii) falsework, and (iii) special schemes such as float in, jacking, launching along the longitudinal axis of the superstructure and shifting in the transverse direction. The erection of cable stayed bridges with steel decks is efficiently carried out by push out method. In this method deck elements of appropriate length are pushed out in the longitudinal direction from the erected deck and welded.

The free cantilever erection of a long span cable-stayed bridge can be regarded as one of the most skilful erection feats in the art of bridge building. A free span of about 300 meters can certainly be erected without using any intermediate auxilliary pier or jack. However, it may be necessary to arrange special temporary stays for raising the cantilever end to mount the cables. This method is particularly advantageous in a deep terrain or over rivers with busy navigation, as is the case in the Rhine river.

The loads to be considered during erection are, (i) dead load of the girder, (ii) wind load, (iii) temperature effect, (iv) self weight of the cross section, and also, (v) the erection loads such as the load of equipments.

Aerodynamic stability should also be investigated during erection. Wind velocities depend upon the locality in which the bridge is to be constructed. According to German Standards the wind pressure of 125 kg per m^2 is considered during erection and it can act either in the longitudinal or in the transverse direction. In Austria a wind speed of 42 m/sec (151 km/hr) is used at erection stages which is similar to German practice. The same value is used for the finished bridges. For steel bridges, British Standard BS153, gives a load factor between 1.6 and 1.7 during service and 1.3 during erection. Recently the Draft Marrison Rules (103) have been introduced which provide the loading factor given in Table 2.5, against unserviceability during erection.

Table 2.5

Loading Factors during Erection

Type of Load	Loading Factor
Dead load for steel bridges	1.3
Wind load	1.6
Temperature	1.2
Inaccuracy of support	1.3
Settlement of support	1.3

The German Standard DIN 1073 does not differentiate the loading factors as in U.K. and gives constant loading factor of 1.5 during erection. The amendment made in 1972 has modified the value to 1.71. Leonhardt (219) has proposed that a factor of safety should be at least 1.4 in a finished structure and 1.5 during erection.

In general the construction of the superstructure of a cable-stayed bridge starts from the two main towers, simultaneously on both sides of the towers. The cables provide tempoerary support to the deck and much of the requirement for staging is saved. When the deck approaches the centre of the main span from both sides, the gap is plugged with a central block.

2.6 ECONOMIC CONSIDERATIONS

It has been found that cable-stayed bridges are economical in a span range of 150 m to 360 m. Thul (196,151) has compared the central span length to the total span length for existing continuous girder, cable-stayed and suspension bridges in Germany, as shown in Fig.2.8. This comparison shows that three span continuous girder bridges have an upper limit of about 210 m for the centre span, with a ratio of centre span to total span not more than 50% . Suspension bridges have a lower limit of central span of 300 m with a centre span to total span ratio of approximately 65% . Cable-stayed bridges fill the gap between these two types of bridges, with a centre span to total span ratio of approximately 55% .

Taylor (194) has presented the comparison in respect of weight of structural steel per square m of orthotropic steel plate deck as it varies with the centre span length

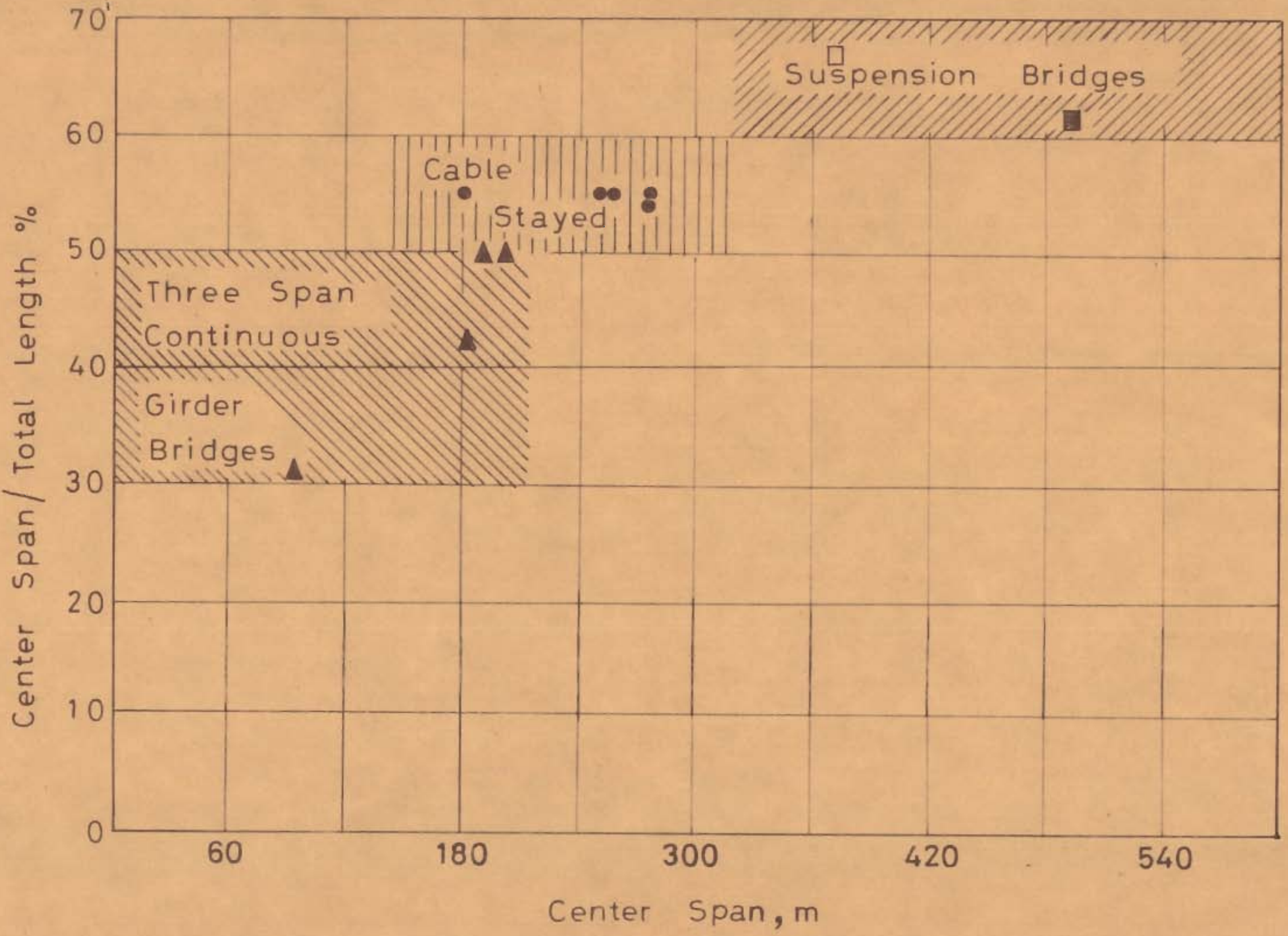


Fig. 2.8 Bridge Types – Their Centre Span and Span Range.

for the above three types of bridges (Fig.2.9). This comparison also indicates that cable stayed bridges fall in between continuous girder and suspension bridges. He has also indicated that for highway bridges in Canada ranging between 210 m and 240 m in span, cable-stayed bridges are 5% to 10% more economical than the other types. It is important to note that in respect of material and labour costs, economic considerations in Europe are quite different to what they are in India.

Thul (221) has stated that it is considered highly unlikely or unrealistic to build cable-stayed bridges with very long spans. Some span lengths will be reserved for suspension bridges because there are considerable difficulties in construction. Leonhardt (118,120,121) however has indicated that cable stayed girder bridges are particularly suitable for spans larger than 600 m and may be employed for spans over 1500 m in future. Leonhardt has supported the suitability of cable-stayed bridges on the basis of stiffness and economic comparison discussed in section 2.6.2. However, it may be pointed out that the largest span constructed so far is 315 m in case of Knie bridge at Dussuldorf and under the circumstances it is difficult to agree with his recommendation for very large spans. The proposed main span of second Hooghly bridge is 464.52 m. The total length of this bridge is 838.20 m.

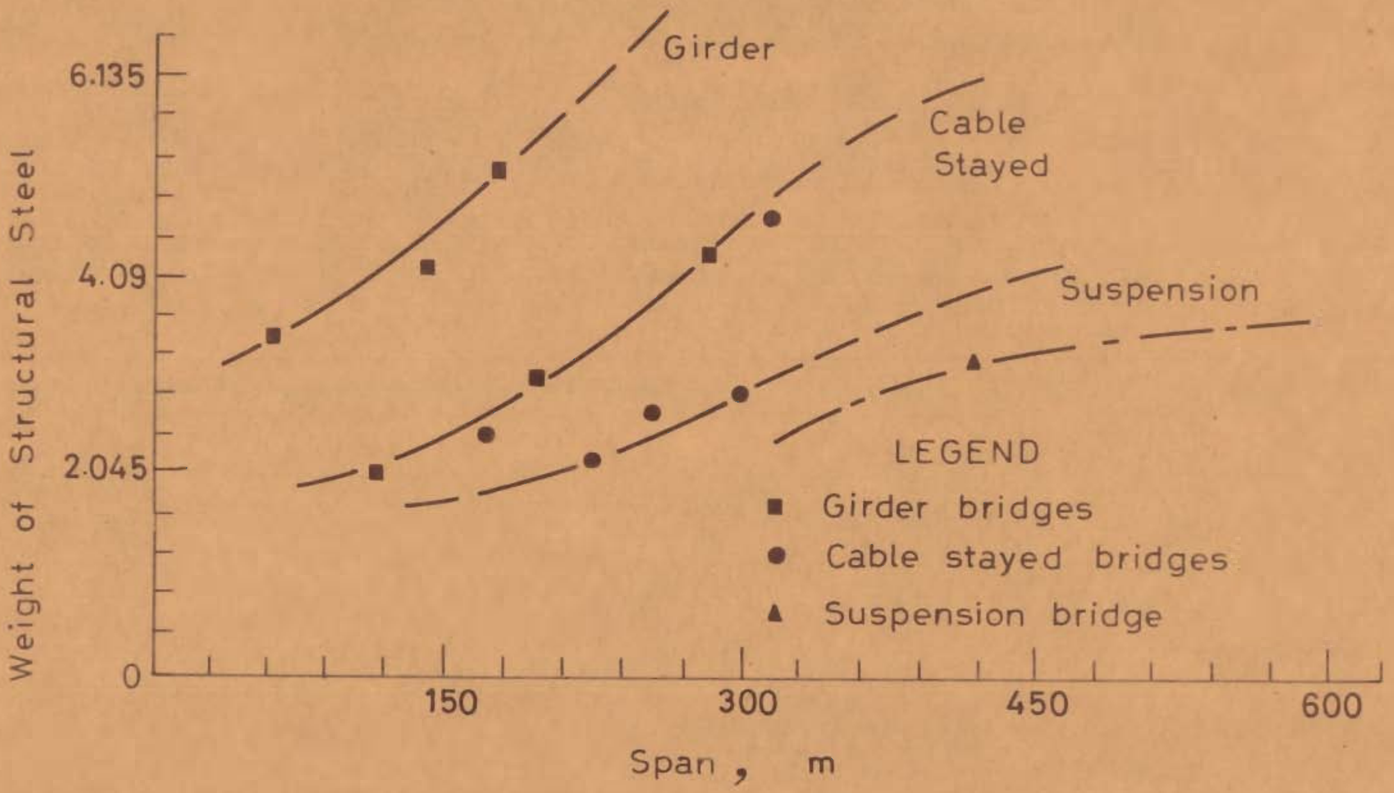


Fig. 2.9 Weight of Structural Steel in Kg/sq. m. of Deck for Orthotropic Steel Bridges.

2.6.1 Cable Steel Comparison

The amount of steel used in cables is directly proportional to the forces developed in the cables. The amount of cable steel is a function of the ratio of pylon height to centre span. This can be represented approximately by the following equation, which involves the assumption that self-weight of cables and any load concentration are neglected

$$W = \frac{q\gamma\ell^2}{\sigma} C \qquad \ldots (2.1)$$

where,

W = weight of steel in cables,

q = total load (D.L. + L.L.),

γ = specific weight of cable steel;

σ = allowable cable stress;

ℓ = length of main span, and

C = dimensionless coefficient

This dimensionless coefficient is determined for classical suspension, cable-stayed harp and cable-stayed fan type bridges as,

$$C_s = \frac{2\ell_1 + \ell}{2\ell}\left[\sqrt{16 + \frac{1}{\mu^2}}\left(\frac{1}{4} + \frac{2}{3}\mu^2\right) + \frac{2}{3}\mu\right] \qquad \ldots (2.2)$$

$$C_H = 1.25\mu \qquad \ldots (2.3)$$

$$C_F = 2.17\,\mu \qquad \ldots (2.4)$$

The minimum quantity of cable steel, W, for both suspension and radiating type cable-stayed bridges is at $\mu = 0.28$ and for harp type cable-stayed bridges at $\mu = 0.5$. When the steel in the pylon and stiffening girder is also included, the most economical value of μ suspension bridges is 0.125 and for cable-stayed bridge it is about 0.2.

Leonhardt (121) has suggested a modification to incorporate the dead load as follows:

$$W = \frac{q\gamma\ell^2}{\sigma} CK \qquad \ldots (2.5)$$

in which K = a modification factor to accommodate cable self weight and for classical suspension, cable-stayed harp and cable-stayed fan type bridges is given by:

$$K_s = 1 + \frac{\gamma\ell^2}{\sigma(2\ell_1+\ell)} C_s + \left[\frac{\gamma\ell^2}{\sigma(2\ell_1+\ell)} C_s\right]^2 + \ldots \qquad (2.6)$$

$$K_H = 1 + \frac{\gamma\ell}{3\sigma} C_H + \frac{\gamma^2\ell^2}{8\sigma^2} C_H^2 + \ldots. \qquad \ldots (2.7)$$

$$K_F = 1 + \frac{\gamma\ell^2 K}{\sigma C_F} + \frac{\gamma\ell}{6\sigma C_F} + \frac{\gamma\ell}{80\sigma K^2 C_F} + \frac{3\gamma^2\ell^2}{160\sigma^2 K} + \ldots.. \qquad \ldots (2.8)$$

A comparative study for Sitka Harbour bridge was carried out considering various types of bridges by Gute(78) and the comparison is given in Table 2.6.

Table 2.6

Cost Comparison for Various Bridges

Type	Description	Cost Ratio
1.	Plate Girder W/Fenders	1.15
2.	Plate Girder	1.13
3.	Orthotropic Box Girder	1.04
4.	Through Tie Arch	1.04
5.	Half Through Tie Arch	1.06
6.	Cable-stayed Girder	1.00

2.6.2 Stiffness Comparison

Leonhardt and Zellner (121) have carried out some studies for the comparison of suspension and cable-stayed spans of similar proportions and assumptions. The common factors considered by them are:

(i) Cable stress limited to 102 ksi (7.17 t/cm^2).

(ii) modulus of elasticity of the cable is 29,200 ksi (2052.76 t/cm^2).

(iii) hinges located at connections of the stay cables and hangers to the stiffening girders.

(iv) normal forces acting on the girders are neglected in the deflection calculations.

(v) termination of the cables is at a height of 1/6 the span above the girder.

(vi) live load is assumed to be 60% of the dead load.

The deflections due to live load covering entire span in case of suspension bridges were found to be 77% of the deflections in cable-stayed bridges. However, for the condition of loading only on one half the span, the deflection of the suspension system was 4.6 times that of the stayed system. The reason for the large variation is the relatively large unsymmetrical deflection of the main cable in the suspension system as it adjusts itself to a new position of equilibrium. In the stayed system the vertical loads are supported by the cables that transfer the loads directly to the tower or saddles without the large deformations taking place.

It may be concluded that to reduce the deflections in suspension systems, larger bending stiffness is required than for the cable-stayed system. This effect justifies, why a suspension bridge must use a high bending stiffness in the stiffening truss which means higher dead weight requirement.

Leonhardt also investigated the effect of live load concentration for both the system and reported that stayed system has less deflections than suspension systems. For a uniform live load of 1.2 times the unit dead load placed over a length of 0.067 of the span, the deflections in suspension system is 5.5 times than that of a stayed system. In general, it may be concluded that cable-stayed system has better stiffness than a suspension system of comparable span.

2.7 MATERIALS

The main components of a cable-stayed bridge superstructure are:

(i) Cables

(ii) Towers

(iii) Deck

The following four properties of cables are of special interest:

(i) Tensile strength,

(ii) Modulus of elasticity;

(iii) Fatigue strength, and

(iv) Creep.

Generally two types of stays are used: (i) Locked coil ropes; and (ii) parallel wire cables. Ropes have been used in most of the cable-stayed bridges built in Germany. The development of parallel wire cables is recent and gaining popularity. The advantages and disadvantages in using cables and ropes are given below;

(i) The variation and determination of modulus of elasticity is not a serious problem in parallel wire cables. In spirally wound ropes, transverse stresses are developed between the individual wire, which reduce the ultimate strength for two reasons; firstly a combined stress system exists at the contact areas, and secondly, plastic flow occurs at these points causing necking of wires. In addition when a rope bends over a saddle, bending stresses as well as transverse radial forces are developed.

(ii) The modulus of elasticity of the cables is constant and higher than that of ropes.

(iii) Ropes exhibit lower fatigue strength as compared to cables. The practical range of stress variation for cables is 2000 to 2500 kg/cm^2 while for ropes 1500 kg/cm^2.

(iv) Cables do not exhibit any plastic strains, hence the staging can be planned in advance. Ropes exhibit plastic strains.

(v) In stayed girder bridges cables should have larger strength weight ratio. High tensile steel locked coil cables exhibit this property and the nonlinearity due to catenary action of the cables is thus reduced. However, for higher strength to weight ratio of the cables, the cable area required to carry the tension will be smaller. The strains for the cables with higher strength to weight ratio will be larger and hence larger deformations. This will increase the geometric nonlinearity. The relation between stress, modulus of elasticity and length of cables as given by Ernst(58) is shown in Fig.2.10.

(vi) Rope heads have relatively large dimensions. It may be difficult to accommodate them in the tower, specially when all cables converge at the tower top.

(vii) Ropes are better protected against corrosion.

2.7.1 Locked Coil Ropes

The stays in this case consist of a number of ropes grouped together (Fig.2.11). The ropes consist of wires of different shapes, arranged in a particular manner. Inner layer consists of round wires, the central layer consists of wedge shaped wires and outer layer of Z-shaped wires. The locked coil ropes offer more area of steel in cross section, for a given rope diameter than the ordinary ropes with spirally arranged round wires.

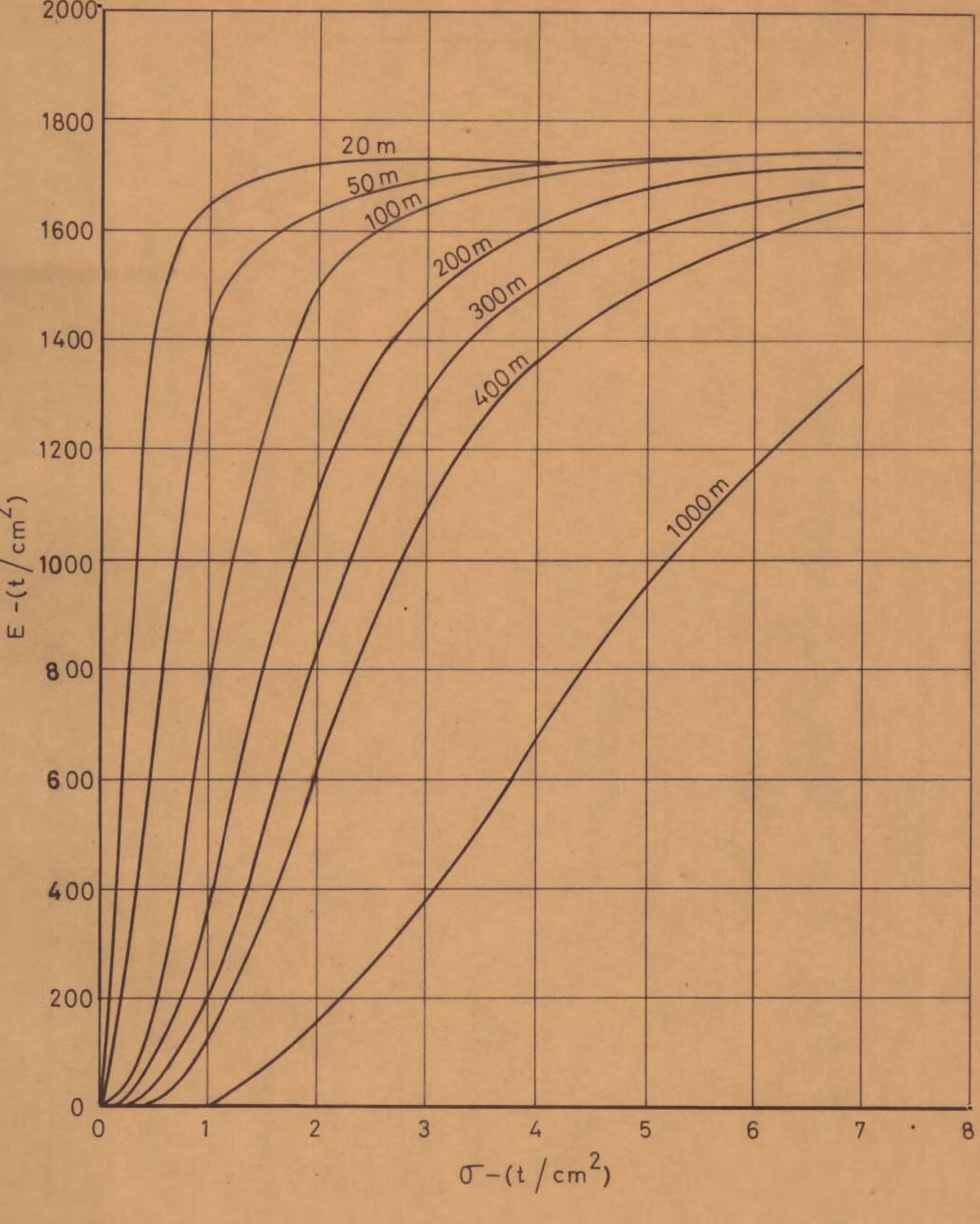


Fig. 2.10 Variation in Modulus of Elasticity of Cables (Ernst)

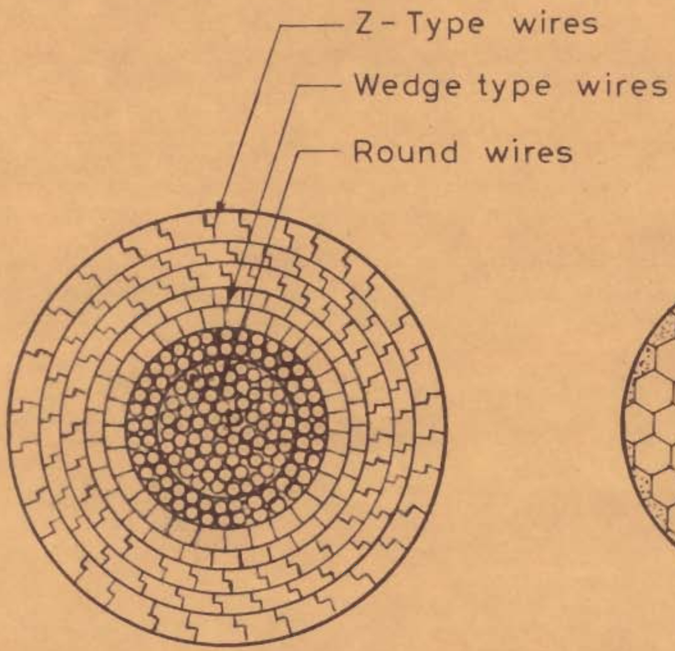


Fig. 2.11 Locked Coil Rope.

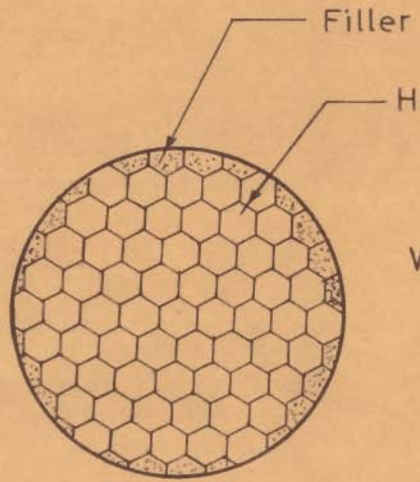


Fig.2.12 High Strength steel Round Cables with Parallel Strands.

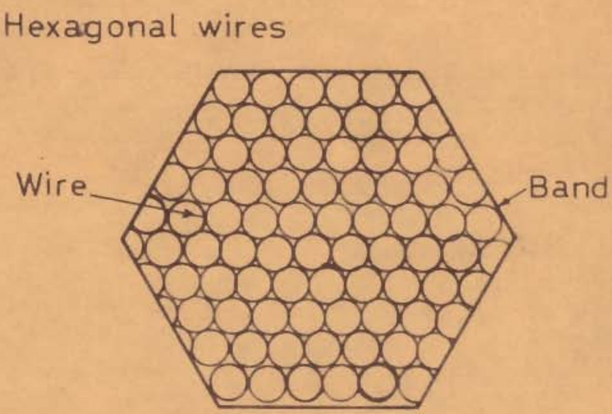


Fig.2.13 Hexagonal High Strength Steel Cables with Parallel Strands.

2.7.2 High Strength Cables

High strength cables developed by Leonhardt consist of cold drawn high tensile steel parallel wires. The cable consists of a number of wire bundles, arranged in a hexagonal form. The voids in these cables are minimum as shown in Fig.2.12. Another type of cables consists of hexagonal wire strands as suggested by Leonhardt(Fig.2.13). The dotted region shows filler material which provides maximum area of cross section.

2.7.3 Towers

The load from the deck is transferred to the towers through cables. These towers are under heavy compressive forces. The towers are in the form of single tower or portal frames and may be constructed from steel or reinforced concrete. The choice of the material depends upon the availability of material and height of the tower.

2.7.4 Decks

Various types of bridge decks have been employed in cable stayed bridges; a few have been of prestressed concrete, but most of these cable-stayed bridges have steel decks. The steel decks used in cable stayed construction may be categorized either as composite steel and concrete or orthotropic plate decks. The decks may be supported on two or more I-shaped girders or on one or more box girders sections.

CHAPTER III

LINEAR ANALYSIS OF CABLE STAYED BRIDGES

3.1 GENERAL

A cable stayed bridge is a statically indeterminate structure with a large degree of redundancy. The girder is like a continuous beam elastically supported at the points of cable attachment and supported on rollers at the towers. The elastic support is provided by the cables. If the nonlinearity due to various factors like large deflections, catenary action of cables and beam column interaction of the girder and tower elements are neglected, the structure can be assumed as linearly elastic. In such a case the linear methods of analysis can be applied and the principle of superposition is admissible.

The modern approach to structural analysis of such structures falls into the broad classification of stiffness (displacement) and flexibility (force) methods. A mixed approach is also sometimes employed. Recent literature shows the utility of the finite element method in accounting for the effect of the deck more accurately.

The literature available on the analysis of cable-stayed bridges has appeared mostly in the last ten years. Usually cable-stayed girder bridges have been dealt as simplified two dimensional structures.

Lazar etal.(110) developed a load balancing method of analysis of cable-stayed bridges, which is suitable for hand computations. A mixed force displacement method has been developed by Smith(182) . while Troitsky and Lazar(199) used a flexibility approach for theoretical analysis, and Tang(192) has illustrated the use of the transfer matrix approach.

Smith, in one of his papers(183) has extended the analysis technique to the analysis of double plane cable-stayed girder bridges and treated the deck as a plate. Kajita and Cheung(96) have illustrated the use of the finite element technique. In this method the bridge deck is divided into a number of rectangular shell elements and the tower is divided into beam elements. The whole structure is treated as a three dimensional system. In order to account for the movement of the saddle, a flexibility approach is used for determining cable forces. The well known 'stiffness' approach seems to be the most commonly employed tool of analysing such structures and is the one used in this thesis. The approach is briefly described for continuity in Sec.3.2.

3.2 FORMULATION FOR STIFFNESS ANALYSIS

Member end forces p_1 and p_2 can be written in terms of end displacements d_1 and d_2 and member stiffness k_{11},k_{12},

k_{21} and k_{22} of the member,

$$\left.\begin{aligned} p_1 &= k_{11}d_1 + k_{12}d_2 \quad \text{at end 1} \\ p_2 &= k_{21}d_1 + k_{22}d_2 \quad \text{at end 2} \end{aligned}\right\} \qquad \ldots (3.1)$$

These expressions are applicable only when the structure is linearly elastic and the principle of superposition is admissible. The general force displacement equation for a structure can be written as

$$p = kd \qquad \ldots (3.2)$$

where, p is the complete set of applied joint loads, d is the corresponding set of unknown joint displacements, and, k indicates the stiffness matrix of the structure. The condition of joint equilibrium is used to formulate the set of simultaneous equations.

Equations (3.1) are singular for a single member. The complete load displacement relation for a stable structure is given by Eq.(3.2). After applying proper boundary conditions this equation will always have a unique solution. The matrix k will always be non-singular and hence it can be inverted to obtain the displacements as follows,

$$d = K^{-1}\,p = F\,p \qquad \ldots (3.3)$$

where F is the flexibility matrix. The stiffness matrix k of the structure is symmetric.

Equation (3.3) is formulated in the member axes system, and has to be transformed to conform to system coordinates. The load vector in system coordinates can be written as

$$p' = T\ p \qquad \text{... (3.4)}$$

where, T is the transformation matrix which is square and orthogonal, with its inverse equal to its transpose. Hence,

$$p = T^t\ p \qquad \text{... (3.5)}$$

Similarly,

$$d = T^t\ d \qquad \text{... (3.6)}$$

Now from Eq.3.2 set of simultaneous equations can be written in system coordinates as,

$$p' = T\ K\ T^t d \qquad \text{... (3.7)}$$

and hence from this equation it follows,

$$K' = T\ K\ T^t \qquad \text{... (3.8)}$$

where K' is the stiffness of structure in system coordinates.

The analysis of cable stayed bridges can be carried out treating it as a two or three dimensional structure. The basic principles involved in both analyses are the same as discussed above. The formulation of matrices for the two analyses is given in the following sections.

3.2.1 Stiffness Matrix

Equations 3.1 to 3.7 are applicable to both two and three dimensional structures. In formulating the stiffness matrix it is assumed that the members are straight and have uniform cross section. Referring to Fig.3.1 the load displacement relationship at the two ends can be written as,

$$\left.\begin{aligned} p_1 &= k_{11}d_1 + k_{12}d_2 \\ p_2 &= k_{21}d_1 + k_{22}d_2 \end{aligned}\right\} \qquad \ldots (3.9)$$

k_{11}, k_{12}, k_{21} and k_{22} are stiffness matrices of the member in general.

For a member in two dimensional structures, both load and displacement vectors will have three components each. Hence the size of the stiffness matrices k_{11},k_{12} etc. will be 3x3. These matrices can be written as follows:

$$k_{11} = \begin{bmatrix} EA/L & 0 & 0 \\ 0 & 12EI/L^3 & 6EI/L^2 \\ 0 & 6EI/L^2 & 4EI/L \end{bmatrix} \qquad \ldots (3.10a)$$

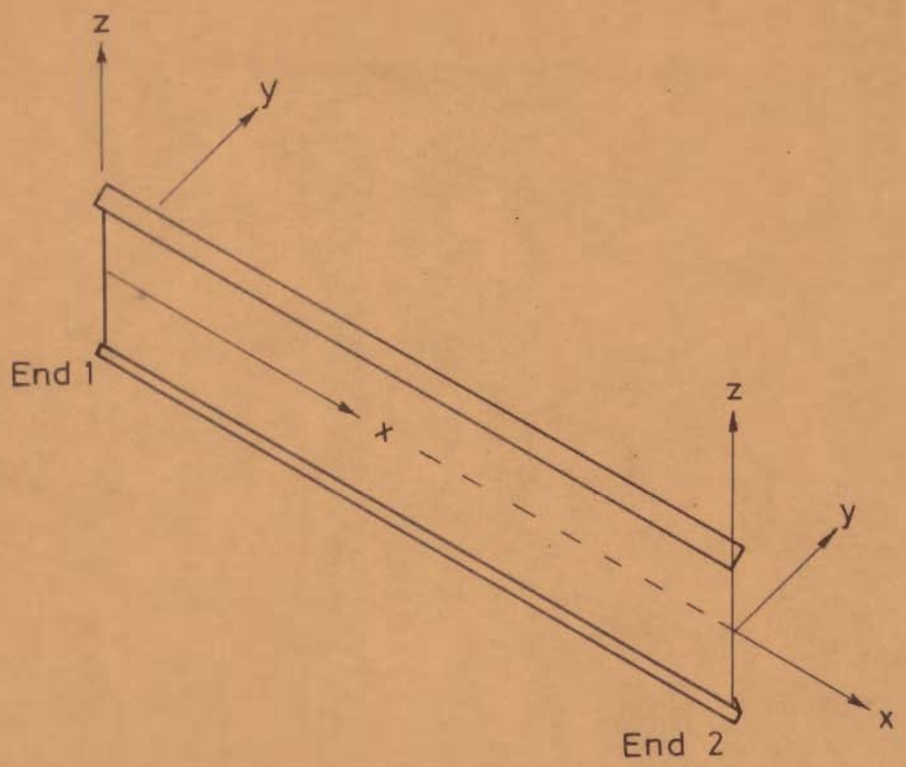


Fig. 3.1 Member Coordinate System for a Member of a Three Dimensional Frame.

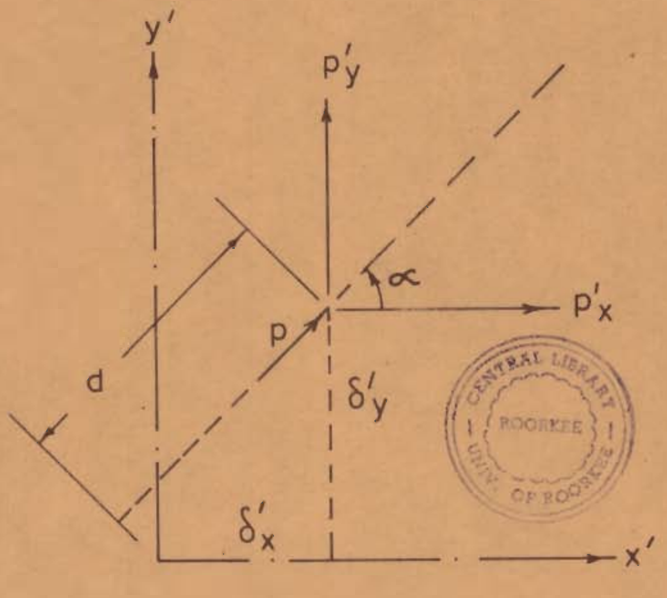


Fig. 3.2 Member and System Coordinate Axes for a Member of a Three Dimensional Frame.

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$$k_{22} = \begin{bmatrix} EA/L & 0 & 0 \\ 0 & +12EI/L^3 & -6EI/L^2 \\ 0 & -6EI/L^2 & 4EI/L \end{bmatrix} \quad \ldots (3.10b)$$

$$k_{12} = \begin{bmatrix} -EA/L & 0 & 0 \\ 0 & -12EI/L^3 & 6EI/L^2 \\ 0 & -6EI/L^2 & 2EI/L \end{bmatrix} \quad \ldots (3.10c)$$

and

$$k_{21} = \begin{bmatrix} -EA/L & 0 & 0 \\ 0 & -12EI/L^3 & -6EI/L^2 \\ 0 & 6EI/L^2 & 2EI/L \end{bmatrix} \quad \ldots (3.10d)$$

In a three-dimensional structure the load and displacement vectors each will have six components. Hence, the size of stiffness matrices for each end will be 6x6. For members in space these matrices can be written as

$$k_{11} = \begin{bmatrix} EA/L & 0 & 0 & 0 & 0 & 0 \\ 0 & 12EI_z/L^3 & 0 & 0 & 0 & 6EI_z/L^2 \\ 0 & 0 & 12EI_y/L^3 & 0 & -6EI_y/L^2 & 0 \\ 0 & 0 & 0 & GJ/L & 0 & 0 \\ 0 & 0 & -6EI_y/L^2 & 0 & 4EI_y/L & 0 \\ 0 & 6EI_z/L^2 & 0 & 0 & 0 & 4EI_z/L \end{bmatrix}$$

... (3.11a)

$$k_{12}=k_{21}^{t} = \begin{bmatrix} -EA/L & 0 & 0 & 0 & 0 & 0 \\ 0 & -12EI_z/L^3 & 0 & 0 & 0 & 6EI_z/L^2 \\ 0 & 0 & -12EI_y/L^3 & 0 & -6EI_y/L^2 & 0 \\ 0 & 0 & 0 & -GJ/L & 0 & 0 \\ 0 & 0 & 6EI_y/L^2 & 0 & 2EI_y/L & 0 \\ 0 & -6EI_z/L^2 & 0 & 0 & 0 & 2EI_z/L \end{bmatrix}$$

... (3.11b)

k_{22} is equal to k_{11} with the signs of the off-diagonal elements $-6EI_y/L^2$ and $6EI_z/L^2$ reversed.

In general the complete load displacement relationship for a member can be written as,

$$\begin{bmatrix} p_1 \\ p_2 \end{bmatrix} = \begin{bmatrix} k_{11} & k_{12} \\ k_{21} & k_{22} \end{bmatrix} \begin{bmatrix} d_1 \\ d_2 \end{bmatrix} \quad \text{... (3.12)}$$

3.2.2 Transformation Matrix

The above load displacement relations for a member are in the member coordinate system and are to be transformed in system coordinates. In general the complete transformation matrix for a member can be written as,

$$T = \begin{bmatrix} T_{11} & 0 \\ 0 & T_{22} \end{bmatrix} \qquad \ldots (3.13)$$

For a member in two dimensional structure the size of this transformation matrix will be 6x6. If the direction of the member makes an angle α with the x-axis in system coordinates as shown in Fig.3.2, then the transformation submatrices will be

$$T_{11} = T_{22} = \begin{bmatrix} \cos\alpha & \sin\alpha & 0 \\ \sin\alpha & \cos\alpha & 0 \\ 0 & 0 & 1 \end{bmatrix} \qquad \ldots (3.14)$$

For a member in three-dimensional structure the size of the complete transformation matrices will be 12x12. The transformation submatrices can be written as,

$$T_{11} = T_{22} = \begin{bmatrix} \widehat{x'x} & \widehat{x'y} & \widehat{x'z} & 0 & 0 & 0 \\ \widehat{y'x} & \widehat{y'y} & \widehat{y'z} & 0 & 0 & 0 \\ \widehat{z'x} & \widehat{z'y} & \widehat{z'z} & 0 & 0 & 0 \\ 0 & 0 & 0 & \widehat{x'x} & \widehat{x'y} & \widehat{x'z} \\ 0 & 0 & 0 & \widehat{y'x} & \widehat{y'y} & \widehat{y'z} \\ 0 & 0 & 0 & \widehat{z'x} & \widehat{z'y} & \widehat{z'z} \end{bmatrix} \qquad \ldots (3.15)$$

where $\widehat{x'x}$, $\widehat{x'y}$, $\widehat{x'z}$ etc. are the cosines of the angles made by the appropriate member axis with the corresponding axis in system coordinates.

These stiffness and transformation matrices are written for each member separately. The stiffness matrices are written in member coordinate system and are transformed in system coordinates for each member separately. Then the complete matrix of the structure is assembled in the form of Eq.3.7. This set of simultaneous equations is solved to obtain the unknown displacements after applying appropriate boundary conditions.

3.3 PROPOSED ANALYSES

In using the standard methods of analysis for the study of cable bridges contained in this thesis, the following assumptions are made:

(i) The model for linear analysis is the equilibrium configuration of the bridge under the effect of the dead loads and prestress. Deformations and forces are computed for the structure under live load. The initial configuration is assumed not to be affected by the deformations.

(ii) All the joints between members in the structure are assumed to be rigid. The joint between the cable and the girder or the tower is in effect pinned joint on account of assumption (iii)

(iii) Cables are assumed to be perfectly flexible. The flexural rigidity of cables is very small as compared to that of girder and tower elements and hence neglected.

(iv) Effect of creep in steel is neglected. Concrete deck and towers are not considered here. Where such construction is used, effect of shrinkage and creep will merit consideration.

(v) Loads are taken to act as they are applied i.e., as concentrated or uniformly distributed. The uniformly distributed loads are taken as equivalent joint loads and the fixed end forces due to these loads are superimposed on the member end forces calculated on the basis of equivalent joint loads.

(vi) The cables are assumed to be capable of taking tensile force as well as compressive force. Compressive forces that occur on account of applied live loads are usually small, if at all they occur. It is implied that the dead load tension and prestress in the cables are much larger than this compressive force and hence cables do not become slack.

(vii) Cables are assumed to be straight members, i.e., the effect of catenary action due to self-weight of cables is neglected. Podolny(150) has shown that the effect of catenary action for moderate sag to span ratio is not large. However, this effect can be incorporated by modifying the modulus of elasticity(150,58).

(viii) In linear analysis the structure is assumed to remain elastic.

(ix) The effect of change in geometry, beam-column interaction of girder and tower elements and the effect of warping are neglected.

The analysis treating the structure as two-dimensional is carried out to study the effect of various parameters on the behaviour of the bridge. This analysis is also applied to analyse the bridge during various stages of erection, the details of which are given later in chapters 5 and 6.

The linear analysis treating the bridge as a three-dimensional structure is carried out to study the space action of the bridge. This analysis includes the determination of the bending moments, shear forces, axial forces and torsional moments in two perpendicular planes. The three displacements and the three rotations at each joint are also calculated. The three-dimensional structure covers the following loading cases:

(i) Main span of one longitudinal girdgi is loaded except the central panel.

(ii) Main spans of both the longitudinal girders are loaded.

(iii) Central panel of one longitudinal girder is loaded.

(iv) The windward longitudinal girder is loaed with the static wind load.

3.4 DISCUSSION OF RESULTS

The details of the bridge analysed here are given in Fig.3.3 along with the sectional properties. The maximum values of the forces and deflections are given in Table 3.1. The results obtained for each loading case are discussed in detail in the following paragraphs.

3.4.1 Loading Case I

In the first loading case the main span of the one longitudinal girder is loaded except the central panel. The resulting forces and deflections for the loaded girder are much larger than those for the unloaded girder. In the radiating structural arrangement, the vertical deflection is observed at the two ends of the central panel in both

Data

Longitudinal Girder

$A = 0.30 m^2$

$I_x = 0.40 m^4$

$I_y = 0.50 m^4$

$I_z = 0.60 m^4$

Tower

$A = 0.30 m^2$

$I_x = 0.40 m^4$

$I_y = 0.20 m^4$

$I_z = 0.20 m^4$

Cable

$A = 0.03 m^2$

$I_x = I_y = I_z = 0.0$

Cross Beam

$A = 0.056 m^2$

$I_x = 0.000012 m^4$

$I_y = 0.0006 m^4$

$I_z = 0.040 m^4$

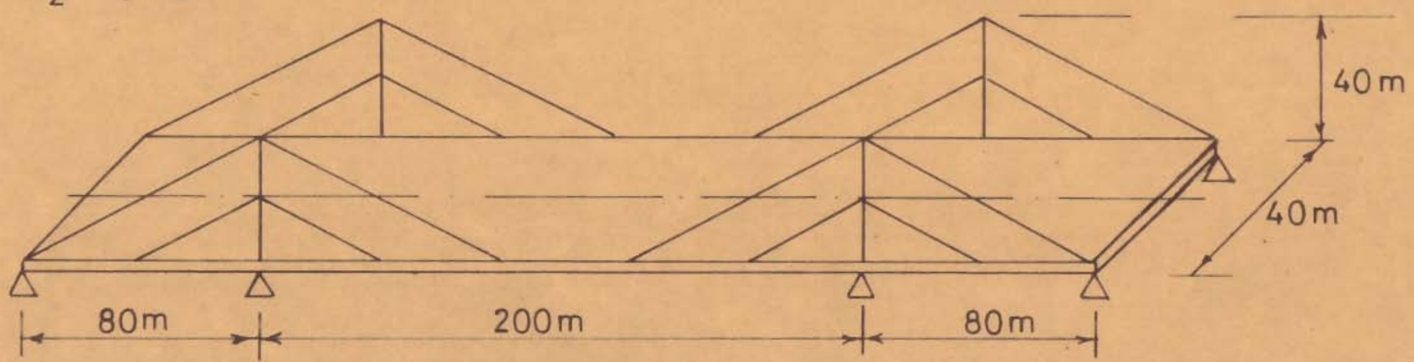

Fig. 3.3(a) Details of the Bridge Analysed as Three Dimensional Structure. (Harp Arrangement)

Data

Longitudinal Girder

$A = 0.30 m^2$

$I_x = 0.40 m^4$

$I_y = 0.50 m^4$

$I_z = 0.60 m^4$

Cable

$A = 0.03 m^2$

$I_x = I_y = I_z = 0.0$

Tower

$A = 0.30 m^2$

$I_x = 0.40 m^4$

$I_y = 0.20 m^4$

$I_z = 0.20 m^4$

Cross Beam

$A = 0.056 m^2$

$I_x = 0.000012 m^4$

$I_y = 0.0006 m^4$

$I_z = 0.040 m^4$

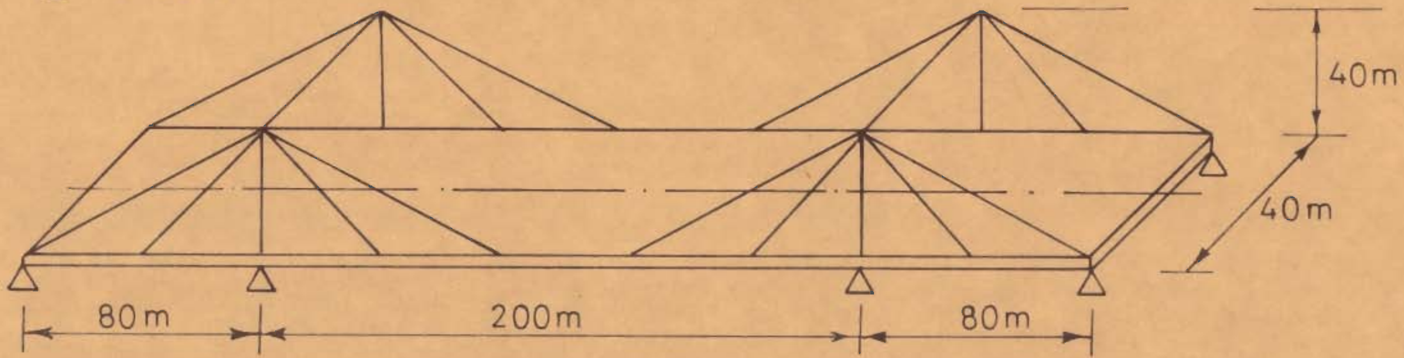

Fig. 3.3(b) Details of the Bridge Analysed as Three Dimensional Structure. (Radiating Arrangement)

Table - 3.1

MAXIMUM VALUES OF FORCES AND DISPLACEMENTS BY THREE DIMENSIONAL ANALYSIS

Deflections or Forces	Loaded or Unloaded girder	Loading case 1		Loading case 2		Loading case 3		Loading case 4*	
		Radiating	Harp	Radiating	Harp	Radiating	Harp	Radiating	Harp
1	2	3	4	5	6	7	8	9	10
Main Girders Maximum deflections in m.	Loaded	0.1821	0.2068	0.3103	0.3420	0.1153	0.1193	0.9577	0.9577
	Unloaded	0.0080	0.0106	0.3103	0.3420	0.0049	0.0053	0.9541	0.954
Cable Tension in tonnes.	Loaded	361.4	326.7	556.9	526.8	172.4	174.9	0.0	0.0
	Unloaded	15.0	17.2	556.9	526.8	8.2	8.1	0.0	0.0
B.M. in vertical plane in t.m.(Mz).	**Loaded**	993.6	1809.1	1611.2	2502.8	542.2	497.3	0.0	0.0
	Unloaded	61.6	152.7	1611.2	2502.8	32.7	43.7	0.0	0.0
B.M. in horizontal plane (My) in t.m.	**Loaded**	0.1	0.1	0.0	0.0	0.0	0.0	8693.7	8693.7
	Unloaded	0.1	0.1	0.0	0.0	0.0	0.0	7933.1	7933.1
Torsional Moment in t.m.	Loaded	162.7	225.3	0.0	0.0	83.2	82.5	0.0.0	0.0
	Unloaded	162.7	225.3	0.0	0.0	83.2	82.5	0.0	0.0
Tower Moments in t.m.	Loaded	0.0	971.0	0.0	1088.7	0.0	46.0	0.0	0.0
	Unloaded	0.0	604.4	0.0	1088.7	0.0	11.3	0.0	0.0
Cross Beams									
B.M. in vertical plane in t.m. (Mz)		105.0	145.1	0.0	0.0	83.2	83.4	0.0	0.0
		0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Torsional Moment in t.m.		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Loading cases 1. Main span of one longitudinal girder except the central panel
2. Main span of both the longitudinal girders.
3. Central panel of the one longitudinal girder.
4. Wind ward longitudinal girder loaded with the static wind load.

Note:- The Uniformaly distributed load is of the intensity 5 t/m

* The deflections are in the horizontal plane perpendicular to the axis of the bridge.

the longitudinal girders. This maximum deflection in the unloaded girder is about 4.3% of that in the loaded girder. The maximum cable tension in the cable in the plane of the unloaded girder is about 4.1% of that in the cables contained in the plane of the loaded girder. The maximum bending moment in the vertical plane of the unloaded longitudinal girder is about 6.1% of that in the loaded longitudinal girder. The maximum bending moment in the horizontal planes of both loaded and unloaded longitudinal girders is negligible as compared to that in the vertical plane of the loaded girder. The torsional moment in both the longitudinal girders is same and is about 16.3% of the bending moment in the vertical plane of the loaded girder. The towers are assumed to be hinged at their bases.

The moments in the cross beams are much smaller than those in the main girders. The maximum bending moment in the vertical plane containing the cross beam is about 10.5% of that in the loaded longitudinal girder. The bending moment in the horizontal plane and the torsional moments in the cross beams are negligible as compared to the bending moment in the vertical plane. The moments are much smaller in the cross beams as their stiffnesses and the torsional rigidities are very small as compared to those of the longitudinal girders.

In the harp arrangement the behaviour of the bridge is very similar to that in the radiating arrangement. The forces and deflections are much smaller in the unloaded girder as compared to those in the loaded girder. The maximum vertical deflection in both the longitudinal girders is observed at the two ends of the central panel. This deflection in the unloaded girder is about 5.2% of that in the loaded girders. The maximum cable tension in the cables lying in the plane of the unloaded girder is about 5.3% of that in the cables lying in the plane of the loaded girder. The bending moment in the horizontal planes of both the loaded and the unloaded girders is negligible as compared to the bending moment in the vertical plane of the loaded girder. The maximum bending moment in the vertical plane of the unloaded girder is about 8.4% of that in the loaded girder. The torsional moments in the loaded and unloaded girders are equal and are about 12.5% of the bending moment in the vertical plane of the loaded girder.

The bending moments at the bases of the towers in the planes of both the main girders are zero as the towers are assumed with pinned bases. However, the maximum moment in the towers in the plane of the unloaded girder is about 62.3% of that in those in the plane of the loaded girder. This maximum tower moment is about 53.7% of the moment in the vertical plane of the loaded girder. The bending moment in the horizontal plane of the cross beams and the torsional moment are negligible. The maximum moment in the vertical plane of the cross beam is about 8.0% of that in the loaded girder.

3.4.2 Loading Case 2

In this loading case the main spans of both the longitudinal girders are loaded. As the two girders are loaded symmetrically the cable tensions, girder deflections and moments are symmetrical about the centre of the bridge as would be expected. The forces and deflections are equal in the two planes of the girders. The bending moments in the horizontal plane of the girders and the torsional moments are absent. The cross beams are not subjected to any bending or torsional moments. The tower bases being pinned, the tower base moments are absent. The above behaviour is observed in both radiating and harp arrangements.

3.4.3 Loading Case 3

In this loading case only the central panel of the main span of one longitudinal girder is loaded. In the radiating arrangement, the maximum bending moment in the vertical plane of the unloaded girder is about 6% of that in the loaded girder. The bending moment in the horizontal plane of both the main girders is equal and is negligible as compared to the bending moment in the vertical plane. The maximum torsional moment in two main girders is equal and is about 15.3% of the maximum bending moment in the girder. The maximum force in the cables of the unloaded girder is 4.7% of that in the cables of the loaded girder. The maximum vertical deflection in both the main girders is observed at the two ends of the central panel. This deflection in the unloaded girder is about 4.2% of that in the loaded girder. The bending moment at the base of the towers is absent. The maximum bending moment in the vertical plane of the cross beam is equal to the torsional moment in the main girder. This bending moment in the cross beam is about 15.3% of that in the loaded girder. The torsional moment and the bending moment in the horizontal plane of the cross beam are negligible as compared to the maximum bending moment in the girder.

The behaviour of the harp arrangement is very similar to that of the radiating arrangement. In this case the maximum bending moment in the vertical plane of the unloaded girder is about 10.0% of that in the loaded girder. The bending moment in the horizontal plane of the loaded and the unloaded girders is negligible as compared to the bending moment in the vertical plane. The torsional moment in both the girders is equal and is about 16.6% of the maximum bending moment in the girder. The maximum girder deflection is observed at the two ends of the central panel in both the loaded and unloaded girders. This deflection in the unloaded girder is about 4.9% of that in the loaded girder. The maximum cable tension in the cables of the unloaded girder is about 4.6% of that in the cables of the loaded girder. The bending moment at the base of the tower is absent. The maximum moment is at the point where the second cable is attached and is about 9.2% of the maximum bending moment in the girder. The maximum bending moment in the tower in the plane of unloaded girder is about 24.3% of that in the tower which is in the plane of the loaded girder. The bending moment in the horizontal plane and the torsional moment in the cross beam are negligible as compared to the bending moment in the vertical plane of the cross beams. The maximum bending moment in the vertical plane of the cross beam is about 17.2% of that in the vertical plane of the loaded girder.

3.4.4 Loading Case 4

In this loading case the effect of static wind load on both radiating and harp arrangement of the bridge has been investigated. The wind loads are assumed to act in a horizontal plane perpendicular to the longitudinal axis of the bridge. Table 3.1 gives the values of maximum forces and the deflections. The change in the cable tensions and the vertical deflections are negligible. The values of horizontal deflection in the direction of the loads are exactly equal in the above two arrangements. The behaviour of both the arrangements is exactly same. This indicates that the presence of the cables does not affect the behaviour of the cable stayed bridge. The reason of this behaviour is that the cables are in a plane normal to the plane of loads. Hence they do not provide any resistance to the deflection in a horizontal plane - not unless the deflections are large, in which case the small deflection analysis carried out in this chapter will become inapplicable.

The results of this investigation indicate that the girder moments and deflections in the vertical plane and the torsional moments are absent. The girder moments in the horizontal plane and the deflections in the horizontal direction are significant. These deflections and moments are larger in the loaded girder as compared to those in the unloaded girder. The deflections are larger by 0.4% in the loaded girder. The bending moment in the horizontal plane of the loaded girder is larger by 9.6% . This difference in the deflection and the girder moments in loaded and the unloaded girders is due to axial deformations of the cross beams.

3.4.5 Comparison of Two Dimensional and Three Dimensional Analyses of the Bridge.

This study has been carried out to investigate the difference in the behaviour of a cable stayed bridge when it is analysed as a two dimensional or three dimensional structure. For this purpose only radiating arrangement of the cable stayed bridge has been investigated. Four loading cases as discussed in Art.3.4.1 to 3.4.4 have been considered.

The details of the structural system analysed and the sectional properties are given in Fig.3.3.

In the three-dimensional analysis the longitudinal girders are subjected to the torsional and bending moments as discussed in the above articles. The torsional moment is absent in the symmetrical loads. In the two dimensional analysis the torsional moments are absent as expected. Table 3.2 shows the maximum values of forces and deflections obtained by the two dimensional and three dimensional analyses of the bridge. This table also gives the percentage difference in the two values. The values of forces and deflections obtained by two dimensional analysis are larger than those obtained by three dimensional analysis. In the first loading case only the main span of one longitudinal girder except the central panel is loaded. In this loading case the maximum vertical deflection obtained by the two dimensional analysis is larger by 4.39% than that obtained by three-dimensional analysis. The maximum cable tension and the girder moment are larger by 4.14% and 6.20% respectively. In the second loading case main spans of both the girders are symmetrically loaded. In this case

Table 3.2

Comparison of Maximum Forces and Deflections obtained by Two Dimensional and Three Dimensional Analyses of Radiating Arrangement.

Loading Case	Type of Analysis	Vertical deflections in meters	Cable tension in tonnes	Girder moment in t-m
	Three-Dimensional	0.1821	361.40	993.60
1	Two-Dimensional	0.1901	376.38	1055.17
	% Difference	4.3932*	4.14	6.20
	Three-Dimensional	0.3103	556.93	1611.20
2	Two-Dimensional	0.3103	556.92	1611.16
	% Difference	0.0	0.0	0.0
	Three-Dimensional	0.1153	172.37	542.20
3	Two-Dimensional	0.1202	180.54	566.81
	% Difference	4.2498	4.74	4.54
	Three-Dimensional	0.9577		8693.70
4*	Two-Dimensional	1.9200		16666.70
	% Difference	100.4800		91.70

* Girder moments and deflections are in the horizontal plane.

the maximum values of the vertical deflection, cable tension and the girder moment are equal in two- and three-dimensional analyses. In the third loading case only the central panel of the main span of the one longitudinal girder is loaded. In this case the forces and deflections obtained by two-dimensional analysis are larger than those obtained by three-dimensional analysis. The maximum vertical deflection is larger by 4.25% while the cable tension and girder moment are larger by 4.74% and 4.54% respectively.

In the fourth loading case the static wind pressure is assumed to act in the horizontal plane in a direction perpendicular to the longitudinal axis of the bridge, and only the three dimensional analysis is used in this case. In the case of two-dimensional analysis the elements of the bridge are assumed to lie in one vertical plane. The cross beams are assumed to be absent. In this study only radiating arrangement is considered.

Table 3.2 gives the maximum values of forces and the horizontal deflections for bridges with one and two longitudinal girders. This table shows that in case of a bridge with one longitudinal girder the girder moment (M_y) and the horizontal displacements are approximately twice of those in a bridge with two longitudinal girders. The maximum horizontal deflection is approximately larger by 100.5% in case of a bridge with one longitudinal girder. The girder moment in the horizontal plane is approximately larger by 91.7%. . The similar behaviour will be observed in bridges with harp arrangement.

CHAPTER-IV

NONLINEAR ANALYSIS

4.1 GENERAL

In classical linear elastic analysis, it is assumed that the deflections of the structure are much smaller as compared to its overall dimensions. Equilibrium conditions are strictly applicable to an undeformed structure and compatibility conditions ignore second order effects of deflections. The extent of influence of deflections on the behaviour of a structure depends directly on the initial geometry of the structure, the sizes and material properties of its elements and the disposition of loads.

The change of geometry influences the equations of equilibrium of the structure and the structure behaves nonlinearly. In addition to the geometric nonlinearity, other types of possible nonlinearities are: (i) Beam-column behaviour of compression elements of the structure, and (ii) Nonlinear behaviour of the material. The beam-column behaviour of the elements of a structure is due to the presence of flexural stresses in addition to the axial stresses in a member which would give rise to nonlinear load deflection characteristics of the structure.

4.2 HISTORICAL REVIEW

Many research workers have contributed to the area of structural analysis for nonlinear structures in the last few decades. Martin (128) and Oden (145) have given a review of the development of nonlinear analysis of structures. Stickling, Haisler, MacDoughall, and Stebbins (186), Kawai andYoshimura (98), and Ball (21) solved the nonlinear algebraic equilibrium equations using an iterative procedure combined with under-relaxation. This approach involves only single inversion of the stiffness matrix and hence saves on computer time. The nonlinear analysis of shells of revolution (186) has shown that the method may effectively be used for problems where the nonlinear deflection is $1\frac{1}{2}$ to 2 times that given by linear analysis. However, as pointed out by Brebbia and Connor(29), among others, the method should not be used to solve highly nonlinear problems. Walker and Hall(212) have used a Newton-Raphson approach and an incremental approach to solve the nonlinear algebraic equations.

Murray and Wilson (138) solved the nonlinear problem by determining the unbalance in nodal forces based on geometric arguments and then used an iterative approach to reduce the unbalance to zero or a specified permissible value. This method is essentially a modified Newton-Raphson approach. Since the element deformations, in particular the rotations, are referred to the deformed geometry, this method is valid for moderate rotation and rigid body rotations.

Marcal (127) has emphasized the use of the initial stress and displacement matrices in an incremental procedure, which is the same as proposed by Turner et-al.(203), but the formulation is in terms of the undeformed coordinates of the structure. Hofmister, Greenbaum and Evesen (89) have extended the approach to include large strains and have improved the solution techniques by correcting for the unbalance in nodal force at each increment. Many others have formulated the geometric nonlinear problem as an initial value problem. Brotton ignored the instability effects caused by the axial loading, while Saafan and Brotton (167) assumed that the differences between arc lengths and chord lengths of the deformed members was too small to influence the equilibrium of the structure.

Saafan (168) has suggested an approach for the nonlinear analysis of frames, considering the effect of instability caused by axial loading, bowing of deformed members and finite deflection. He made the assumptions that the modulus of elasticity was constant; the members were prismatic and initially straight shearing deformations were neglected, buckling out of the plane of the frame was restrained and that the stresses remained elastic.

The nonlinearity in the behaviour of cable stayed girder bridges may arise due to (i) change in geometry;

(ii) bending and axial force interaction (iii) sag in cables, and (iv) material properties. The nonlinearity due to change in geometry may be caused by (i) stretching of cables; (ii) shortening of girders; (iii) shortening of towers; and (iv) rotation of towers whereas nonlinear analysis of many structural forms has been presented in existing literature, there is limited amount of work on the non-linear analysis of cable stayed girder bridges. The major difference that would arise here from the other structures would be the effect of stretching (straightening) of the cables. In the linear analysis of cable-stayed bridges the cable configuration is assumed straight as a chord of the curved shape. In reality the cable takes the shape of a catenary under its own weight and tension, the ordinates of which undergo modification due to changes in the cable tension resulting in nonlinear change in length, hence deflection of the connected point.

Tang (192) and Podolny (150) have considered the nonlinearity in cable-stayed bridges, due to the change of sag-span ratio of cables. Tang(193) has pointed out that bending moment in a beam-column increases due to axial compressive stresses and the relationship is nonlinear, It has been shown by Tamms (191) that in case of Knie cable-stayed bridge the increase of stress was more than 10%. in several critical sections, both in girder and in towers. However, it should be pointed out that the girders of Knie bridge are extremely slender and flexible. Usually the nonlinearity is not so pronounced except in case of towers. Gupta (220) has pointed out the nonlinear effect due to large deflections is negligible with smaller spans and has suggested an iterative approach in which the geometry is modified and applied loads remain the same in each cycle. Feige (62) has reported that the effect of nonlinearity due to large deflections on girder moments is of the order of 6 to 12% . He further indicates that the true moments are less than those obtained by linear analysis. In the case of North bridge, the nonlinearity was found tobe of the order of 12.4% as indicated by Wintergerst. Lazar (108) has also suggested an iterative approach for nonlinear analysis of cable stayed bridges. He has considered the nonlinearity due to (i) large displacements, (ii) bending-axial force interaction and (iii) catenary action of cables. He has analysed an example of harp type arrangement with twelve cables in one plane. The linear values of cable tension and girder moments were smaller than nonlinear values. The maximum effect of nonlinearity in cable forces was 8.3% and that in girder moments 8% . The results apply to a specific case and generalisation will need further studies.

4.3 PROPOSED ANALYSIS

As mentioned already the literature available todate reveals that very little work has been done in the direction of nonlinear analysis of cable stayed bridges. In general the work is limited to study of two dimensional cases with joint loads only. Here an attempt is made to investigate the nonlinear behaviour due to large deflections only, but to include both the two- as well as three-dimensional actions of the bridge and to prepare a computer program to take into account the loads applied at joints as well as the distributed loads of different intensities in different parts of the member.

An iterative procedure based on the Newton Raphson method has been used for the solution of the problem. This approach makes use of the stiffness matrix method, details of which are discussed in detail with respect to linear elastic analysis in chapter 3. It is assumed that the structure is linearly elastic within each cycle of analysis; the geometry as well as the stiffness matrix is modified at the end of each cycle, and member and forces are determined on the basis of modified geometry. Steps in the method are explained below:

(i) Analyse the structure with original geometry considering the total applied load.

(ii) Calculate the deflections and modify the geometry accordingly by adding the deflections at the nodes to their coordinates.

(iii) On the basis of the modified geometry in step (ii), recalculate the stiffness matrix of the structure and solve the same for the imposed deflections of step (ii) so as to obtain the member-end forces.

(iv) Calculate the unbalance between the external loads and the end forces of the members meeting at any particular joint.

(v) Apply the unbalance loads on the structure as at step (ii) above.

(vi) Repeat the steps (i) to (v) till the unbalance force becomes negligible, or less than a specified value. In the present case, the unbalanced forces were reduced to less than 0.001 in their absolute value.

The rate of convergence of the solution depends upon the magnitude of nonlinearity. For most cases analysed, satisfactory convergence was obtained within three cycles of iterations. The structural arrangement and types of loads considered for two and three dimensional analyses are discussed in the following sections.

4.3.1 Nonlinear Analysis Treating the Bridge as a Plane System.

The following structural arrangements are treated in this section.

(i) Radiating arrangement,

(ii) Harp arrangement, and

(iii) Star arrangement.

Details of structural systems considered here are shown in Fig.4.1.

Two loading cases are considered:

(i) Uniformly distributed load over main span

(ii) Uniformly distributed over one side span.

The nonlinear effects of girder flexibility and support conditions are tower base are also studied. The results obtained for various cases are discussed below:

(a) Loading Case 1

In this case the main span of the bridge is loaded with a uniformly distributed load of 5 t/m. Table 4.1 gives a comparison of the linear and nonlinear values of maximum deflection and cable tension. In radiating arrangement the maximum nonlinearity effects in girder deflection and cable tension are about 1.2%. and 0.6%. respectively. Corresponding values for the harp arrangement are 1.5%. and 1.3%. and in the star arrangement 1.6%. and 1.1%. . Nonlinear analysis of the proposed second Hooghly bridge has also been carried out. In this bridge nonlinearity

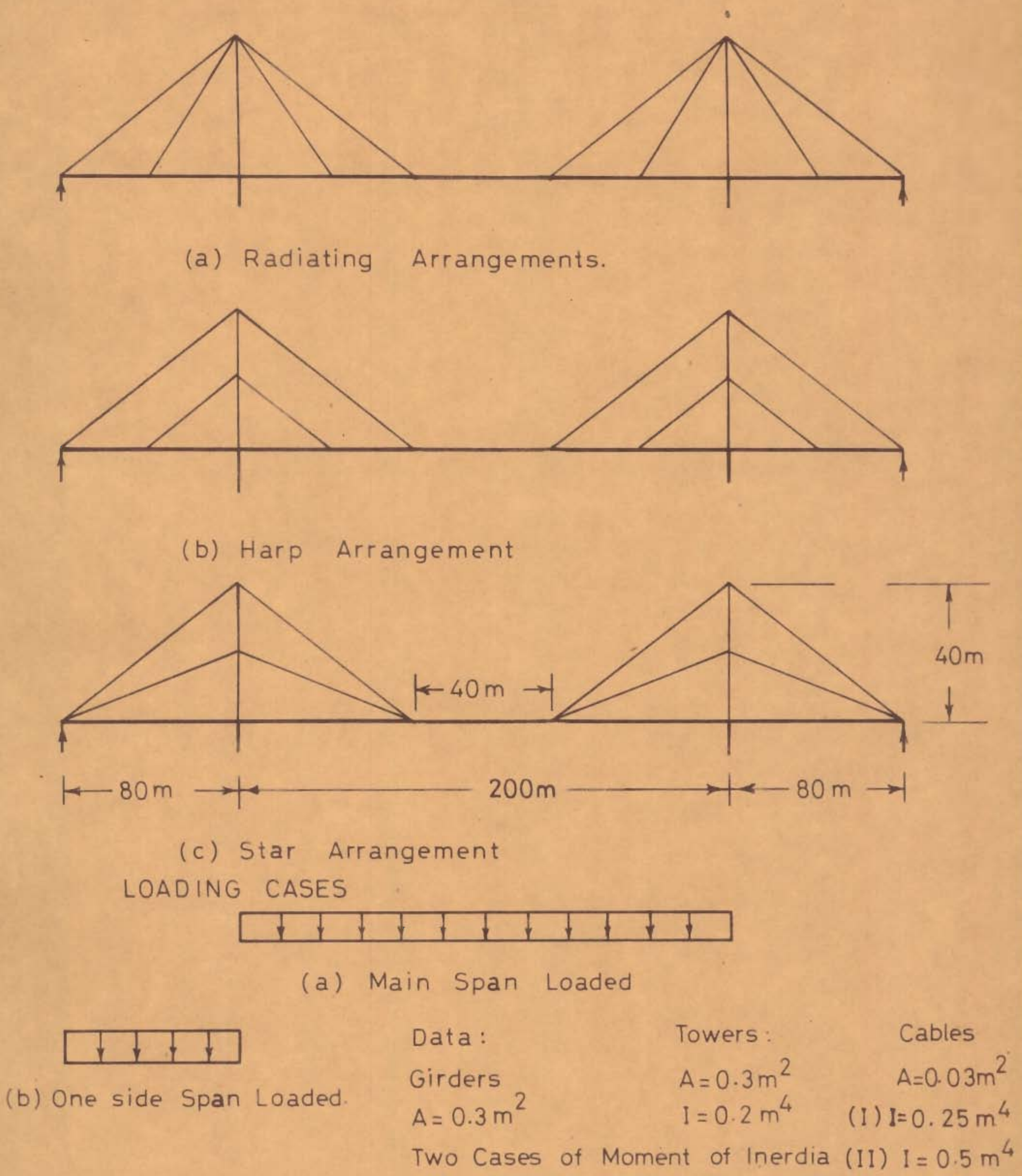

Fig. 4.1 Structural Systems for Two Dimensional Non-linear Analysis of the Bridge.

Table - 4.1

Comparison of Forces and Deflections Obtained by Linear and Nonlinear Analyses Treating the Bridge two Dimensional Structure. Main Span Loaded with Uniformly Distributed Load of 5 t/m.

Cable Arrangement	Tower base condition	Girder Moment of inertia in m^4	Max^m Deflection in m			Max^m Cable tension in tonnes		
			Linear	Nonlinear	% Nonlinearity	Linear	Nonlinear	% Nonlinearity
1	2	3	4	5	6	7	8	9
Radiating Arrangement	Tower base hinged	0.50	0.3191	0.3158	+1.045	573.00	570.10	+0.509
		0.25	0.3471	0.3428	+1.254	622.00	618.30	+0.598
	Tower base fixed	0.50	0.3137	0.3105	+1.031			
		0.25	0.3406	0.3365	+1.218			
Harp Arrangement	Tower base hinged	0.50	0.3500	0.3452	+1.390	542.10	536.70	+1.006
		0.25	0.3728	0.3672	+1.525	592.20	584.60	+1.300
	Tower base fixed	0.50	0.3245	0.3203	+1.311			
		0.25	0.3406	0.3357	+1.460			
Star Arrangement	Tower base hinged	0.50	0.3281	0.3233	+1.485	476.40	471.80	+0.975
		0.25	0.3437	0.3382	+1.626	499.10	493.70	+1.094
	Tower base fixed	0.50	0.3211	0.3164	+1.485	468.60	464.40	+0.904
		0.25	0.3361	0.3307	+1.633	490.50	485.50	+1.030
Second Hooghly Bridge			0.6436	0.6439	-0.047	722.60	723.38	-0.108

effect in the maximum deflection is about 0.05% and in cable tension about 0.11% . Nonlinear values for both forces and deflections are **Smaller** than the linear values.

(b) Loading Case 2

In this case only one side span is loaded with a uniformly distributed load of 5 t/m. This is a case of unsymmetrical load. Table 4.2 gives the linear and non-linear values of maximum deflection and cable tension. For the radiating arrangement, maximum nonlinearity in the girder deflection is about 0.4 and for cable tension, it is about 0.1%. Corresponding values for the harp arrangement are 2.2% and 0.5% and in the star arrangement 0.2% and 0.1% . In the proposed second Hooghly bridge (prototype) the nonlinearity in the vertical deflection and the cable tension was too small to be noticed since the linear and nonlinear values of forces and girder deflections were very close.

The nonlinearity in the structure increases with increase in girder flexibility. The nonlinearity is further increased for the base condition for the tower is changed from fixed to hinged. The maximum nonlinearity in cable tension is of the order of 1.3% and that in girder deflections is of the order of 1.6% . The nonlinear values of cable tension and girder deflection are smaller than linear values.

Table - 4.2

Comparison of Forces and Deflections Obtained by Linear and Nonlinear Analyses Treating the Bridge as two Dimensional Structure, One Side Span is Loaded with Uniformly Distributed Load of 5 t/m.

Cable Arrangement	Tower base Condition	Moment of Inertia in m^4	Max^m Deflection in Meters			Max^m Cable Tension in Tonnes		
			Linear	Nonlinear	% Nonlinearity	Linear	Nonlinear	% Nonlinearity
1	2	3	4	5	6	7	8	9
Radiating Arrangement	Tower base hinged	0.50	0.0601	0.0600	0.17		229.10	
		0.25	0.0714	0.0711	0.42	272.40	272.00	+0.15
	Tower base fixed	0.50						
		0.25	0.0705	0.0702	0.43		272.60	
Harp Arrangement	Tower base hinged	0.50				187.30	188.20	-0.48
		0.25	0.2044	0.2000	2.20	274.90	274.90	0.00
	Tower base fixed	0.50						
		0.25	0.1541	0.1515	1.72		327.80	
Star Arrangement	Tower base hinged	0.50	0.0371			51.70	51.65	+0.10
		0.25	0.0472	0.0473	-0.21	64.66	64.74	-0.12
	Tower base fixed	0.50	0.0366			50.82		
		0.25	0.0465	0.0466	-0.22	63.97	64.06	-0.14
Second Hooghly Bridge			0.0726	0.0726	0.000	137.40	137.40	0.00

4.3.2 Three-Dimensional Nonlinear Analysis

Nonlinear analysis treating the bridge as a three dimensional structure has been carried out for a limited number of cases since it requires solution of larger number of equations and is expensive. Only radiating and harp arrangements have been considered. The sectional properties and details of the radiating arrangement are given in Fig.3.3 in Chapter 3. For each structural arrangement following two loading cases have been investigated.

(i) Main span of one longitudinal girder is loaded except the central panel,

(ii) Central panel of one longitudinal girder is loaded.

The linear and the nonlinear values of maximum forces and deflections for both the arrangements are given in Table 4.3. This table also gives the degree of the nonlinearity in cable tension, girder moments and deflections. A discussion for each loading case follows:

(a) Loading Case 1

In this case the main span of the one longitudinal girder, except the central panel is loaded. The load is unsymmetrically placed on the bridge. In both radiating and harp arrangements the forces and deflections obtained from nonlinear analysis are smaller as compared to linear

Table - 4.3

Maximum forces and deflections obtained by three dimensional nonlinear and linear analyses.

Loading case No.	Type of Analysis	Radiating Arrangement: Vertical Deflection meters	Radiating Arrangement: Cable Tension Tonnes	Radiating Arrangement: Longitudinal girder B.M. t.m.	Radiating Arrangement: Longitudinal girder Torsional moment t.m.	Radiating Arrangement: B.M. in cross beam t.m.	Harp Arrangement: Vertical Deflection meters	Harp Arrangement: Cable Tension tonnes	Harp Arrangement: Longitudinal girder B.M. t.m.	Harp Arrangement: Longitudinal girder Torsional moment t.m.	Harp Arrangement: B.M. in cross beam t.m.
1	Linear	0.1821	361.40	993.60	162.70	105.00	0.2068	326.70	1809.10	225.30	146.10
	Non-linear	0.1811	360.60	993.40	162.00	104.94	0.2052	324.71	1808.69	223.86	144.21
	% Non-linearity	0.55	0.22	0.02	0.43	0.06	0.77	0.61	0.02	0.64	0.80
2	Linear	0.1153	172.37	542.20	83.20	83.19	0.1193	174.90	497.30	82.50	83.40
	Non-linear	0.1148	171.92	541.18	82.87	82.87	0.1187	174.42	495.85	83.01	83.01
	% Non-linearity	0.4337	0.26	0.19	0.40	0.38	0.50	0.27	0.29	0.62	0.47

values. However, the difference is very small. The nonlinearity in the vertical deflection is about 0.55% while in the maximum cable tension it is about 0.22% . Nonlinearity in the maximum bending moment and the torsional moment in the longitudinal girder is about 0.02% and 0.43% respectively while in bending moment in the cross beam it is about 0.06% .

In the harp arrangement the nonlinearity due to large deflection in the maximum cable tension is about 0.61% while in the girder deflection about 0.77% . The nonlinearity in the bending and torsional moment in the longitudinal girder is about 0.02% and 0.64% respectively while in bending moment in the cross beam it is about 0.8% .

(b) Loading Case 2

In this case only the central panel of one longitudinal girder is loaded. This is a case of the unsymmetrical load. Table 4.3 shows that the forces and deflections obtained by the nonlinear analysis are smaller than those obtained by linear analysis in both harp and radiating arrangements. The nonlinearity due to large deflection in the maximum cable tension is about 0.26% and 0.27% for the radiating and the harp arrangements respectively. Corresponding values for the maximum vertical deflection are about 0.43% and 0.50% . In the radiating arrangement the nonlinearity in the bending and the torsional moments in the girder is about 0.19% and 0.4% respectively, while in the harp arrangement it is about 0.29% and 0.62% . The non-linearity in the bending moment in the cross beam is about 0.38% and 0.47% for the radiating and harp arrangements respectively.

CHAPTER V

ANALYSIS OF CABLE STAYED GIRDER BRIDGES DURING ERECTION

5.1 GENERAL

The usual methods of construction of bridges require a huge amount of shuttering and false work. This creates obstructions in a navigation, and specially in the rivers with busy navigation the false work can not be allowed for a long period. This problem can be avoided by using the double cantilever method of construction. As discussed in Art. 2.4, in this method the construction of the deck starts from each tower simultaneously. In steel bridges one panel length of deck is pushed out, and welded to the previous panel. A cable is tied to the free end of the panel. A similar construction technique is used for concrete bridges where one panel is constructed and tied with the cable.

While using the double cantilever method of construction large moments are expected in the girder to occur. These moments may be much larger as compared to the live load girder moments. The literature available todate does not present a detailed analysis of these bridges during erection. It also does not suggest any method to determine the prestressing sequence and the amount of prestress in the cables to minimise moments in the girder. Hence, there is a need to develop a suitable erection procedure in which the bending moments in the girder are small and the navigation in the river is also not obstructed. The research work presented in this chapter is a step towards providing an efficient erection procedure. The investigations presented in this chapter inlude mainly, the effect of the panel lengths on the behaviour of the bridge and reduction of erection moments by prestressing the cables. In the latter study the cables are prestressed to a predetermined value of tension. This reduces the bending moments in the girder to a minimum. The details of the above two studies are discussed in Arts. 5.3 and 5.4 respectively.

5.2 METHOD OF ANALYSIS

The stiffness matrix analysis approach discussed in chapter 3 has been applied to study this problem as well and all assumptions discussed therein are applicable here. In view of the longitudinal symmetry being usually maintained during erection, the bridge is treated as a two dimensional structure. Since the stresses are maintained in elastic range and deflections are kept small, superimposition of results obtained in different stages of erection is considered valid.

The procedure of analysis for determining erection stresses is directly dependent upon the sequence and technique of construction adopted. As mentioned already the 'double cantilever' method of construction is adopted for this study. The two towers are first constructed completely, then the construction of the deck is started from both the towers simultaneously on either side in the longitudinal direction. The different stages of erection for a cable stayed bridge with 12 cables along with the corresponding analytical models are shown in Fig.5.1 for demonstrating the idea. In the first stage one panel length on each side of the tower is constructed. Then one cable on each side of tower is attached to the end of the girder panel, and anchored after prestressing if specified. In the second stage of erection another panel length on each side of the tower is added. Then the next pair of cables is placed as before. This is continued till the last pair of cables is placed. In the last stage the gap between the cantilevers in the central portion is closed with a central panel and in the end spans the last cantilevers are connected to the abutments. The analytical models corresponding to the different stages are analysed in the same sequence as the order of erection and cumulative value of forces and deflections are obtained at the end of each cycle of analysis.

5.3 ERECTION STRESSES WITH NON-PRESTRESSED CABLES

The erection stage analysis is carried out here for the cable stayed girder bridges with thirty six cables in one plane. As stated above two cases (See Fig.5.2) are considered, (a) the panel lengths are kept equal and (b) the panel lengths are unequal. Other sectional properties of the various elements in both these cases are kept the same and are also shown in fig.5.2. The values are worked out for a dead load of 5 t/m. The detailed discussion is given below.

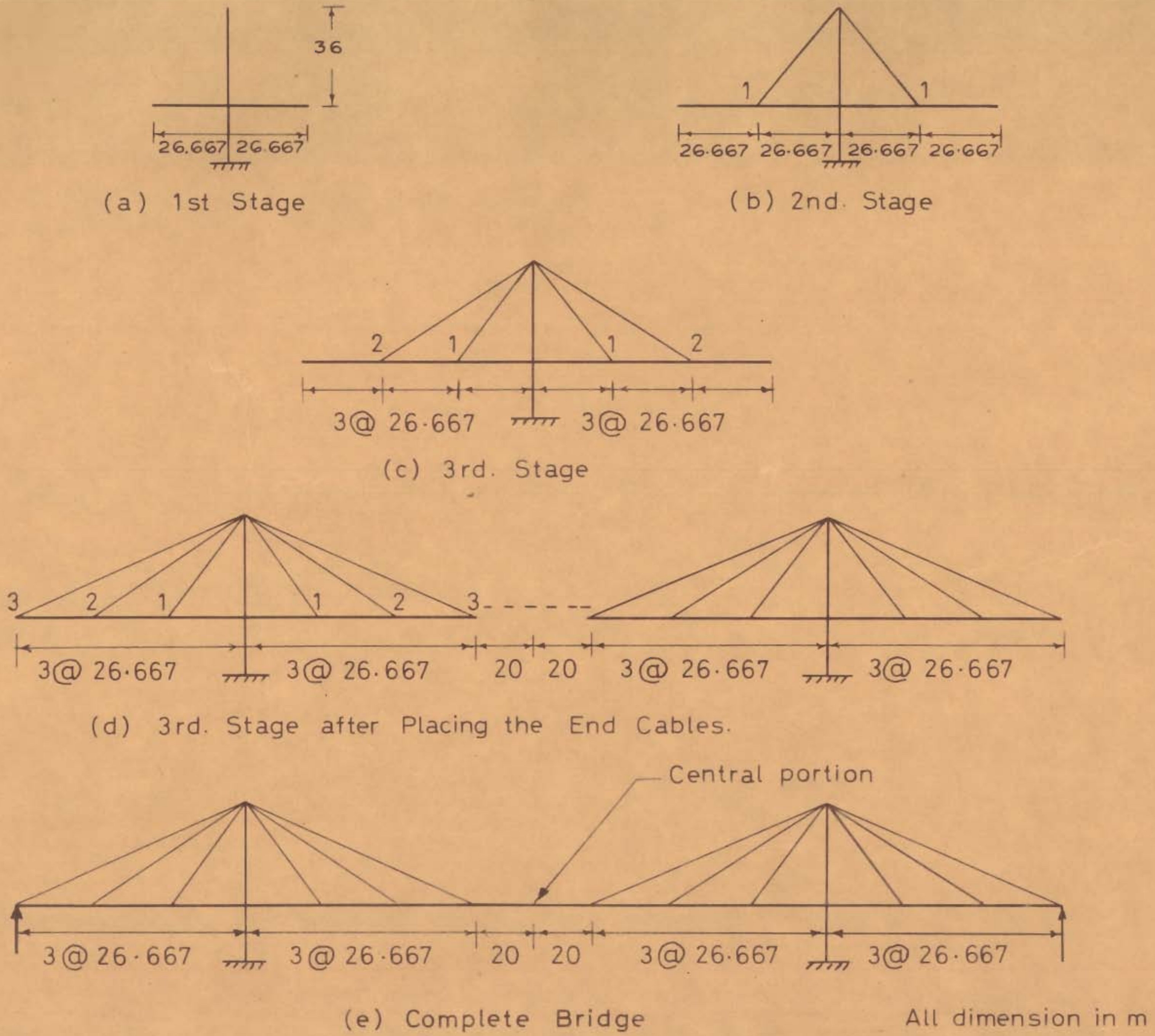

Fig.5.1 Various Erection Stages for a Cable Stayed Bridge with n = 12.

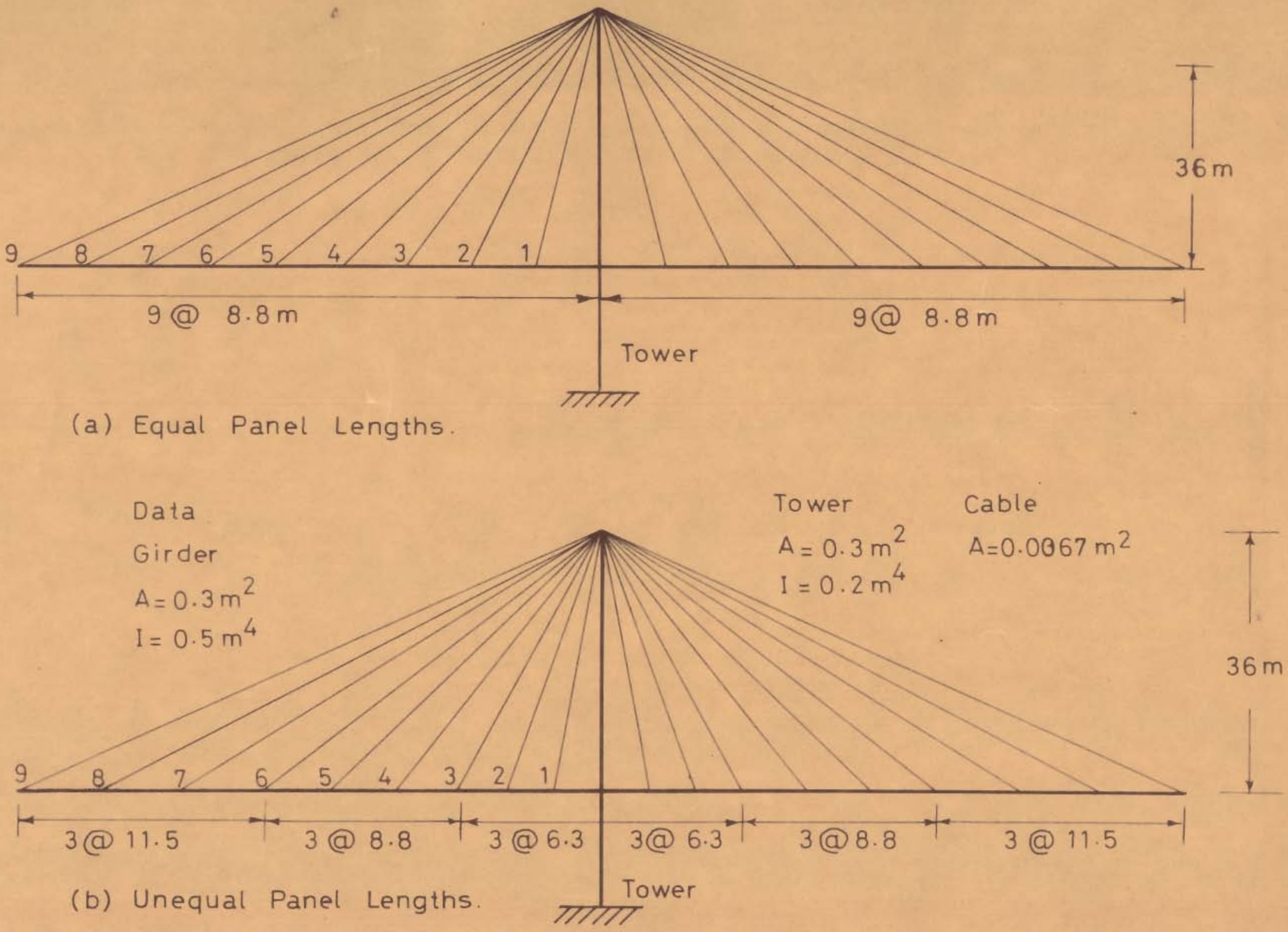

Fig. 5.2. Elevation of Bridges Considered for Erection Analysis.

Girder Moments

Tables 5.1 and 5.2 give the girder moments at different stages of erection for the bridges with the equal and unequal panel lengths respectively. These tables give the absolute values of the girder moments as well as their ratios to the free cantilever moments, at those points. The free cantilever moment at any point is $wx^2/2$ where x is the distance of the free end from that point and w is the intensity of the uniformly distributed load. Figure 5.3 shows the variation of the maximum moments through the different stages and indicates that girder moments are large during erection, the value increasing as the erection proceeds. It is observed that in the initial stages the girder moments in the case with equal panel lengths are larger as compared to those in the bridge with unequal panel lengths. In the last stages the position reverses. The distribution of girder moments during each stage of erection with equal panel lengths is shown in fig.5.4.

Tension in Cables

The values of forces in the cables at each stage of erection for bridges with equal and unequal panel lengths are given in Tables 5.3 and 5.4 respectively. These tables give the absolute values of cable tensions as well as their ratios to a weight parameter F at the corresponding cable point (see inset of fig.5.5 for the definition of F). The variation of the maximum tension in the cables in different stages is shown in fig.5.5. This figure indicates that in the

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Table - 5.1

Girder Moments in t-m during Erection of a Bridge with equal panel lengths (n = 36)

Cable point No.	First stage	Second stage	Third stage	Fourth stage	Fifth stage	Sixth stage	Seventh stage	Eighth stage	Nin-th stage
1	2	3	4	5	6	7	8	9	10
Tower	197.5 (1.00)	741.5 (0.939)	1338.0 (0.753)	1749.0 (0.553)	1973.5 (0.400)	2067.0 (0.291)	2077.5 (0.215)	2040.0 (0.161)	1979.5 (0.124)
1	0.0 (0.000)	197.5 (1.000)	621.0 (0.786)	1024.5 (0.576)	1329.0 (10.421)	1536.0 (0.311)	1663.5 (0.234)	1731.0 (0.179)	1756.0 (0.139)
2		0.0 (0.000)	197.5 (1.000)	565.5 (0.716)	945.5 (0.532)	1277.0 (0.404)	1542.5 (0.312)	1742.0 (0.245)	1882.0 (0.194)
3			0.0 (0.000)	197.5 (1.000)	569.0 (0.720)	984.5 (0.554)	1377.5 (0.436)	1719.0 (0.348)	1997.5 (0.281)
4				0.0 (0.000)	197.5 (1.000)	587.5 (0.744)	1046.5 (0.589)	1502.5 (0.475)	1918.0 (0.388)
5					0.0 (0.000)	197.5 (1.000)	605.5 (0.766)	1103.5 (0.621)	1617.0 (0.512)
6						0.0 (0.000)	197.5 (1.000)	620.5 (0.786)	1152.5 (0.648)
7							0.0 (0.000)	197.5 (1.000)	633.0 (0.801)
8								0.0 (0.000)	197.5 (1.000)
9									0.0 (0.000)

Moments expressed as ratios of free cantilever moments are given in the parenthesis

Table-5.2

Girder Moments in t-m during erection of a Bridge with unequal pannel lengths (n= 36)

Cable point No.	First stage	Second stage	Third stage	Fourth stage	Fifth stage	Sixth stage	Seventh stage	Eighth stage	Ninth stage
1	2	3	4	5	6	7	8	9	10
Tower	98.7 (1.000)	385.5 (0.976)	779.5 (0.878)	1314.0 (0.684)	1676.5 (0.500)	1872.5 (0.365)	1964.0 (0.244)	1949.0 (0.166)	1882.5 (0.118)
1	0.0 (0.000)	98.7 (1.000)	351.3 (0.890)	802.5 (0.698)	1182.0 (0.519)	1442.0 (0.378)	1644.0 (0.257)	1737.5 (0.180)	1757.0 (0.129)
2		0.0 (0.000)	98.7 (1.000)	455.0 (0.799)	852.0 (0.593)	1185.0 (0.439)	1518.0 (0.308)	1743.0 (0.223)	1875.5 (0.164)
3			0.0 (0.000)	197.5 (1.000)	568.0 (0.719)	953.5 (0.536)	1415.0 (0.388)	1788.0 (0.290)	2056.0 (0.218)
4				0.0 (0.000)	197.5 (1.000)	571.5 (0.723)	1159.0 (0.541)	1729.0 (0.416)	2207.5 (0.320)
5					0.0 (0.000)	197.5 (1.000)	767.5 (0.738)	1455.0 (0.572)	2116.0 (0.445)
6						0.0 (0.000)	330.7 (1.000)	984.0 (0.744)	1737.0 (0.584)
7							0.0 (0.000)	330.7 (1.000)	1009.0 (0.763)
8								0.0 (0.000)	330.7 (1.000)
9									0.0 (0.000)

Moments expressed as ratios of free cantilever moment are given in the parenthesis.

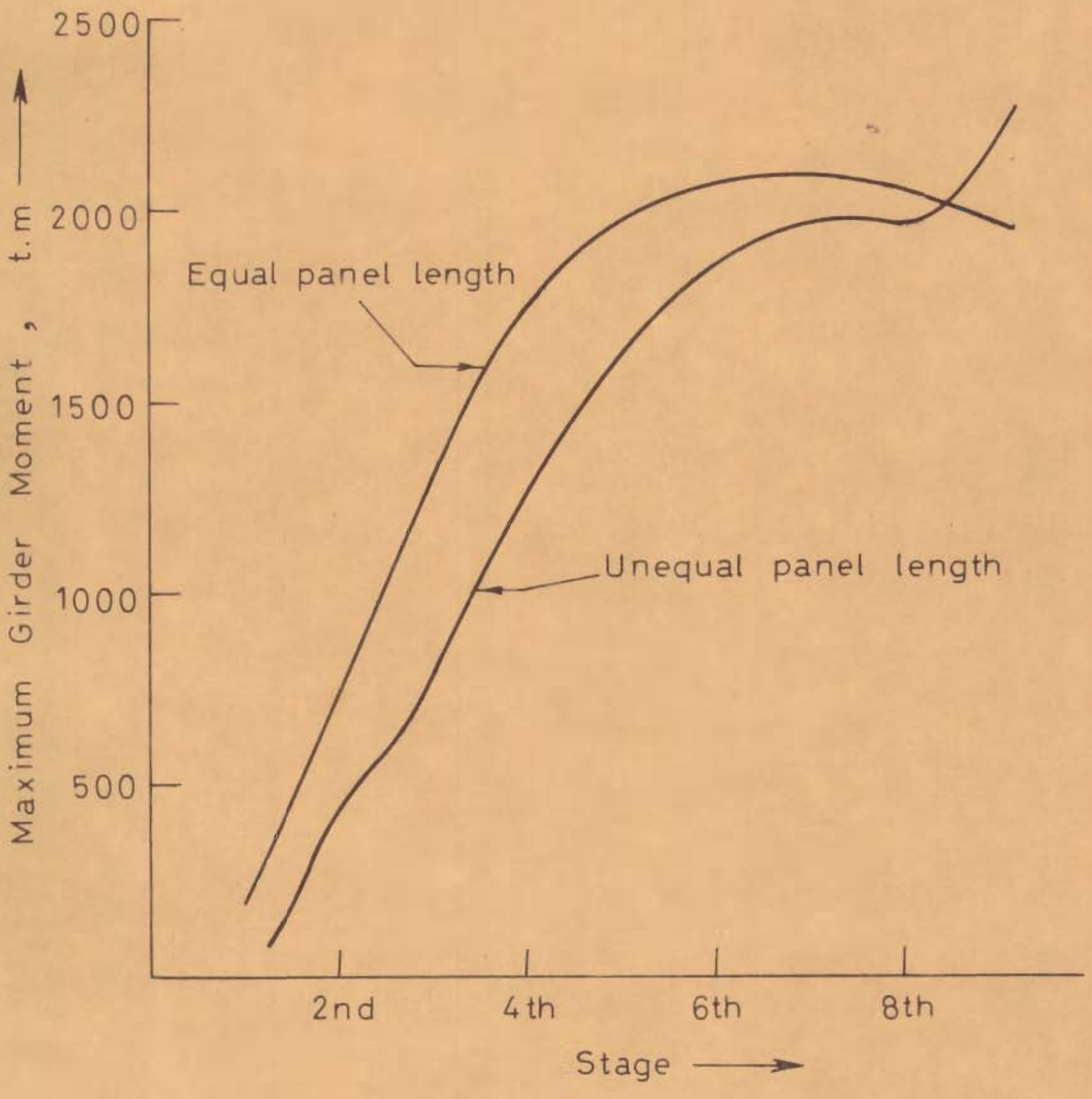


Fig.5.3 Maximum Girder Moment in Various Stages of Erection. (Ref. Fig 5.2)

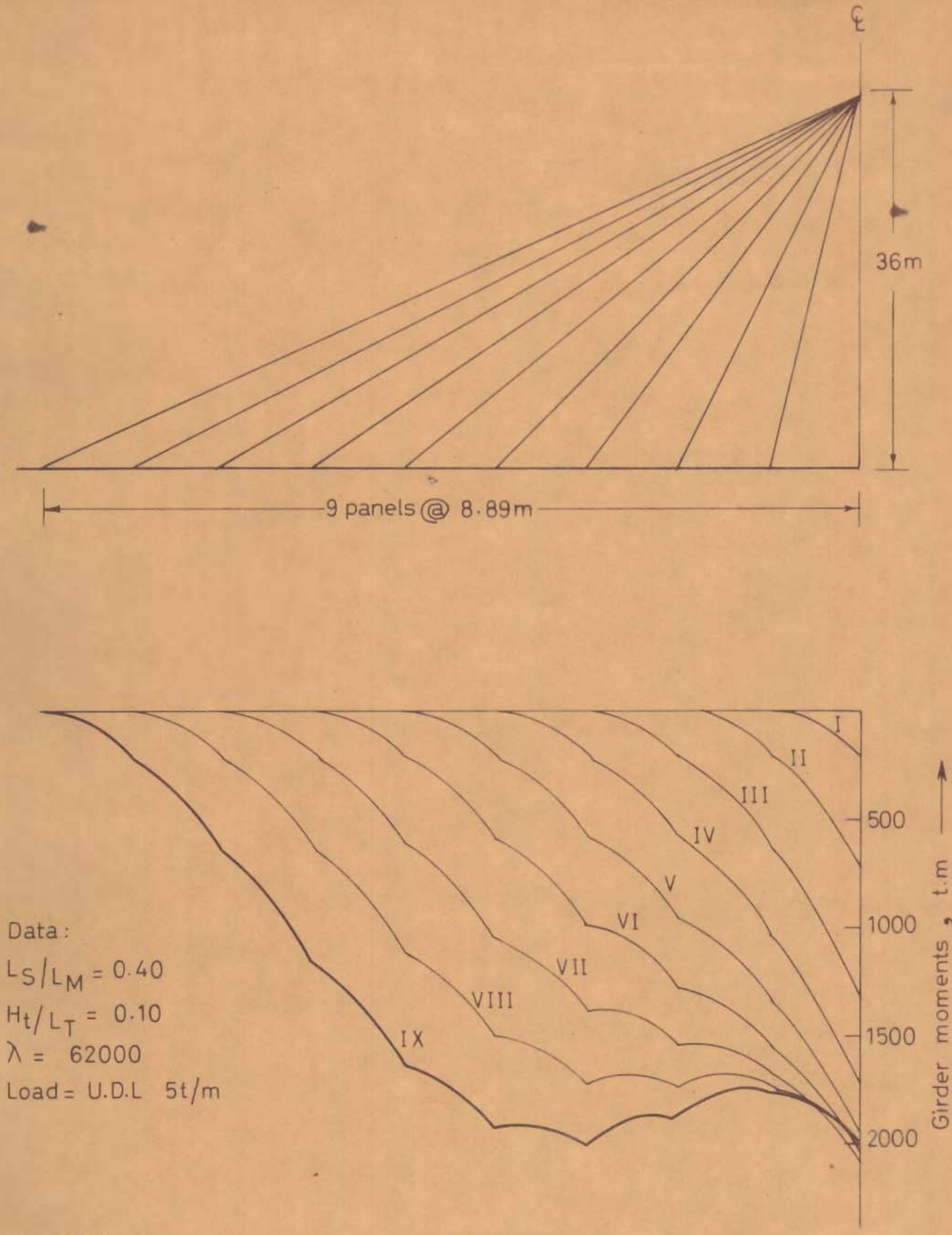


Fig. 5.4. Girder Moments During Erection for the Structure in Fig. 5.2

Table-5.3

Cable tensions in tonnes during erection of a bridge with equal panel lengths (n=36)

Cable point No. 1	First stage 2	Second stage 3	Third stage 4	Fourth stage 5	Fifth stage 6	Sixth stage 7	Seventh stage 8	Eighth stage 9	Nin-th stage 10
1	0.0 (0.000)	5.6 (0.126)	11.7 (0.263)	15.0 (0.337)	15.6 (0.351)	14.3 (0.322)	11.8 (0.265)	8.7 (0.196)	5.3 (0.119)
2		0.0 (0.000)	21.5 (0.484)	38.2 (0.859)	48.6 (1.093)	53.8 (1.210)	55.1 (1.240)	53.9 (1.213)	50.9 (1.145)
3			0.0 (0.000)	31.4 (0.706)	54.6 (1.228)	69.9 (1.573)	78.6 (1.768)	82.4 (1.854)	86.6 (1.948)
4				0.0 (0.000)	34.9 (0.785)	61.4 (1.381)	79.8 (1.795)	91.4 (2.056)	97.5 (2.193)
5					0.0 (0.000)	36.3 (0.817)	64.8 (1.458)	85.6 (1.926)	99.7 (2.243)
6						0.0 (0.000)	37.1 (0.835)	67.4 (1.516)	90.5 (2.036)
7							0.0 (0.000)	38.1 (0.857)	69.9 (1.573)
8								0.0 (0.000)	39.2 (0.882)
9									0.0 (0.000)

Tensions expressed as ratios of weight parameter (F). (see inset of Fig.5.5, art. 5.3) at a corresponding cable point are given in the parenthesis.

Table-5.4

Cable tensions in tonnes during erection of a bridge with unequal panel lengths (n=36)

Cable No.	First stage	Second stage	Third stage	Fourth stage	Fifth stage	Sixth stage	Seventh stage	Eighth stage	Ninth stage
1	2	3	4	5	6	7	8	9	10
1	0.0 (0.000)	1.5 (0.048)	3.5 (0.113)	5.4 (0.174)	5.4 (0.174)	3.9 (0.126)	0.7 (0.023)	-3.3 (-0.106)	-7.6 (-0.245)
2		0.0 (0.000)	7.3 (0.235)	18.1 (0.584)	25.5 (0.823)	28.9 (0.932)	29.3 (0.945)	28.9 (0.932)	22.8 (0.735)
3			0.0 (0.000)	21.6 (0.573)	38.9 (1.031)	49.8 (1.320)	56.8 (1.506)	58.3 (1.545)	56.0 (1.484)
4				0.0 (0.000)	31.4 (0.706)	55.0 (1.237)	75.4 (1.696)	86.5 (1.946)	90.6 (2.038)
5					0.0 (0.000)	35.0 (0.787)	70.6 (1.588)	95.1 (2.139)	109.5 (2.463)
6						0.0 (0.000)	49.3 (0.967)	88.2 (1.730)	115.5 (2.266)
7							0.0 (0.000)	55.2 (0.960)	99.6 (1.732)
8								0.0 (0.000)	58.8 (1.023)
9									0.0 (0.000)

Tensions expressed as ratios of a weight parameter (F) (see inset of Fig.5.5,art.5.3)at corresponding cable point are given in the parenthesis.

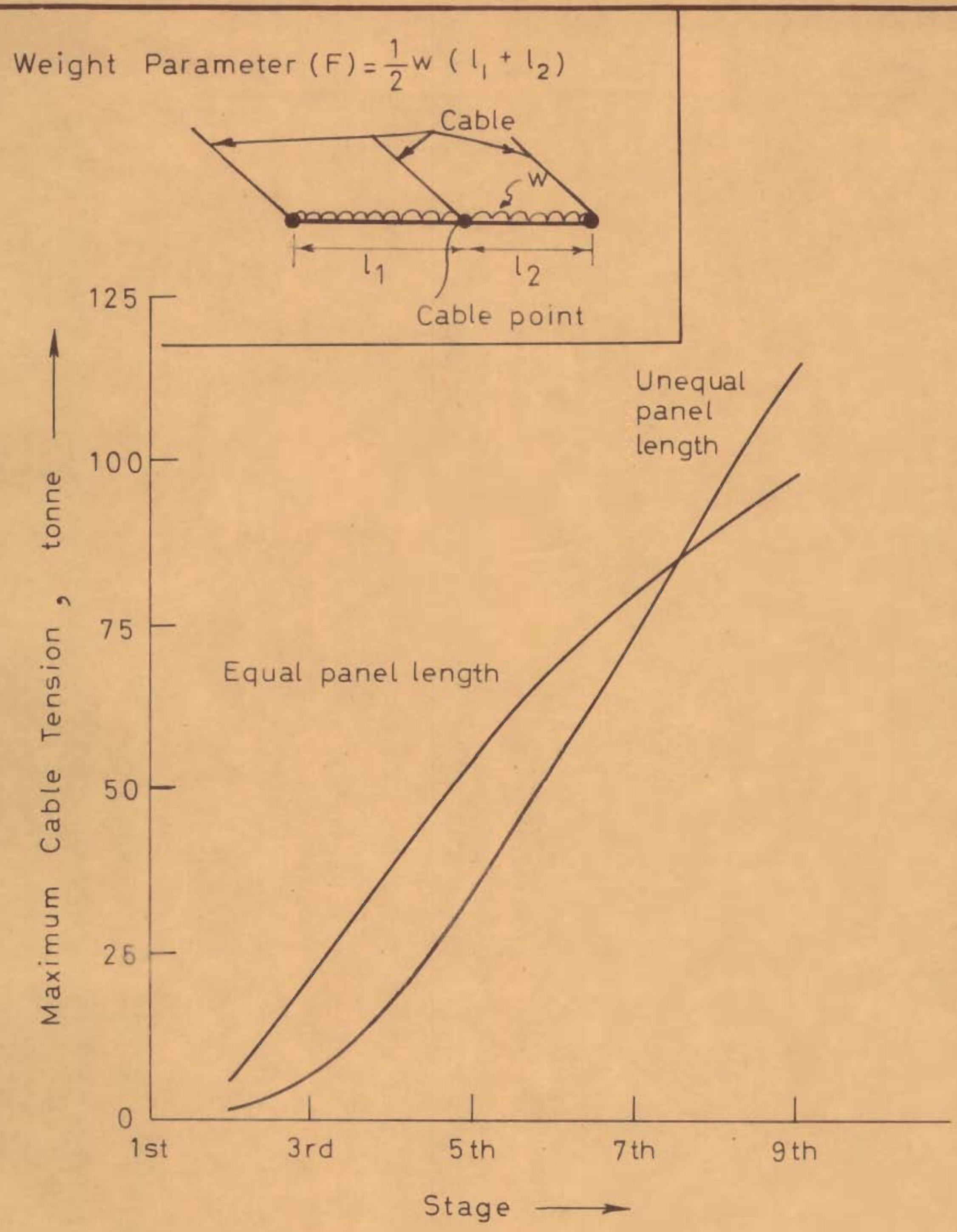


Fig.5.5. Maximum Cable Tension in Various Stages of Erection. (Ref. Fig.5.2)

initial stages the maximum tension in the cables is larger in the bridge with equal panel lengths as compared to those in the case with unequal panel lengths. In the last two stages this tension becomes larger in the latter case.

Girder Deflections

Table 5.5 and 5.6 give the values of girder deflections at each stage of erection for bridges with equal and unequal panel lengths respectively. These tables give the absolute values of deflections as well as their ratios to the main span of 200 m. The variation of the maximum deflection through different stages in both the cases is shown in Fig.5.6. Figure 5.6(a) shows the deflected shapes of the girder in the final stage of erection for the two bridges. Figure 5.6 indicates that the girder deflections are initially smaller in the bridge with unequal panel lengths as compared to those in the bridge with equal panel lengths. In the last two stages the deflections become smaller in the latter case.

5.4 REDUCTION OF GIRDER MOMENTS BY INTRODUCING APPROPRIATE DEGREE OF CABLE PRESTRESS DURING ERECTION

The girder moments obtained during erection of cable bridges can be large as may be seen from results in art.5.3. A proposed scheme to reduce these girder moments is discussed herein. Minimum moments will occur if cables provide non-deflecting supports to the girder at their connection point to the girder. The analysis proceeds on the premise that

Table - 5.5

Girder Deflections in m during erection of a bridge with equal panel lengths (n=36)

Cable No.	First stage * 10^{-4}	Second stage 10^{-4}	Third stage 10^{-4}	Fourth stage 10^{-4}	Fifth stage 10^{-4}	Sixth stage 10^{-4}	Seventh stage 10^{-4}	Eighth stage 10^{-4}	Ninth stage 10^{-4}
1	2	3	4	5	6	7	8	9	10
1	0.0000 (0.000)	0.0015 (0.075)	0.0040 (0.200)	0.0055 (0.275)	0.0065 (0.325)	0.0070 (0.350)	0.0070 (0.350)	0.0070 (0.350)	0.0070 (0.350)
2		0.0045 (0.225)	0.0120 (0.600)	0.0185 (0.925)	0.0230 (1.150)	0.0255 (1.275)	0.0270 (1.350)	0.0275 (1.375)	0.0278 (1.390)
3			0.0140 (0.700)	0.0280 (1.400)	0.0385 (1.925)	0.0460 (2.300)	0.0505 (2.525)	0.0535 (2.675)	0.0545 (2.725)
4				0.0225 (1.125)	0.0420 (2.100)	0.0565 (2.825)	0.0690 (3.450)	0.0760 (3.800)	0.0805 (4.025)
5					0.0295 (1.475)	0.0555 (2.775)	0.0765 (3.825)	0.0925 (4.625)	0.1040 (5.200)
6						0.0388 (1.940)	0.0715 (3.575)	0.0995 (4.975)	0.1210 (6.050)
7							0.0478 (2.390)	0.0910 (4.550)	0.1275 (6.375)
8								0.0595 (2.975)	0.1140 (5.700)
9									0.0735 (3.675)

Deflections expressed as ratios of main span of 200 m are given in the parenthesis.

* The values given in the parenthesis are to be multiplied by 10^{-4}.

Table-5.6

Girder deflections in m during erection of a bridge with unequal panel length (n=36)

Cable No.	First stage * 10^{-4}	Second stage 10^{-4}	Third stage 10^{-4}	Fourth stage 10^{-4}	Fifth stage 10^{-4}	Sixth stage 10^{-4}	Seventh stage 10^{-4}	Eighth stage 10^{-4}	Nin-th stage 10^{-4}
1	2	3	4	5	6	7	8	9	10
1	0.0000 (0.000)	0.0005 (0.025)	0.0010 (0.050)	0.0020 (0.100)	0.0030 (0.150)	0.0035 (0.175)	0.0035 (0.175)	0.0035 (0.175)	0.0035 (0.175)
2		0.0010 (0.050)	0.0035 (0.175)	0.0075 (0.375)	0.0105 (0.525)	0.0120 (0.600)	0.0135 (0.675)	0.0140 (0.700)	0.0140 (0.700)
3			0.0045 (0.225)	0.0125 (0.625)	0.0190 (0.950)	0.0235 (1.175)	0.0275 (1.375)	0.0290 (1.450)	0.0295 (1.475)
4				0.0145 (0.725)	0.0290 (1.450)	0.0400 (2.000)	0.0500 (2.500)	0.0560 (2.800)	0.0590 (2.950)
5					0.0230 (1.150)	0.0430 (2.150)	0.0645 (3.225)	0.0790 (3.950)	0.0890 (4.450)
6						0.0305 (1.525)	0.0670 (3.350)	0.0960 (4.800)	0.1175 (5.875)
7							0.0580 (2.900)	0.1120 (5.600)	0.1575 (7.875)
8								0.0830 (4.150)	0.1590 (7.950)
9									0.1115 (5.575)

Deflections expressed as ratios of main span of 200 m are given in the parenthesis.

* The values given in the parenthesis are to be multiplied by 10^{-4}.

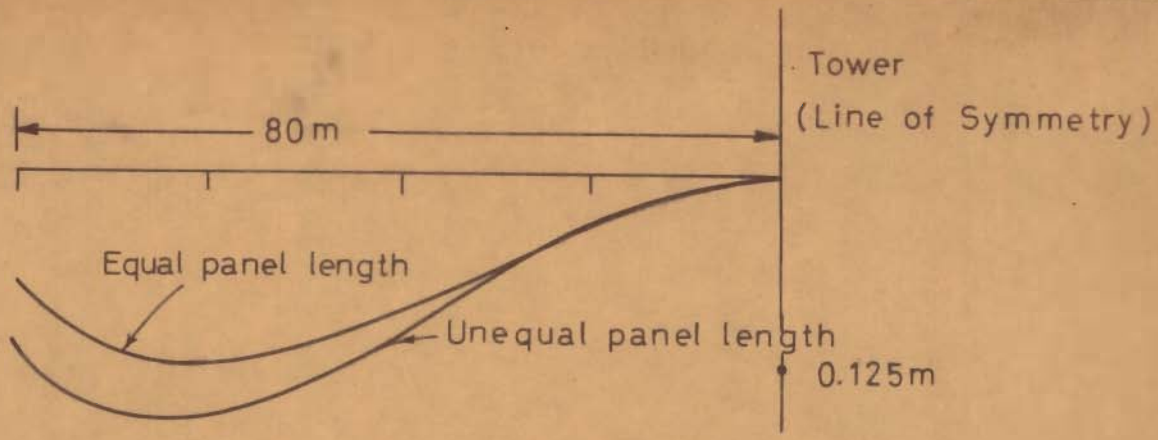


Fig.5.6(a) Deflected Shape of the Girder in the Final Stage of Erection. (Ref. Fig.5.2)

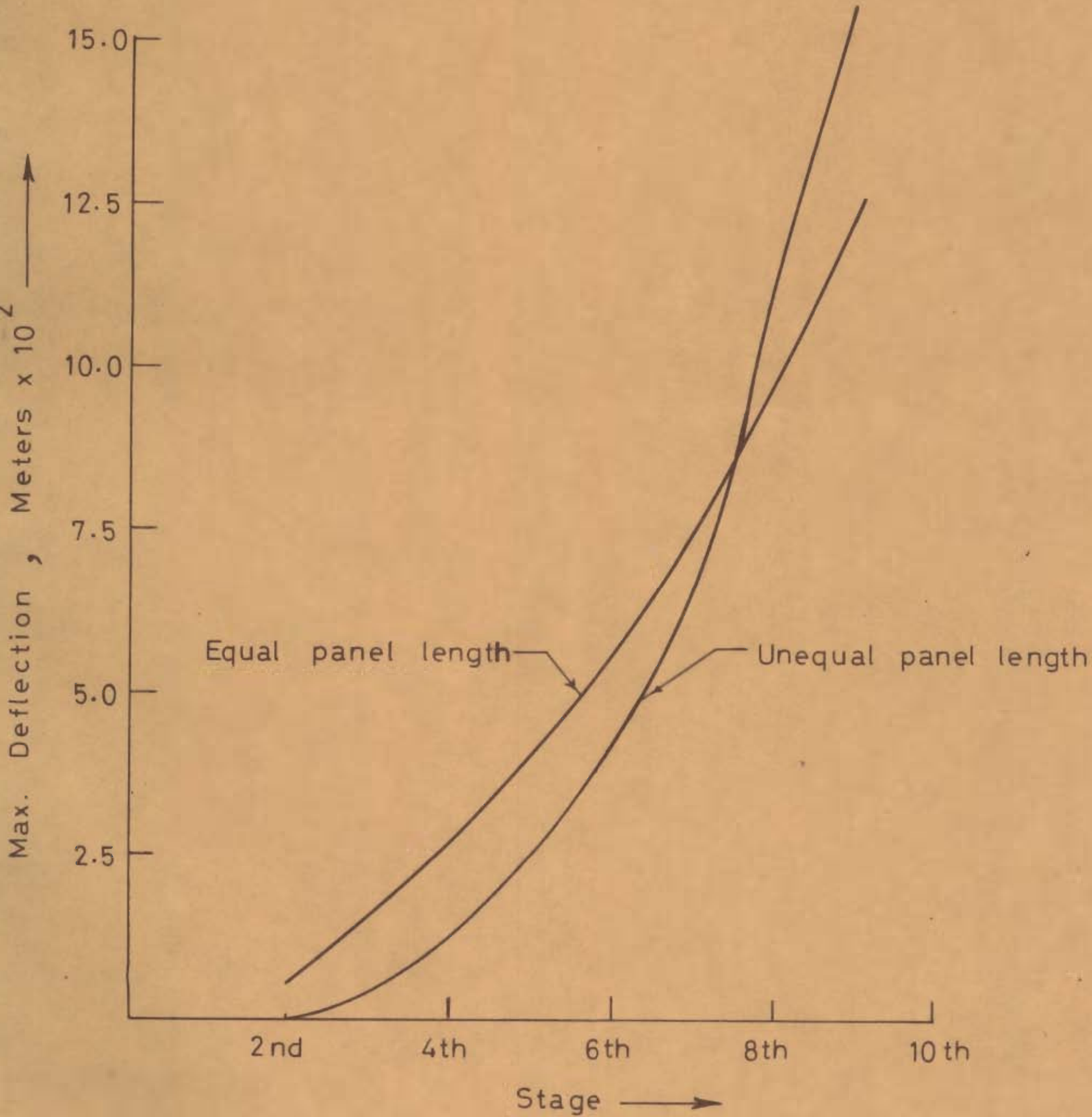


Fig.5.6 Maximum Deflection in Various Stages of Erection. (Ref. Fig.5.2)

this ideal can be achieved by choosing a sequence and value of prestressing forces in the cables. Tension T_f in any cable is the sum of the initial prestress T_i and the change of tension in the Cable T_w due to dead loads and due to prestressing of other cables. The limiting value of T_f is obtained by assuming that cables lead to the ideal position stipulated above (Fig. 5.7).

The relationships between initial prestress, dead load tension, cable tension due to prestressing of other cables and the final forces in the cables is given mathematically in Eq.5.1, and the values of T_i are obtained therefrom.

$$
\begin{array}{l}
T_i(1)\,a_{11} + T_i(2)\,a_{12} + T_i(3)\,a_{13} + \ldots + T_i(r)\,a_{1r} + \ldots + T_i(n)\,a_{1n} = T_f(1) - T_w(1,2) - \ldots - T_w(1,r) - \ldots - T_w(1,n) \\
\qquad\qquad T_i(2)\,a_{22} + T_i(3)\,a_{23} + \ldots + T_i(r)\,a_{2r} + \ldots + T_i(n)\,a_{2n} = T_f(2) - T_w(2,3) - \ldots - T_w(2,r) - \ldots - T_w(2,n) \\
\qquad\qquad\qquad\qquad T_i(3)\,a_{33} + \ldots + T_i(r)\,a_{3r} + \ldots + T_i(n)\,a_{3n} = T_f(3) - T_w(3,4) - \ldots - T_w(3,r) - \ldots - T_w(3,n) \\
\qquad\qquad\qquad\qquad \ldots \qquad\qquad \ldots \qquad\qquad \ldots \\
\qquad\qquad\qquad\qquad \ldots \qquad\qquad \ldots \qquad\qquad \ldots \\
\qquad\qquad\qquad\qquad\qquad\qquad T_i(r)\,a_{rr} + \ldots + T_i(n)\,a_{rn} = T_f(r) - T_w(r,r+1) - \ldots - T_w(r,n) \\
\qquad\qquad\qquad\qquad \ldots \qquad\qquad \ldots \qquad\qquad \ldots \\
\qquad\qquad\qquad\qquad \ldots \qquad\qquad \ldots \qquad\qquad \ldots \\
\qquad\qquad\qquad\qquad\qquad\qquad T_i(n)\,a_{nn} \qquad\qquad\qquad = T_f(n)
\end{array}
\qquad \ldots\ldots \quad (5.1)
$$

where $T_f(r)$ = final tension in rth cable due to dead load and prestressing.

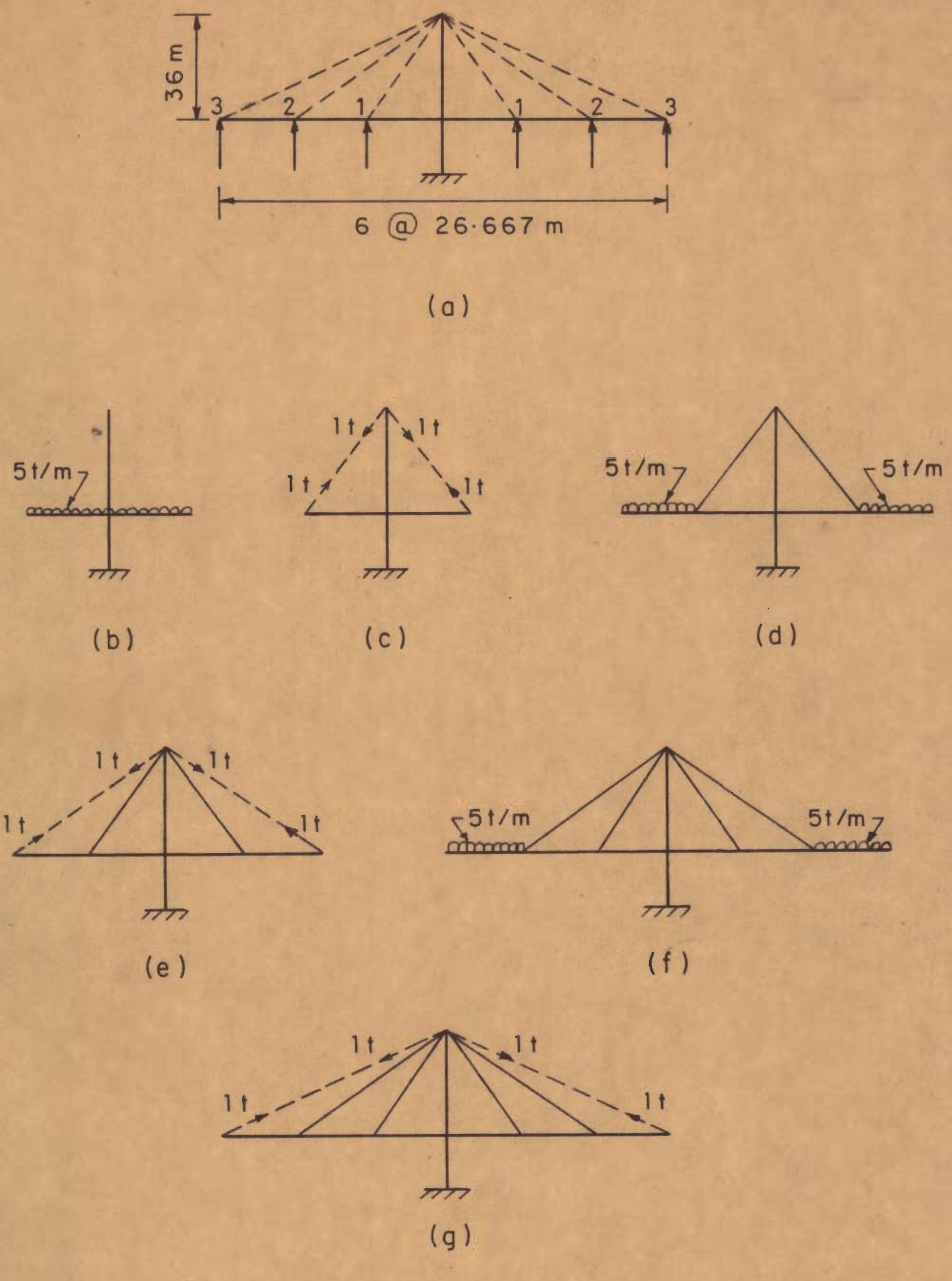


Fig. 5.7 Various Stages of Cable Stayed Bridge for Analysis During Erection

$T_i(r)$ = prestress to be provided in rth cable,

$T_w(i,j)$ = dead load tension in the ith cable due to addition of jth panel,

n = number of cables on each side of the tower,

A_{ij} = change in tension of ith cable due to unit prestress in jth cable.

A_{ij} are similar to tension coefficients, and are determined by analysing the structure in each stage of erection with a unit load in the direction of exterior most cables. The value of A_{ii} when $i = j = 1,2,3,\ldots,n$ is equal to +1.

5.4.1 Results

In this section the behaviour of radiating type cable bridges during erection is studied. Whereas the bridge girder is in actuality like a beam continuous over elastic supports provided by the cables, from the point of view of minimising girder moments the ideal situation will be if cables were to act as nondeflecting supports at their attachment points. The details of the analysis are given in Art.5.4. The behaviour of bridges with 12,20,28, and 36 cables (per plane) respectively has been studied during their erection and is described below:

5.4.1.1 Bridge with 12 cables

For a bridge with 12 cables the three stages of erection are shown in Fig.5.7. The dimensions and the sectional properties of the elements are given in Fig.5.8. The procedure of erection has already been described. The discussion of results in brief is given below.

Girder Moments

Table 5.7 gives the girder moments at the tower and at the cable points. This table gives the absolute values of girder moments as well as their ratios to the support moments ($w\ell^2/12$) in a fixed ended beam with distance between the cable points treated as the span ($w = 5$ t/m and $\ell = 26.67$ m in this case). Figure 5.8 shows the distribution of the bending moment in the girder in each stage separately. The girder moments at the end of first and second stages of erection are sagging in nature. The maximum sagging moment is at point 1 in the second stage of erection and is equal to $W\ell^2/14.3$. In the third stage the girder moments at the tower and at points 1 and 2 become hogging. The values of the moments at these points are $w\ell^2/11.56$, $w\ell^2/13.0$ and $w\ell^2/9.45$ respectively. These moments are approximately equal to those obtained in an equivalent continuous beam.

Cable Tension

Table 5.8 gives the tensions in the cables during erection. This table gives the absolute values of cable tensions, and also their ratios to a weight parameter F at the corresponding cable points (See Fig.5.5). The maximum tension during erection is obtained in the second cable in the third

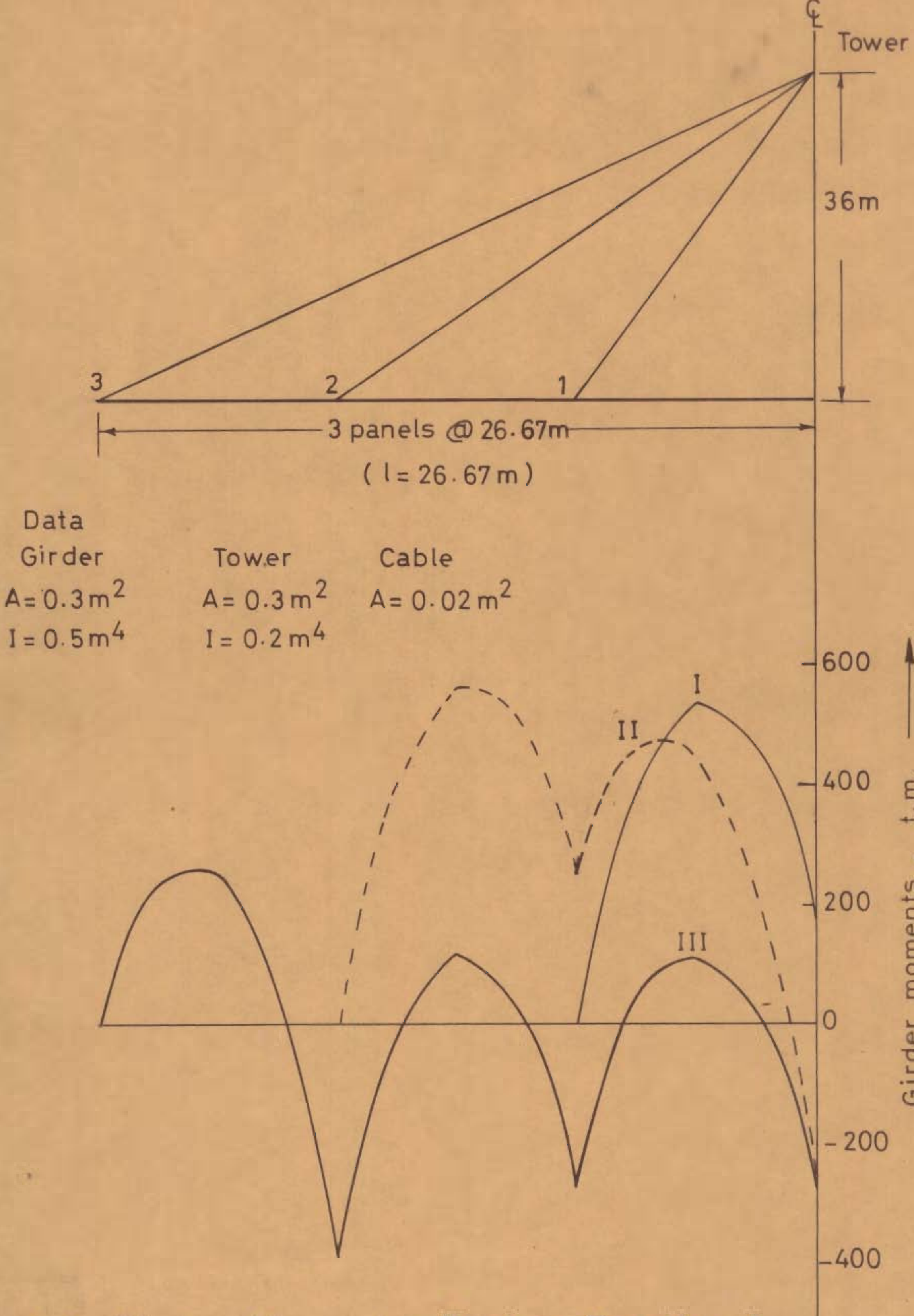


Fig. 5.8 Girder Moments During Erection for a Bridge with n=12

Table - 5.7

Girder moments in t-m during erection of a bridge (n=12)

Cable point No.	First stage	Second stage	Third stage
1	2	3	4
Tower	176.0 (0.594)	-269.6 (-0.010)	-307.7 (-1.038)
1	0.0 (0.000)	249.0 (0.840)	-273.5 (-0.923)
2		0.0 (0.000)	-376.1 (-1.209)
3			0.0 (0.000)

Moments expressed as ratios of support moment in a fixed ended beam with span ℓ ($w\ell^2/12$) are given in the parenthesis.

Table - 5.8

Cable Tensions in tonnes during erection of a bridge (n=12)

Cable No.	First stage	Second stage	Third stage
1	2	3	4
1	91.2 (1.368)	130.3 (0.977)	160.0 (1.200)
2		135.9 (2.038)	270.5 (2.028)
3			128.1 (1.921)

Tensions expressed as ratios of weight parameter (F) (see inset of Fig. 5.5 and art 5.3) at corresponding cable point are given in the parenthesis.

stage and is equal to 2.03 $w\ell$.

5.4.1.2 Bridges with 20, 28 and 36 cables per plane

The bridges with 20, 28 and 36 cables per plane are also analysed to investigate their behaviour during erection. Tables 5.9, 5.10 and 5.11 give the girder moments for bridges with 20, 28, and 36 cables respectively. These tables give the absolute values of moments as well as their ratios to the support moment of an equivalent fixed ended beam. The distribution of girder moments in each stage of erection for these bridges is given in Figs.5.9 to 5.11 respectively. This study indicates that the behaviour of these bridges is very similar to that of a bridge with 12 cables. With the introduction of appropriate values of cable pretension, the girder behaves as a continuous beam. The girder moments at all the cable points except the two points at the free end are approximately equal to $w\ell^2/12$. Tables 5.12, 5.13, and 5.14 give the values of cable tensions during erection of bridges with 20, 28 and 36 cables per plane respectively. These tables give the absolute values of cable tensions as well as their ratios to the parameter F at the corresponding cable point.

Table - 5.9

Girder moments in t-m during erection of a bridge (n=20)

Cable Points No.	First stage	Second stage	Third stage	Fourth stage	Fifth stage
1	2	3	4	5	6
Tower	542.0 (5.081)	290.0 (2.719)	55.6 (0.521)	-103.4 (-0.969)	-107.0 (-1.003)
1	0.0 (0.000)	171.6 (1.609)	94.0 (0.881)	42.6 (0.399)	-106.1 (-0.995)
2		0.0 (0.000)	174.5 (1.636)	257.9 (2.418)	-108.7 (-1.019)
3			0.0 (0.000)	335.4 (3.144)	-99.5 (-0.933)
4				0.0 (0.000)	-135.3 (-1.268)
5					0.0 (0.000)

Moments expressed as ratios of support moment in a fixed ended beam with span ℓ ($w\ell^2/12$) are given in the parenthesis.

Table - 5.10

Girdar Moments in t-m during erection of a bridge (n = 28)

Cable Points No.	First stage	Second stage	Third stage	Fourth stage	Fifth stage	Sixth stage	Seventh stage
1	2	3	4	5	6	7	8
Tower	447.5 (8.220)	458.9 (8.429)	287.0 (5.272)	143.3 (2.632)	35.7 (0.656)	-59.1 (-1.0855)	-53.8 (-0.988)
1	0.0 (0.000)	195.6 (3.593)	174.0 (3.196)	114.4 (2.101)	46.4 (0.852)	5.5 (0.101)	-53.5 (-0.983)
2		0.0 (0.000)	134.5 (2.471)	156.7 (2.878)	117.2 (2.153)	114.5 (2.103)	-54.1 (-0.994)
3			0.0 (0.000)	149.9 (2.753)	176.0 (3.234)	227.6 (4.181)	-53.6 (-0.985)
4				0.0 (0.000)	150.4 (2.763)	285.2 (5.239)	-55.0 (-1.010)
5					0.0 (0.000)	233.7 (4.293)	-50.6 (-0.930)
6						0.0 (0.000)	-69.5 (-1.277)
7							0.0 (0.000)

Moments expressed as ratios of support moment in a fixed ended beam with span ℓ ($w\ell^2/12$) are given in the parenthesis.

Table 5.11

Girder moments in tm during erection of a bridge (n = 36)

Cable Point No.	First stage	Second stage	Third stage	Fourth stage	Fifth stage	Sixth stage	Seventh stage	Eighth stage	Ninth stage
1	2	3	4	5	6	7	8	9	10
Tower	319.4 (9.699)	467.7 (14.206)	394.5 (11.980)	277.8 (8.436)	175.9 (5.342)	94.4 (2.866)	29.1 (0.884)	-39.4 (-1.196)	-31.1 (-0.945)
1	0.0 (0.000)	191.7 (5.821)	219.4 (6.663)	175.5 (5.329)	122.3 (3.714)	74.7 (2.268)	29.0 (0.881)	- 3.5 (-0.106)	-31.8 (-0.966)
2		0.0 (0.000)	117.7 (3.574)	141.5 (4.297)	127.1 (3.860)	103.1 (3.131)	65.3 (1.983)	54.9 (1.667)	-32.1 (-0.975)
3			0.0 (0.000)	102.8 (3.122)	138.3 (4.200)	143.8 (4.367)	115.9 (3.520)	127.5 (3.872)	-32.1 (-0.975)
4				0.0 (0.000)	106.5 (3.234)	158.4 (4.810)	155.8 (4.732)	197.7 (6.004)	-32.9 (-0.999)
5					0.0 (0.000)	117.4 (3.565)	160.8 (4.883)	240.3 (7.299)	-33.0 (-0.002)
6						0.0 (0.000)	111.6 (3.389)	233.6 (7.094)	-33.2 (-1.008)
7							0.0 (0.000)	163.9 (4.977)	-30.7 (-0.932)
8								0.0 (0.000)	-41.8 (-1.269)
9									0.0 (0.000)

Moment expressed as ratios of support moment in a fixed ended beam with span ℓ $(w\ell^2/12)$ are given in the parenthesis.

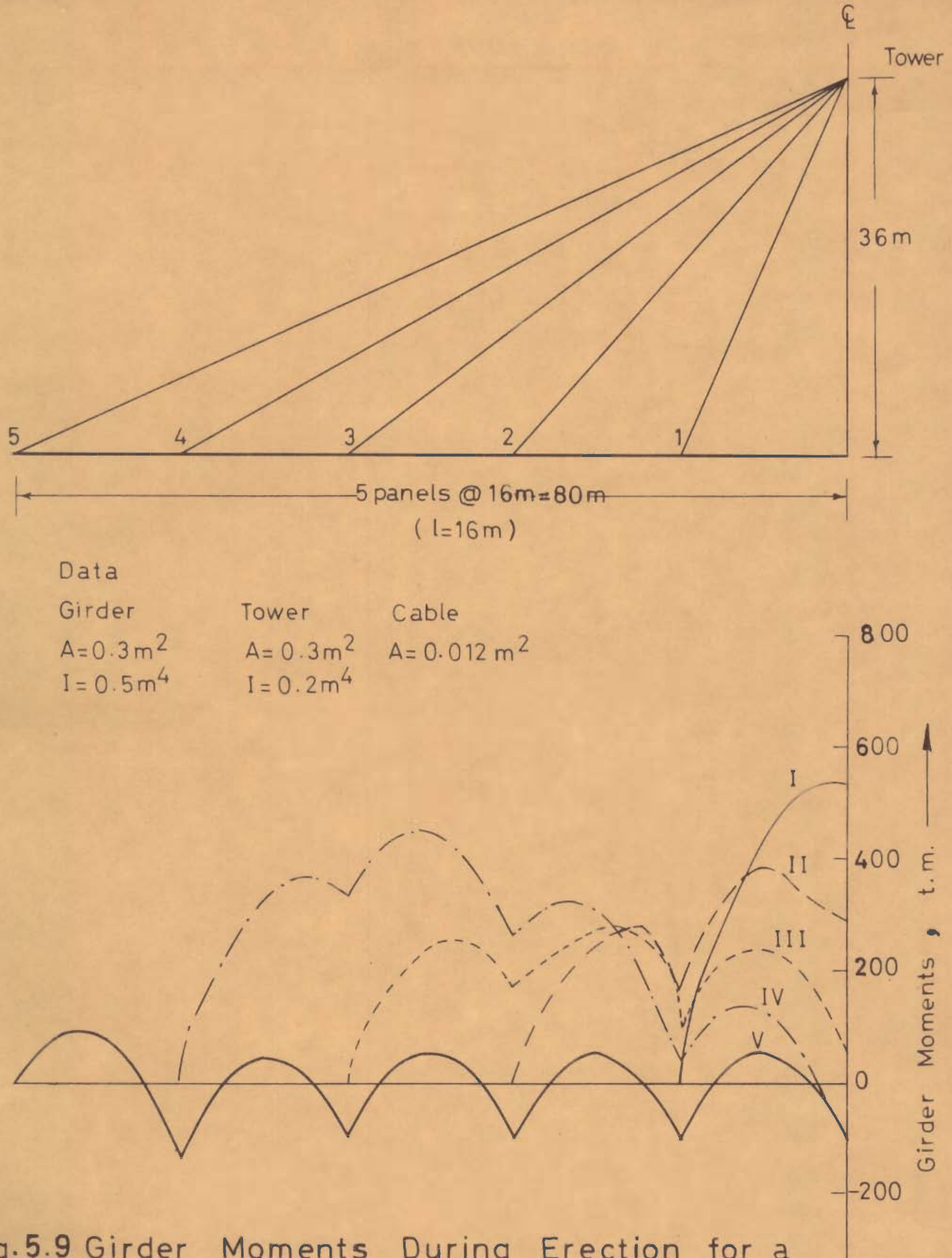


Fig. 5.9 Girder Moments During Erection for a Bridge with n = 20

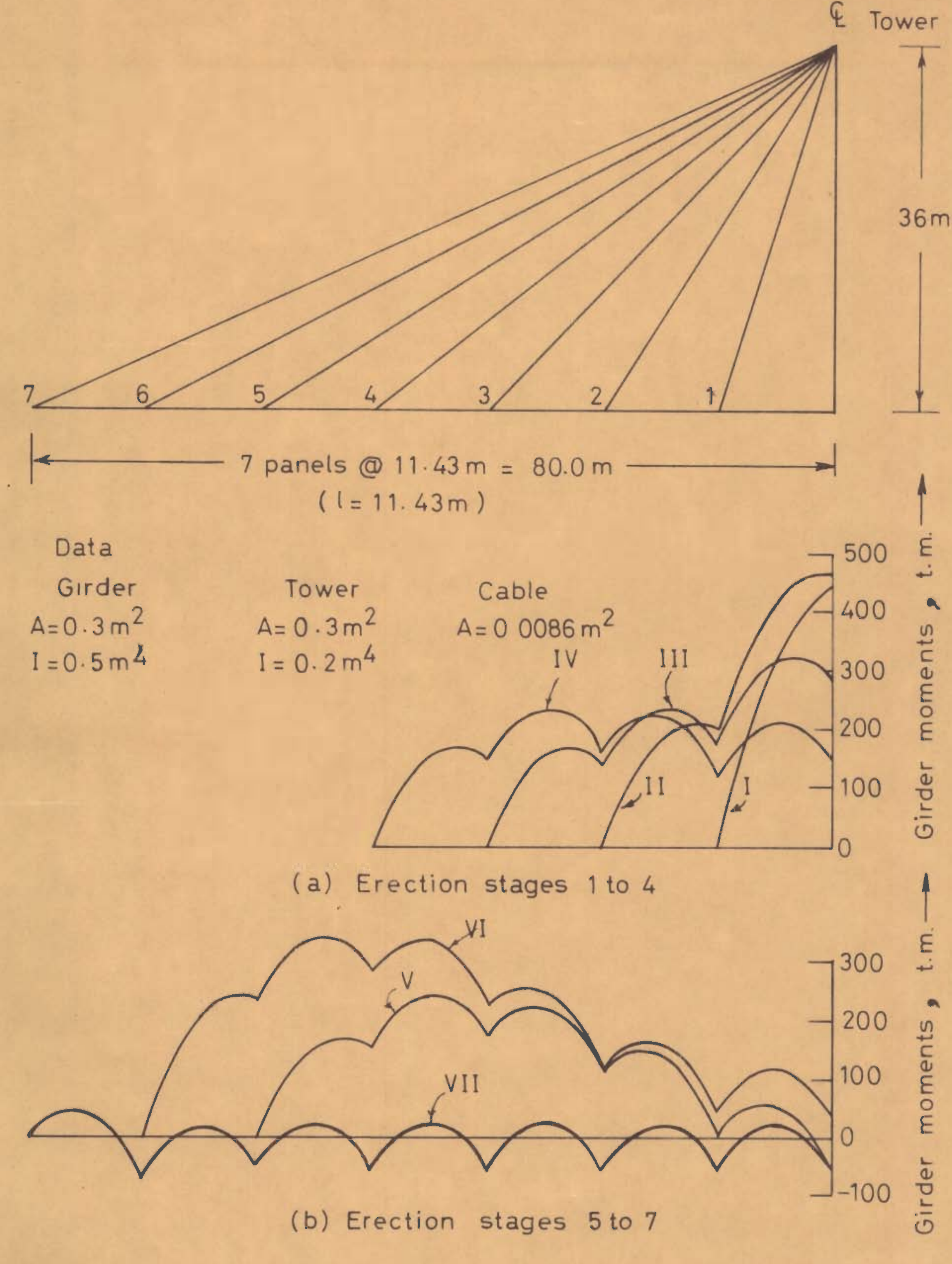


Fig. 5.10 Girder Moments During Erection for a Bridge with n = 28

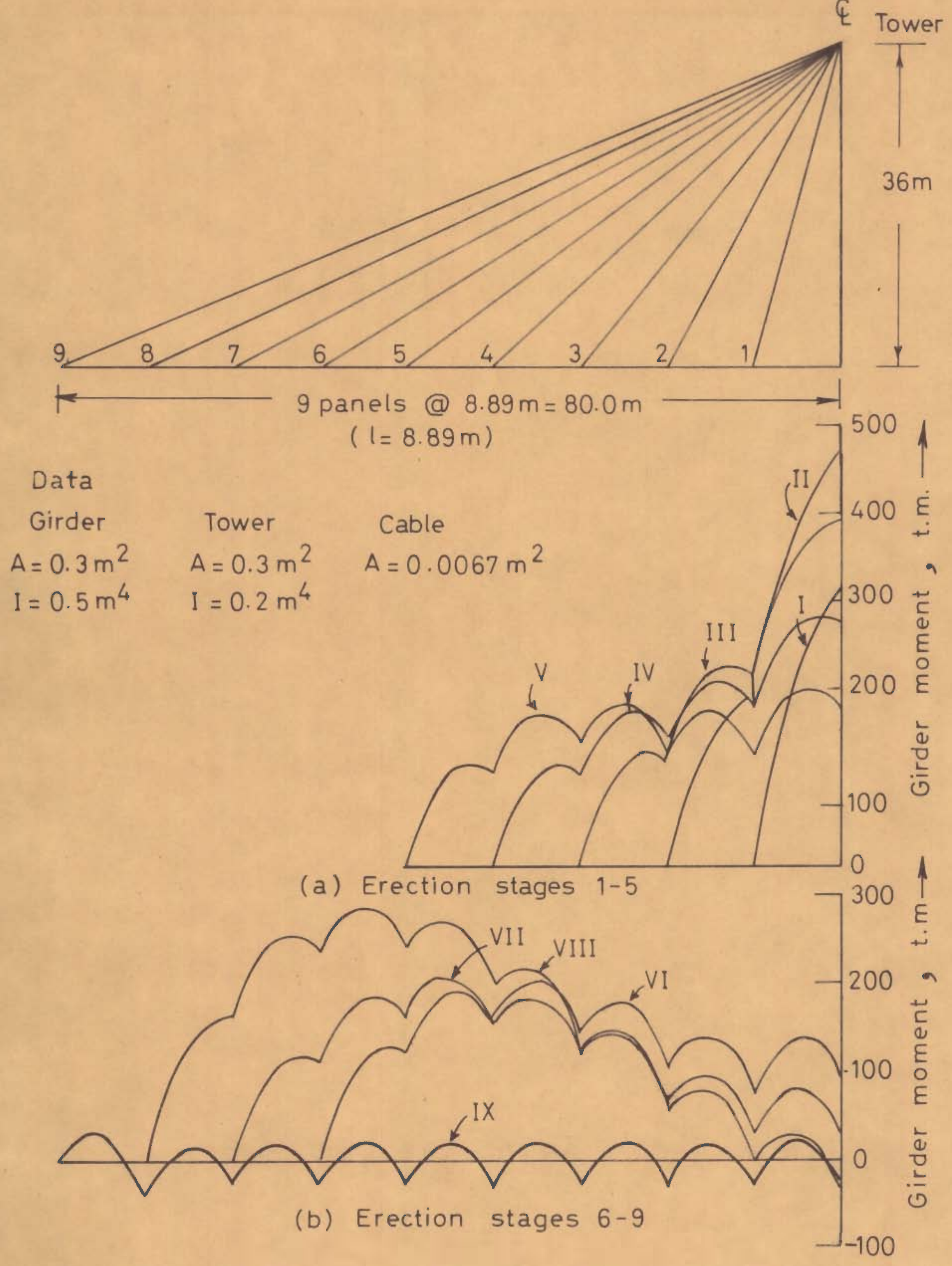


Fig.5.11 Girder Moments During Erection for a Bridge with n = 36

Table - 5.12

Cable Tensions in tonnes during erection of a bridge (n = 20)

Cable No.	First stage	Second stage	Third stage	Fourth stage	Fifth stage
1	2	3	4	5	6
1	80.87 (2.020)	83.8 (1.047)	90.4 (1.130)	92.1 (1.151)	87.3 (1.091)
2		67.9 (1.697)	85.6 (1.070)	100.1 (1.251)	108.1 (1.351)
3			85.0 (2.125)	90.3 (1.129)	128.6 (1.607)
4				124.3 (3.107)	185.0 (2.312)
5					76.9 (1.922)

Tensions expressed as ratios of weight parameter (F) (see inset of Fig. 5.5 and art 5.3) at corresponding cable point are given in the parenthesis.

Table - 5.13

Cable Tensions in tonnes during erection of a bridge (n = 28)

Cable No.	First stage	Second stage	Third stage	Fourth stage	Fifthe stage	Sxith stagre	Seventh stagre
1	2	3	4	5	6	7	8
1	71.1 (2.488)	66.2 (1.158)	66.8 (1.169)	66.6 (1.165)	65.6 (1.148)	64.1 (1.121)	60.1 (1.052)
2		54.1 (1.893)	57.8 (1.011)	62.3 (1.090)	66.2 (1.158)	68.1 (1.192)	67.5 (1.181)
3			55.7 (0.975)	61.6 (1.078)	68.8 (1.204)	72.0 (1.260)	78.1 (1.366)
4				67.4 (2.358)	74.7 (1.307)	77.0 (1.347)	93.3 (1.632)
5					78.3 (2.739)	77.3 (1.352)	103.4 (1.809)
6						105.5 (3.691)	139.4 (2.439)
7							54.9 (1.921)

Tensions expressed as ratios of weight parameter (F)(see inset of Fig.5.5 and art 5.3) at corresponding cable point are given in the parenthesis.

Table - 5.14

Cable Tensions in tonnes during erection of a Bridge (n = 36)

Cable No. 1	First stage 2	Second stage 3	Third stage 4	Fourth stage 5	Fifth stage 6	Sixth stage 7	Seventh stage 8	Eighth stage 9	Ninth stage 10
1	59.9 (2.694)	55.5 (1.248)	54.3 (1.221)	53.6 (1.206)	52.7 (1.180)	51.4 (1.156)	49.9 (1.123)	48.4 (1.089)	45.8 (1.030)
2		48.8 (2.195)	47.6 (1.071)	49.0 (1.102)	50.3 (1.132)	51.1 (1.150)	51.4 (1.156)	51.4 (1.156)	49.6 (1.116)
3			44.1 (1.984)	46.3 (1.042)	49.3 (1.109)	51.6 (1.161)	53.6 (1.206)	54.6 (1.228)	55.2 (1.242)
4				47.5 (2.137)	50.6 (1.138)	62.2 (1.400)	57.4 (1.291)	58.4 (1.314)	62.6 (1.415)
5					54.3 (2.443)	82.1 (1.847)	60.8 (1.368)	61.7 (1.388)	70.4 (1.584)
6						63.3 (2.848)	66.9 (1.505)	66.7 (1.501)	80.2 (1.804)
7							69.4 (3.122)	67.5 (1.518)	85.5 (1.923)
8								90.0 (4.048)	111.6 (2.511)
9									42.7 (1.921)

Tensions expressed as ratios of weight parameter (F) (see inset of Fig.5.5 and art 5.3) at corresponding cable point are given in the parenthesis.

CHAPTER VI

PARAMETRIC STUDY

6.1 GENERAL

A cable stayed girder bridge is a highly indeterminate structure. The methods which can be used for the analysis of these bridges require sectional properties of various elements of the bridge before actual analysis can be carried out. The designer should have an idea of the distribution of the values of cable tensions and girder and tower moments so as to make preliminary estimate of their cross sections. The literature available todate does not provide much guidance to the designer for making decisions regarding various parameters namely, (i) number of cables, (ii) side span to main span ratio, (iii) tower height to total span ratio, and (iv) ratio of cable and girder stiffnesses. In the absence of any guidline to decide the values of above parameters, the designer will have to analyse several trial cases to obtain the most economical combination of these parameters. The aim of this parametric study is to obtain some generalized results for proper understanding of the structural behaviour and arriving at appropriate design data. The studies covered in this chapter can be divided in two categories as follows:.

The first part of the study includes the effect of various parameters on certain critical stress resultants. Specifically the effect of following parameters on girder moments, cable forces and deflections has been studied:

a. Flexural rigidity of tower
b. Flexural rigidity of girder
c. Cable rigidity
d. Central panel length in the main span
e. Configuration-a comparison between harp and radiating type of cable arrangements
f. Effect of tower height

The radiating and harp arrangements are commonly used in cable stayed bridges. In this study the effect of various parameters on the behaviour of bridges with these systems is investigated in detail. The dimensions and other parameters chosen for the study are shown in Fig.6.1. Each parameter is varied individually for finding its effect. Uniformly distributed load over the whole span has been considered.

The second part of the study consists in making a set of design curves in dimensionless form. These design curves include the variation of the following parameters.

a. Number of cables
b. Side span to main span ratio
c. Tower height to total span ratio
d. Length of central panel in main span, and
e. Cable stiffness to girder stiffness ratio

Data

Girder	Tower	Cable
$A = 0.3\,m^2$	$A = 0.3\,m^2$	$A = 0.03\,m^2$ per Cable
$I = 0.5\,m^4$	$I = 0.2\,m^4$	$I = 0.0$

Loading: 1 tonne per metre throughout

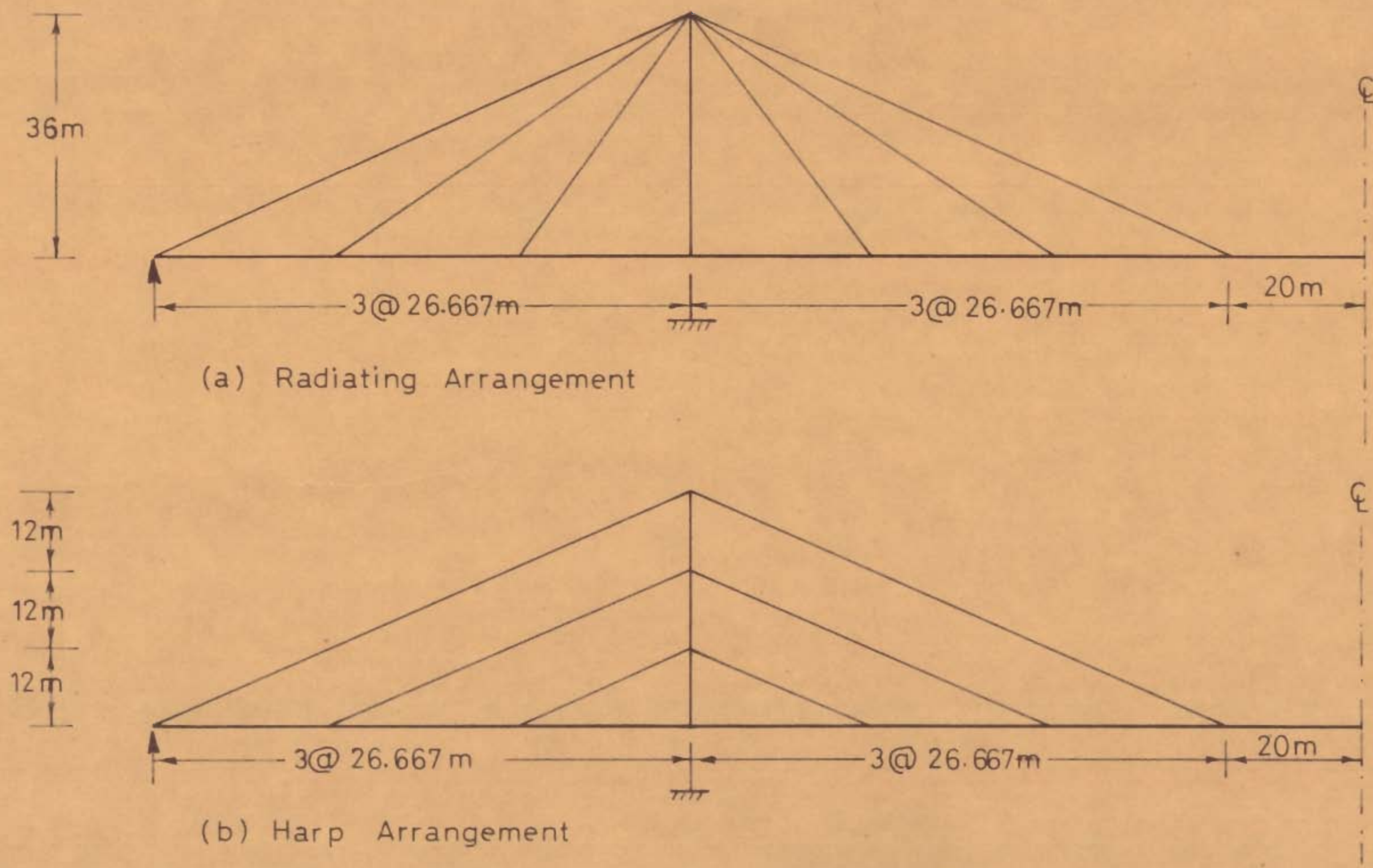


Fig. 6.1. Elevation of Bridges Considered for Parametric Study

6.2 EFFECT OF FLEXURAL RIGIDITY OF TOWER

The effect of flexural stiffness of tower on the girder and tower moments, cable tensions and deflections has been studied. The effect of tower support condition has also been considered. Two extreme cases of the tower base condition, namely fixed and hinged are considered. The moment of inertia of the tower was varied between the values 0.1 to 0.5 m^4. The other parameters considered are shown in Fig.6.1.

The variation of cable tension with tower stiffness for both harp and radiating arrangements of cable stayed bridges is shown in Fig.6.2. It may be seen that the variation of maximum cable tension with tower stiffness is almost linear. In the radiating arrangement, the decrease in maximum cable tension is 1.71% . For the harp arrangement there is a corresponding increase of 1.21% . Similarly, there is little effect of changing the support condition at the tower base. The maximum increase in tension is 2.17% and 1.72% respectively for the radiating and harp arrangements, when tower base condition is changed from fixed to hinged. The cable tension is larger approximately 10-11% in the harp arrangement as compared to the radiating arrangement.

The variation of girder deflection with tower stiffness is shown in Fig.6.3. This relationship is also almost linear. The decrease in girder deflections is 0.75% and 1.12% for the radiating and the harp arrangements respectively for the extreme change in tower moment of inertia. The increase in the maximum deflection value is 1.18% and 1.59% respectively for

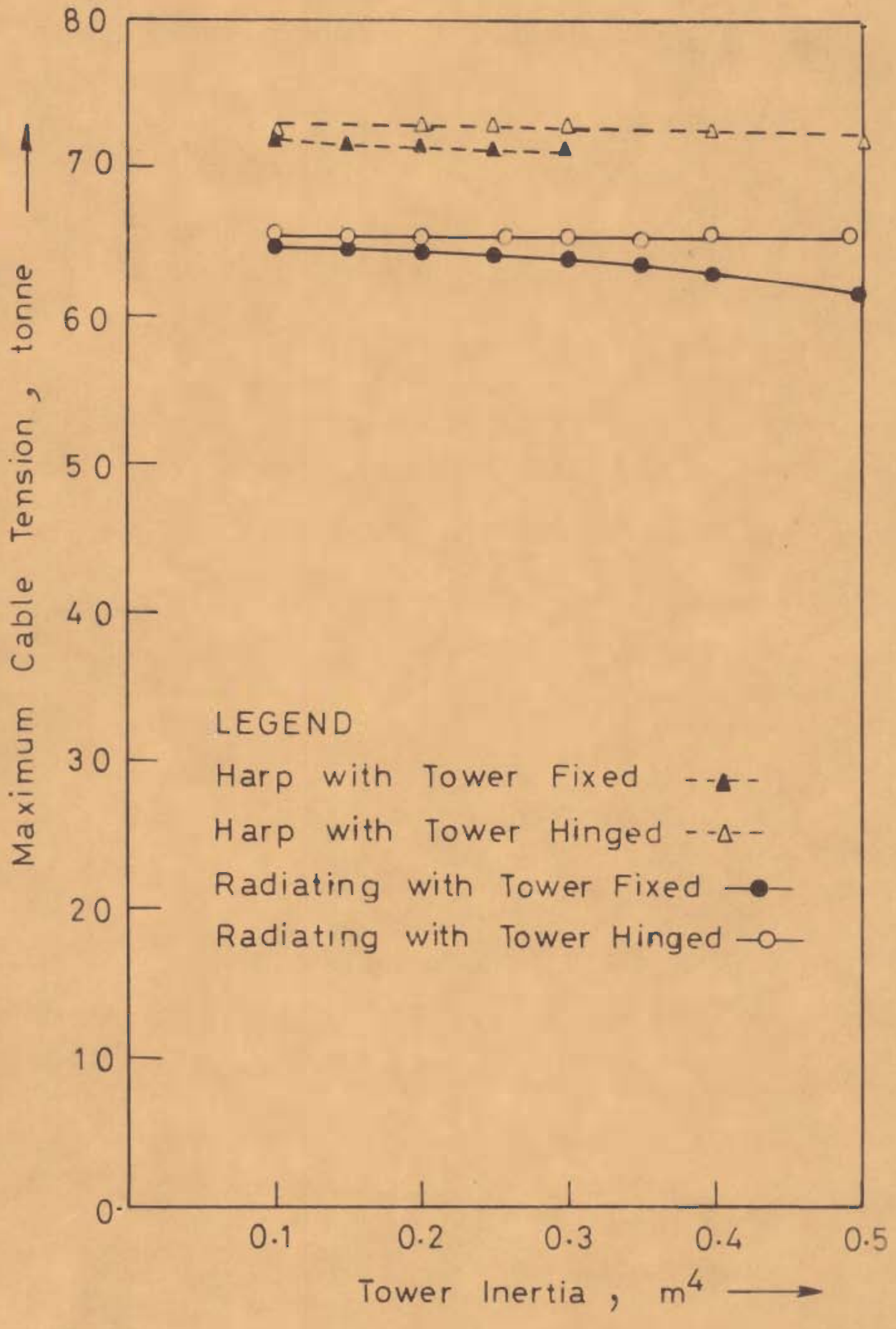


Fig.6.2. Effect of Flexural Rigidity of Tower on Maximum Tension in Cables.

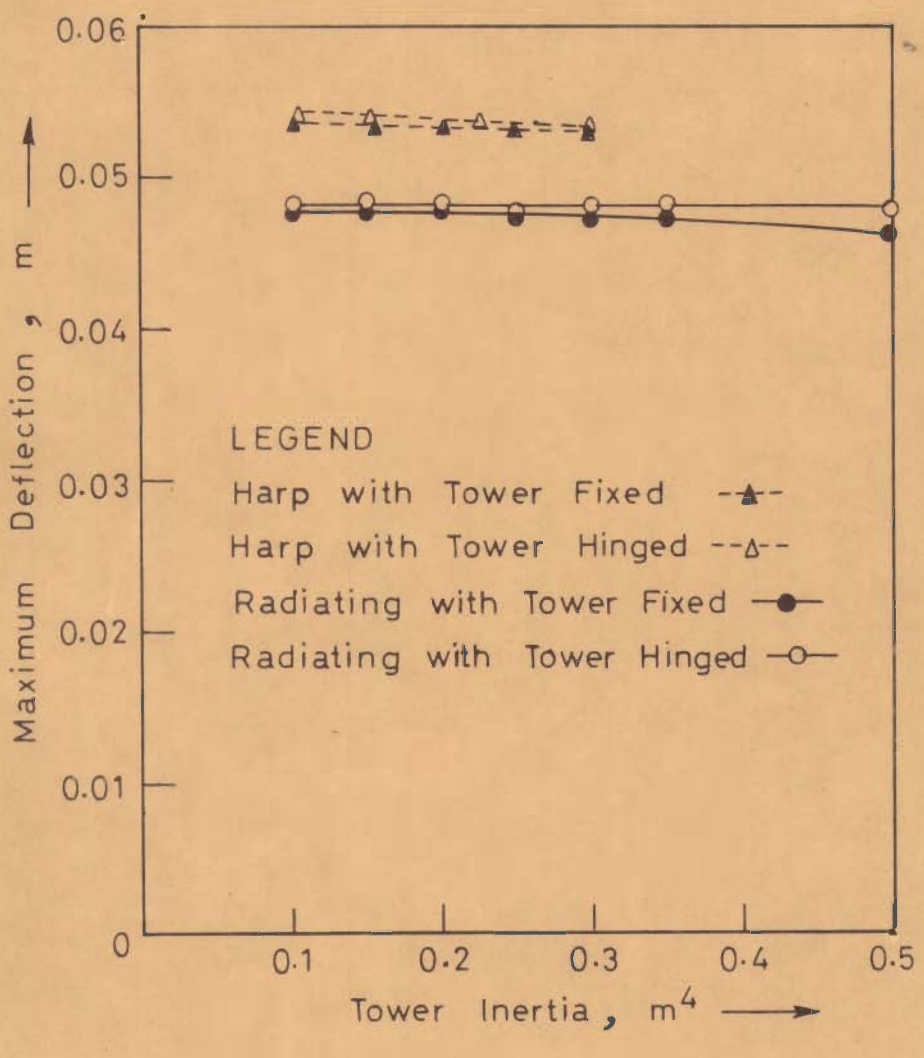


Fig.6.3.Effect of Flexural Rigidity of Tower on Maximum Girder Deflection.

the radiating and the harp arrangements when the tower base condition changes from fixed to hinged. The deflections are larger in case of the harp arrangement by approximately 12%.

The variation of girder moments, both hogging and sagging, as well as the tower moments are shown in Fig.6.4. The variation of moments with tower rigidity is nearly linear. The maximum change in sagging and hogging moments is 0.31% and 0.08% respectively for the radiating arrangement and 0.06% and 0.16% respectively for the harp arrangement. This change in moment has been calculated for a change in tower moment of inertia from 0.1 to 0.3 m^4. The above changes in girder moments indicate that the effect of tower stiffness is negligible. The effect of tower support conditions on girder moments is also small. In the radiating arrangement sagging moments increase and hogging moments decrease slightly while in the harp arrangement both the moments decrease slightly. The maximum decrease in hogging moment is approximately 2.33% for the harp arrangement.

The fixed tower base moments are quite sensitive to tower stiffness, and its value becomes 3.0 and 2.74 times respectively for radiating and harp arrangements when the tower moment of inertia is increased from 0.1 to 0.3 m^4. The fixed tower base moments for the harp arrangement are about 1.6 times those for the radiating arrangement. The maximum tower moment in the harp arrangement with a hinged base is approximately half as compared with the value for the fixed

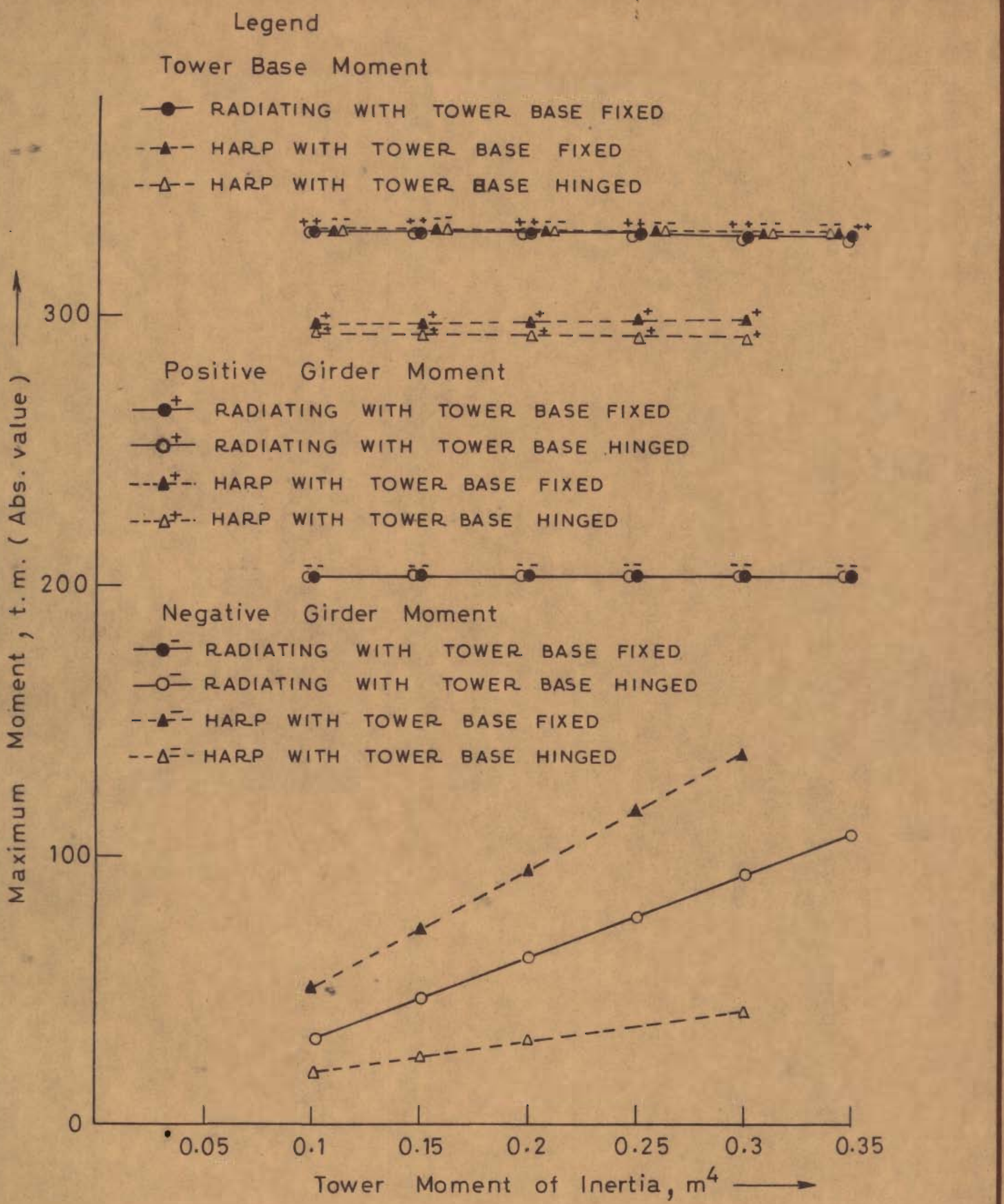


Fig. 6.4. Effect of Flexural Rigidity of Tower on Moments.

base. In radiating arrangement with hinged tower base the tower moments are absent.

The above study shows that the effect of tower stiffness as well as their support condition on cable tension, girder moments and deflections is very small. But the tower base moments in both harp and radiating arrangements increase sharply with the stiffness of the tower. In case of hinged base towers, smaller tower moments occur in harp type and are zero in radiating arrangement.

6.3 EFFECT OF FLEXURAL RIGIDITY OF THE GIRDER

Both the harp and radiating arrangements are considered for studying the effect of girder stiffness also. The basic dimensions and sectional properties of these bridges remain same as shown in Fig.6.1 where except the girder stiffness, all other parameters were kept constant. The values of girder moment of inertia were varied from 0.3 m^4 to 1.0 m^4. Effect of support condition at the tower base i.e. hinged and fixed conditions, was also studied simultaneously.

Figure 6.5 shows the variation of the girder and the tower moments with the girder inertia, and is seen to be nearly linear. In both the harp and the radiating arrangements the girder moments increase sharply with the increase in the flexural rigidity of the girders. However, the tower moments do not change much. Tower base moments are much smaller as compared to the girder moments. When the girder inertia is increased

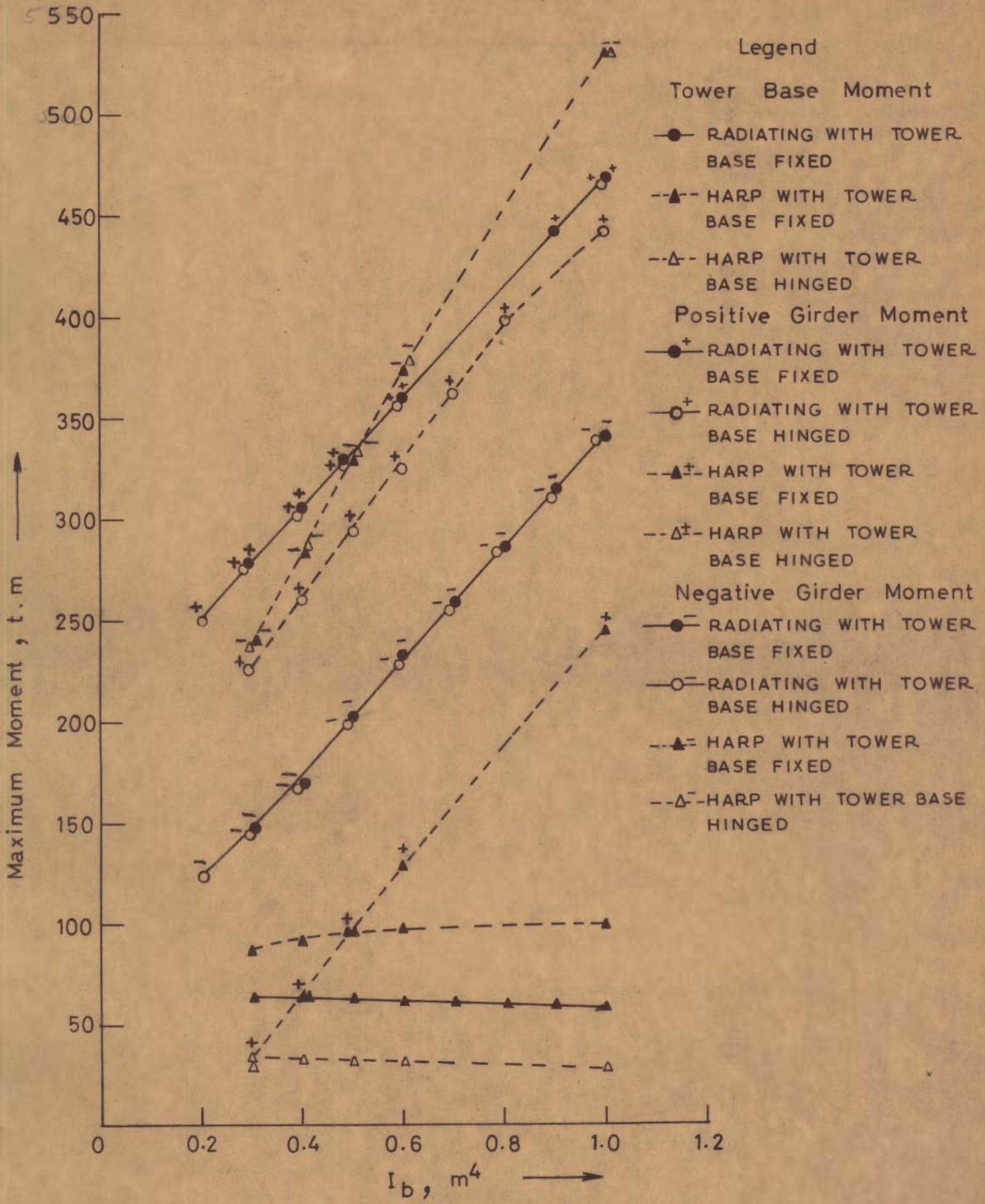


Fig. 6.5 Effect of Girder Moment of Inertia on Moments.

to 1.0 m^4, the negative and positive moments become approximately 2.28 and 3.50 times in the radiating arrangement while in the harp arrangement these moments becomes 2.10 and 7.95 times the values at the inertia of 0.3 m^4. For this increase of girder inertia the tower base moment decreases in the radiating arrangement by 10.6% and increases in the harp arrangement by 13.8% . At the value of girder inertia 1.0 m^4, the negative girder moment and the tower base moment are larger in harp arrangement by 46.8% and 72.4% respectively, while the positive girder moment is smaller by 7.0% than the radiating arrangement. The effect of support conditions at the tower base on the girder moments is small.

Figure 6.6 shows the variation of the maximum tension in the cables with the moment of inertia of the girders. This figure indicates that the cable tension decreases with the increase of the girder inertia. This decrease in the harp and the radiating arrangement is 9.5% and 10.3% respectively when there is increase of girder inertia from 0.3 m^4 to 1.0 m^4. The cable tension is larger in the harp arrangement as compared to the radiating arrangement, the difference being 12.1% when girder inertia is 1.0 m^4.

Figure 6.7 shows the variation of the maximum vertical deflection in the girder with its moment of inertia. The girder deflections are seen to decrease slightly with the increase of the girder inertia. The decrease in the harp and the radiating arrangements is 9.43% and 15.8% respectively

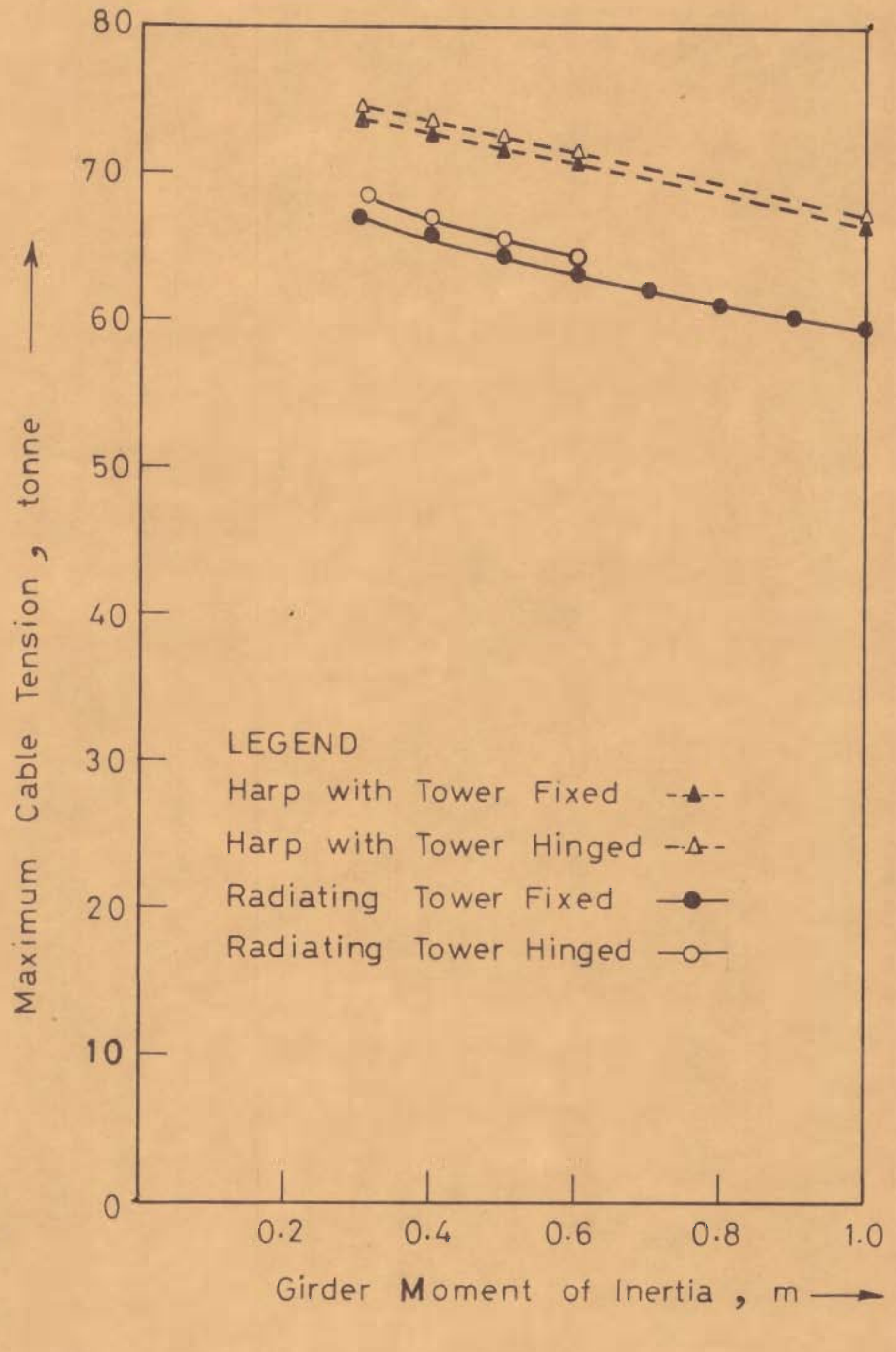


Fig. 6.6. **Effect of Girder Moment of Inertia on Maximum Cable Tension.**

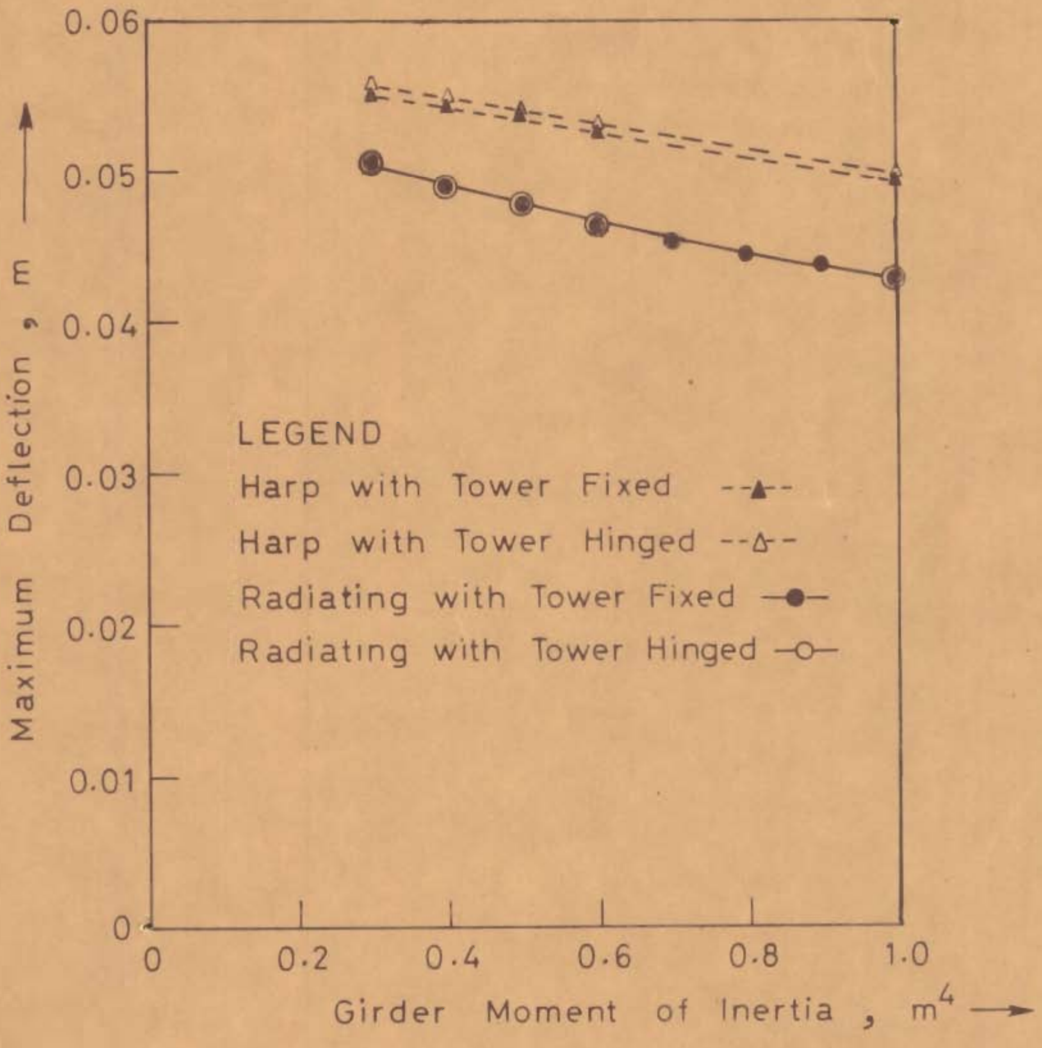


Fig. 6.7. Effect of Girder Moment of Inertia on Maximum Girder Deflection.

for a change in girder inertia from 0.3 to 1.0 m^4. The maximum girder deflection is larger by about 16% in the harp arrangement as compared to that in the radiating arrangement.

6.4 EFFECT OF CABLE STIFFNESS

The effect of cable area on the forces and displacements is studied herein. The basic parameters of the structure considered are same as shown in Fig.6.1 but the cable area is varied from 0.0133 m^2 to 0.150 m^2. The tower base condition in this case is assumed to be fixed.

The variation of cable tension is shown in Fig.6.8. The cable tension increases with the increase in cable stiffness in both radiating and harp arrangements, however, the increase is not large. For a change of cable area from 0.0133 to 0.150 m^2, the increase in cable tension is 17.4 and 14.8% in the radiating and harp arrangements respectively. Cable tension is larger in harp arrangement by 6.2% and 4.1% for cable areas equal to 0.0133 m^2 and 0.150 m^2 respectively.

The variation of girder deflection is shown in Fig.6.9. Initially the girder deflection decreases sharply but tends to stabilise subsequently. In the harp and radiating arrangements the girder deflection reduces to 0.131 and 0.127 times when the cable area is increased from 0.0133 m^2 to 0.15 m^2. The deflections are larger in harp arrangement by 6.9% and 3.8%

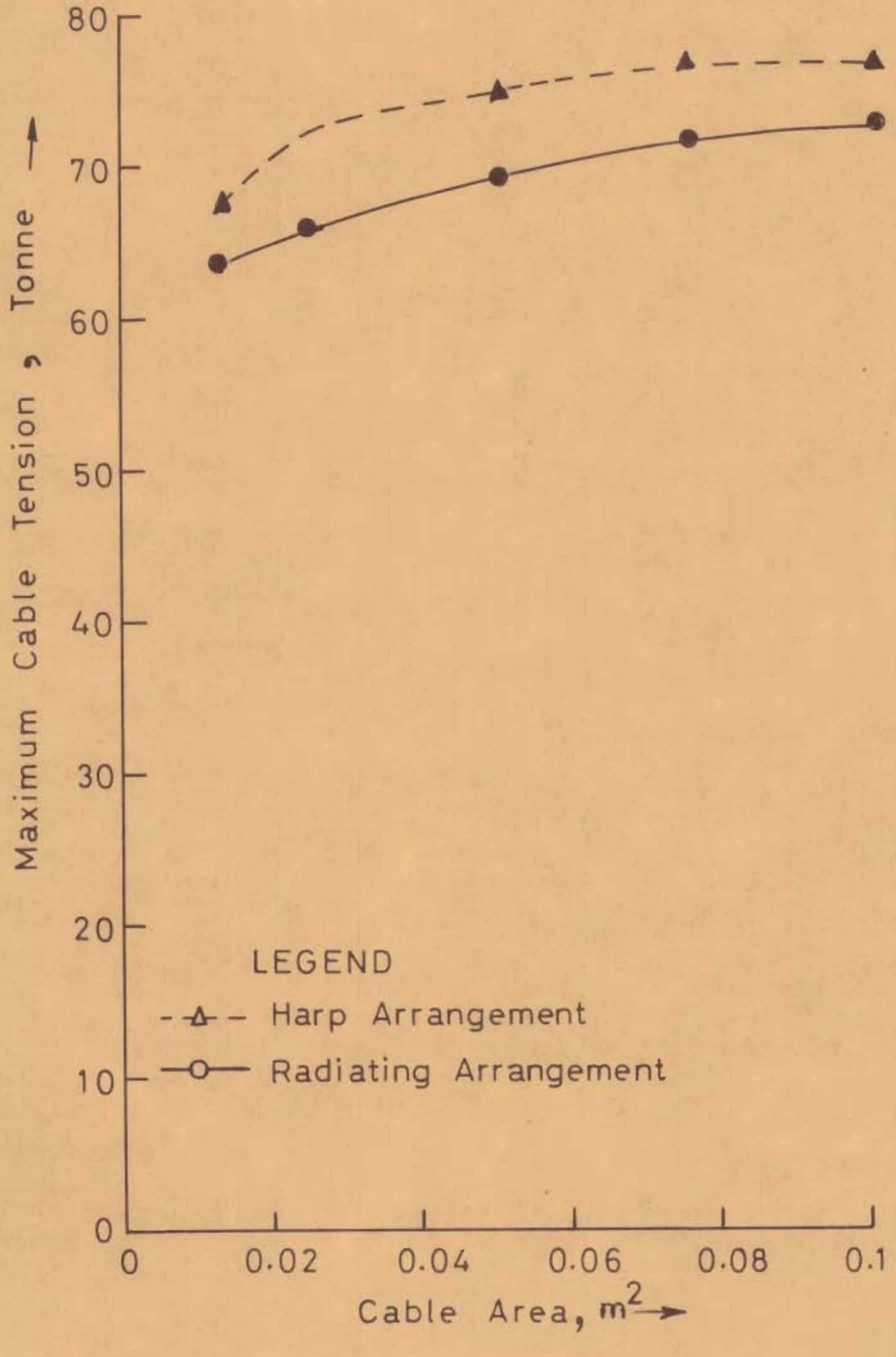


Fig.6.8. Effect of Cable Area on Maximum Tension in Cables.

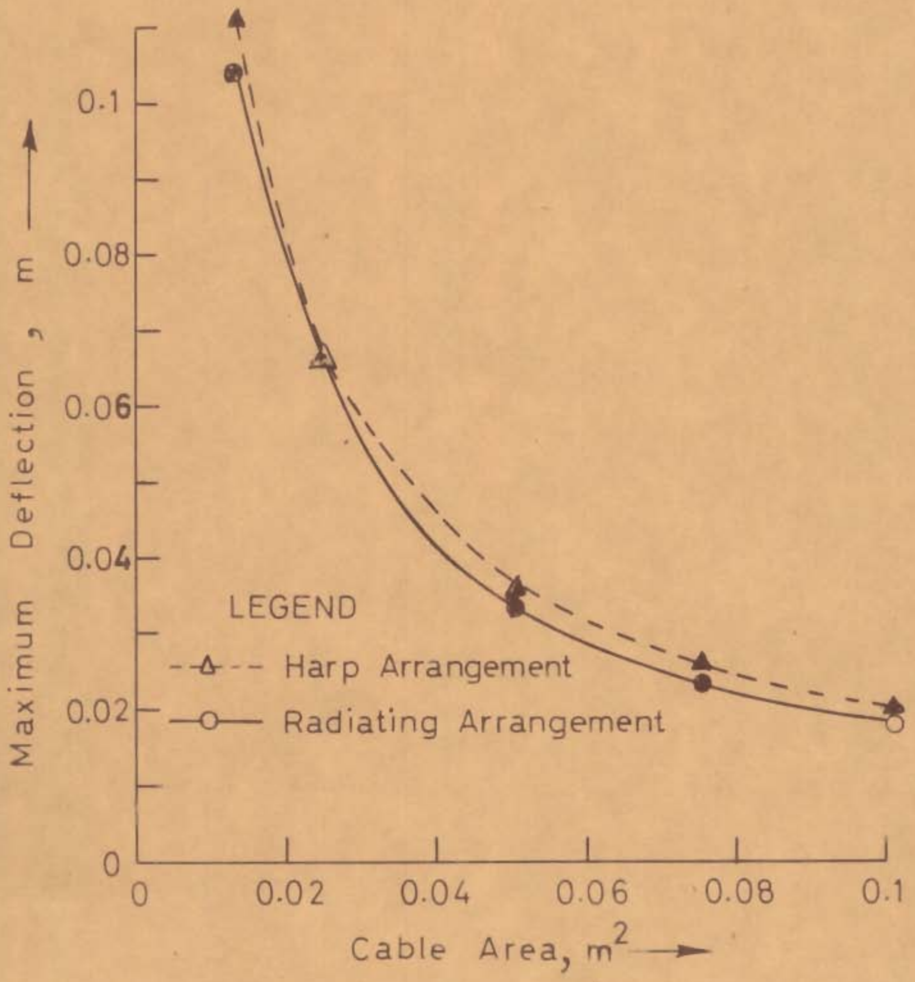


Fig. 6.9. Effect of Cable Area on Maximum Deflection in Girder.

for cable areas equal to 0.0133 and 0.150 m^2 respectively.

The variation of girder and tower moments is shown in Fig.6.10. The positive moments decrease sharply with the increase in cable area. For a change of cable area from 0.0133 m^2 to 0.150 m^2, the hogging and sagging moments reduce to 0.762 and 0.3778 times in the radiating arrangement and 0.313 and 0.145 times in harp arrangement. The tower base moments become 0.126 and 0.208 times in radiating and harp arrangements for the above increase of cable area.

6.5 EFFECT OF LENGTH OF CENTRAL PANEL

The effect of number of cables and the length of the central panel of the main span on the behaviour of the cable-stayed bridge is presented here. The basic dimensions and other parameters are the same as given in Fig.6.1. But only the radiating arrangement with 12,20,28 and 36 cables in one plane is considered. Two cases of central panel length are investigated (Fig.6.11), namely (i) 0.2 times the main span; and (ii) equal to length of other panels. The tower bases are assumed to be fixed. The variation of cable tension vs the number of cables is shown in Fig.6.12 (a) and (b). These figures also show the variation of the forces for both the cases of panel lengths and side to main span ratios Ls/Lm equal to 0.35, 0.40, 0.45 and 0.50.

It is seen that the maximum cable tension decreases

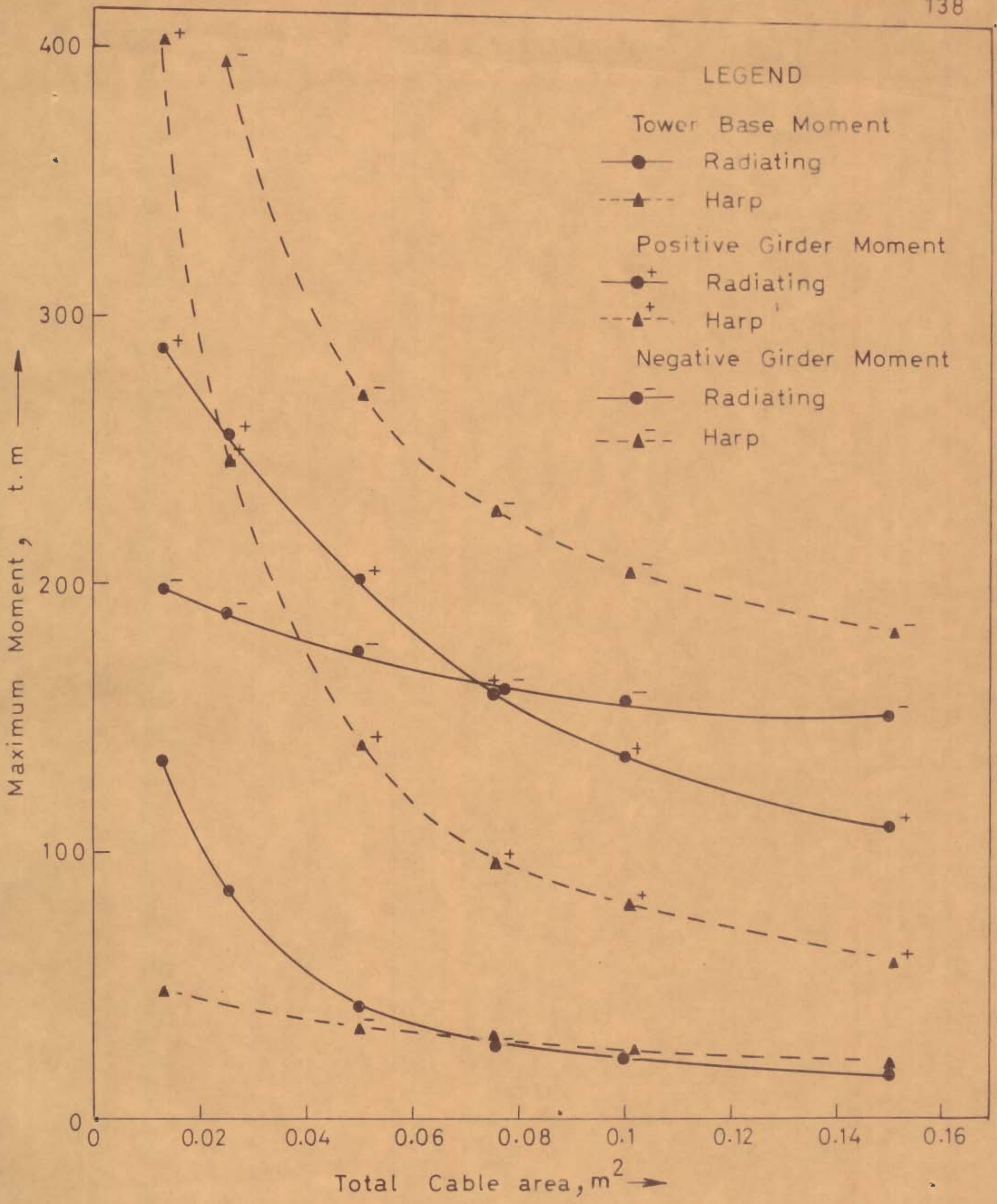


Fig.6.10 Effect of Cable Area on Maximum Girder Moments.

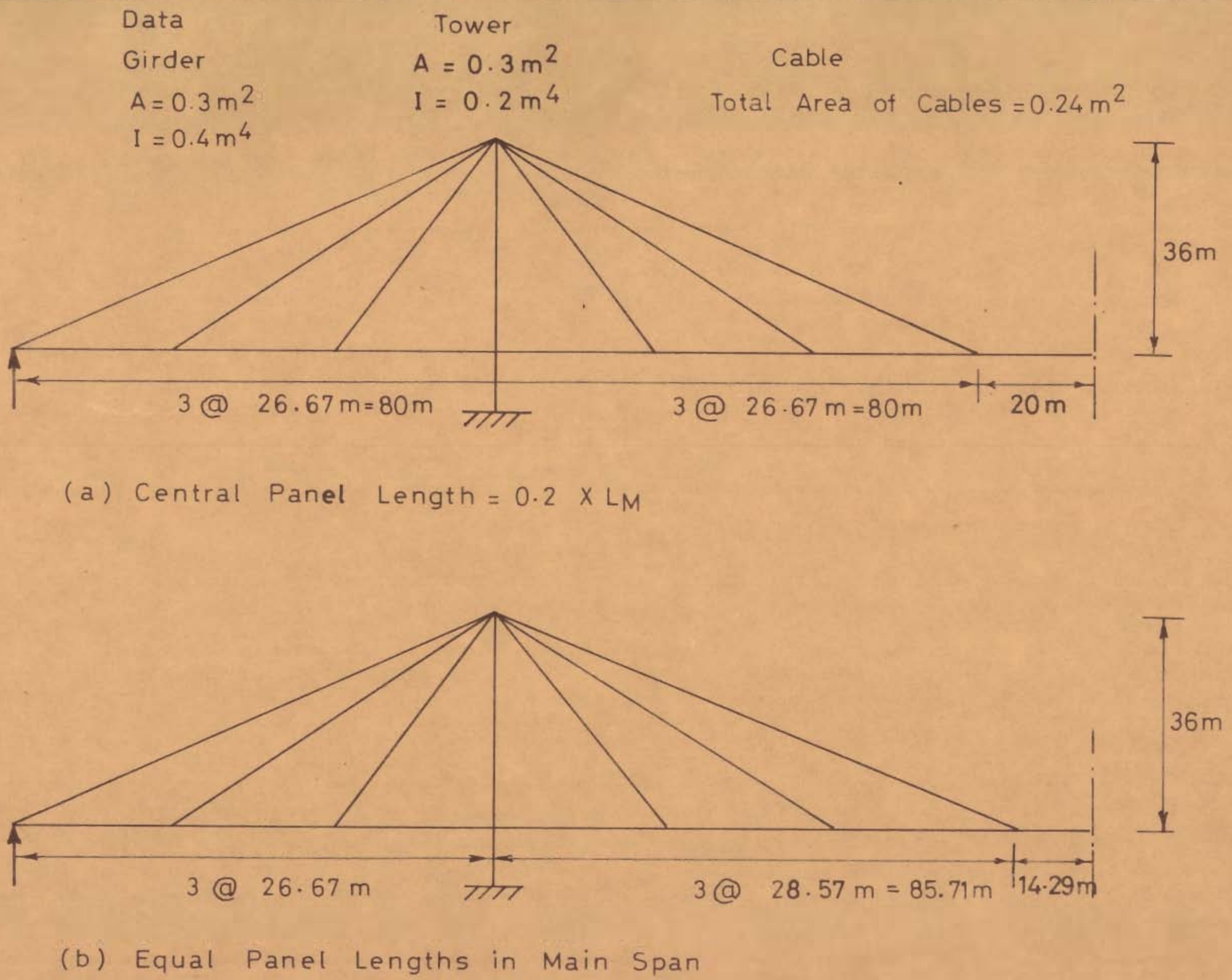


Fig.6.11 Elevation of Bridges Considered for Parametric Study. (Number of Cables = 12)

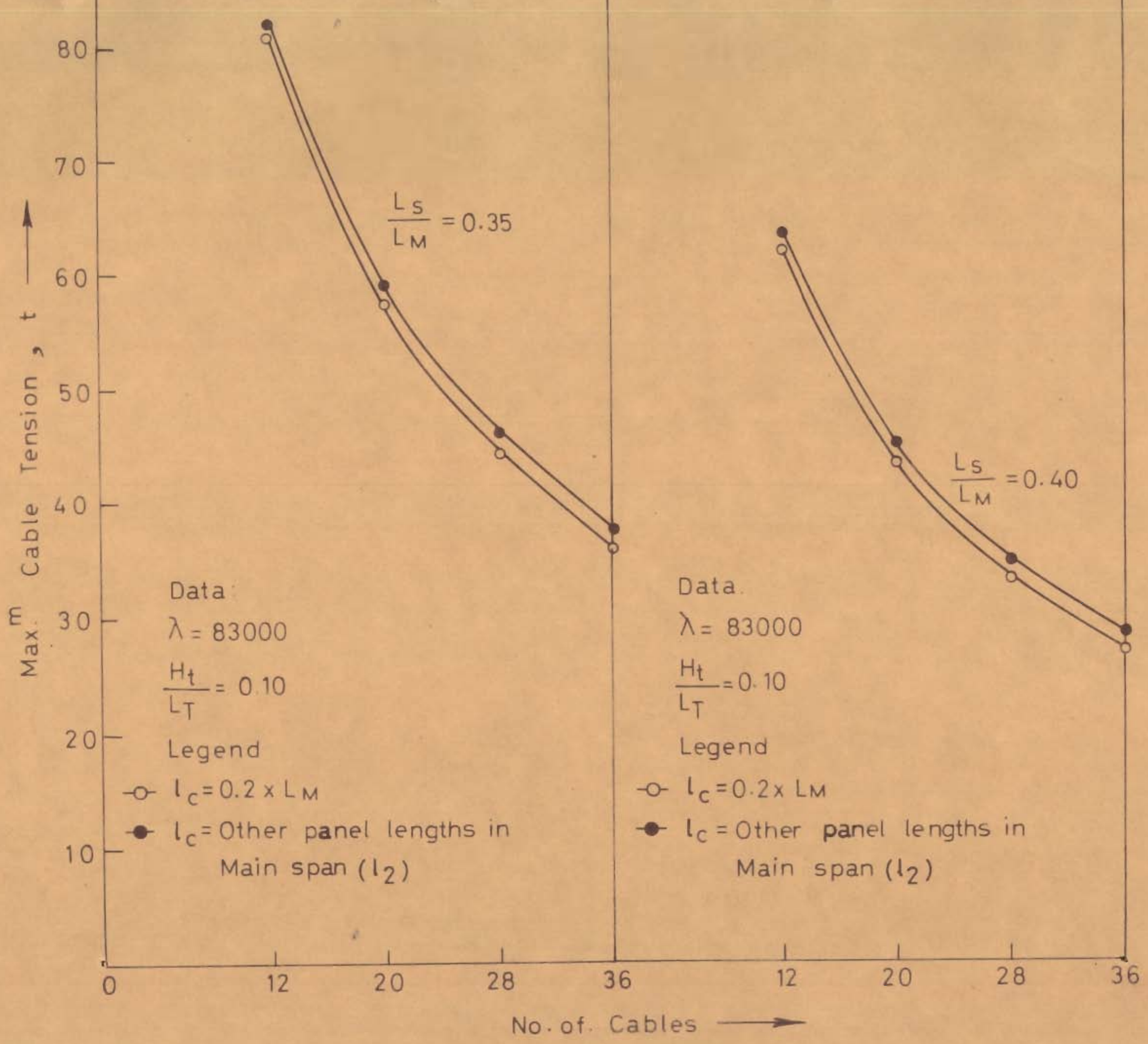


Fig. 6.12 (a) Effect of Number of Cables and Length of Central Panel on Maximum Tension in Cables.

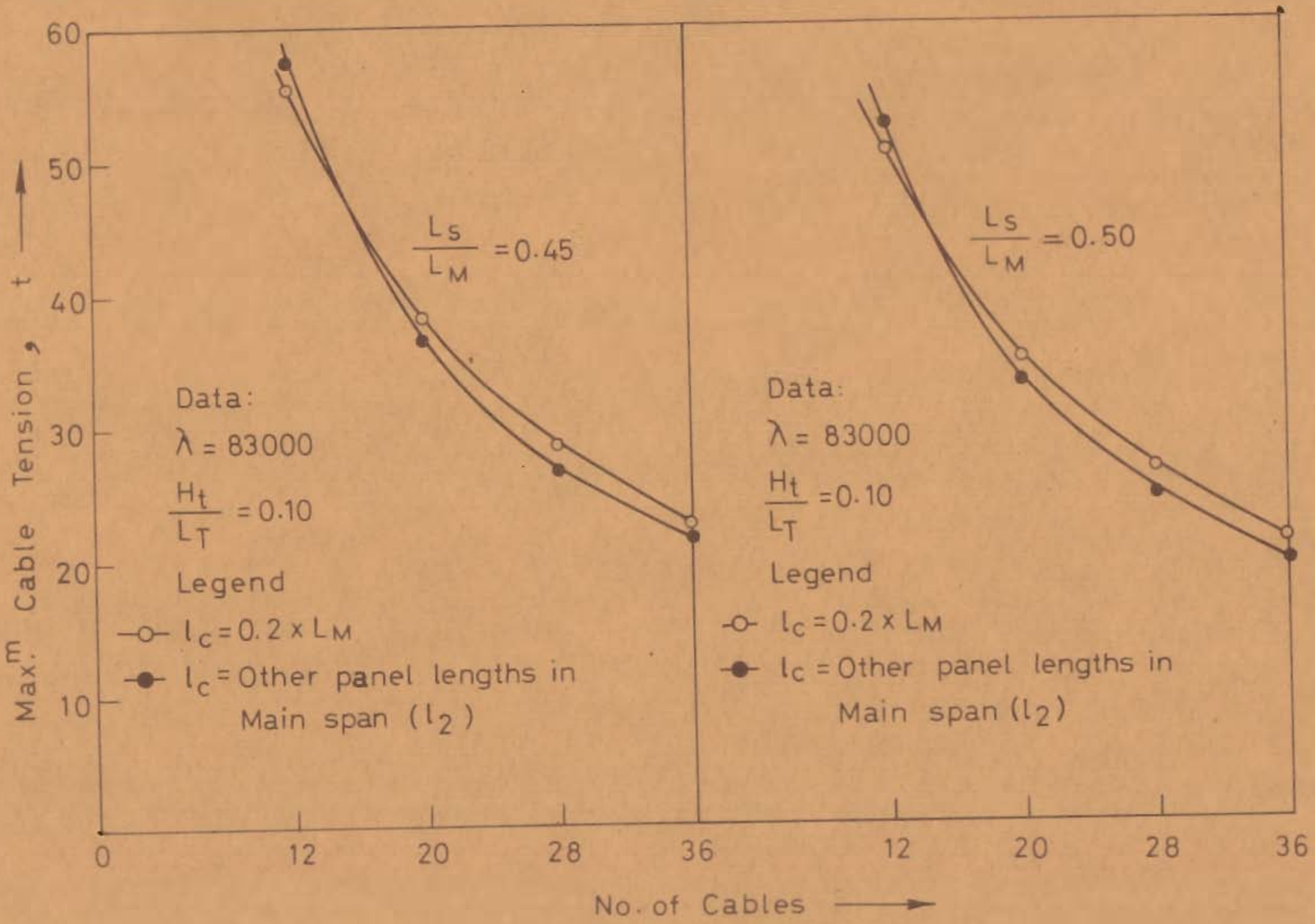


Fig. 6.12 (b) Effect of Number of Cables and Length of Central Panel on Maximum Tension in Cables.

rapidly with the increase in number of cables. In case of the 36 cable system the cable tension reduces to 0.45 and 0.37 times that in the 12 cable system for side span ratios 0.35 and 0.50 respectively. In case of side span ratios 0.35 and 0.40 the cable tension increases with the decrease of length of central panel but for side span ratios 0.45 and 0.50 for smaller number of cables it increases with the decrease in length of central panel, but for larger number of cables the trend reverses. The border line is provided by the sixteen cable system. In general, the difference in cable tension in the two cases of panel length is found to be between 1.8 and 8.5% .

The variation of sagging and hogging moments with number of cables for both cases of panel lengths is shown in Fig.6.13(a) and (b). It is seen that the effect of length of central panel on sagging moment is significant while on hogging moment the effect is not appreciable. In general both hogging and sagging moments increase with the increase in number of cables from 12 to 36. The increase in sagging moment is between 30.88 and 25.23% for side span ratio 0.35 and 0.50 respectively. The increase in hogging moments is between 28.8 and 0.4% . The variation of sagging moment for smaller panel length is linear but in the other cases this variation is non-linear. The hogging moments increase for reduced length of central panel between 2.1 and 8.5%.. The sagging moments decrease between 15.6 and 31.0% with the reduction in the

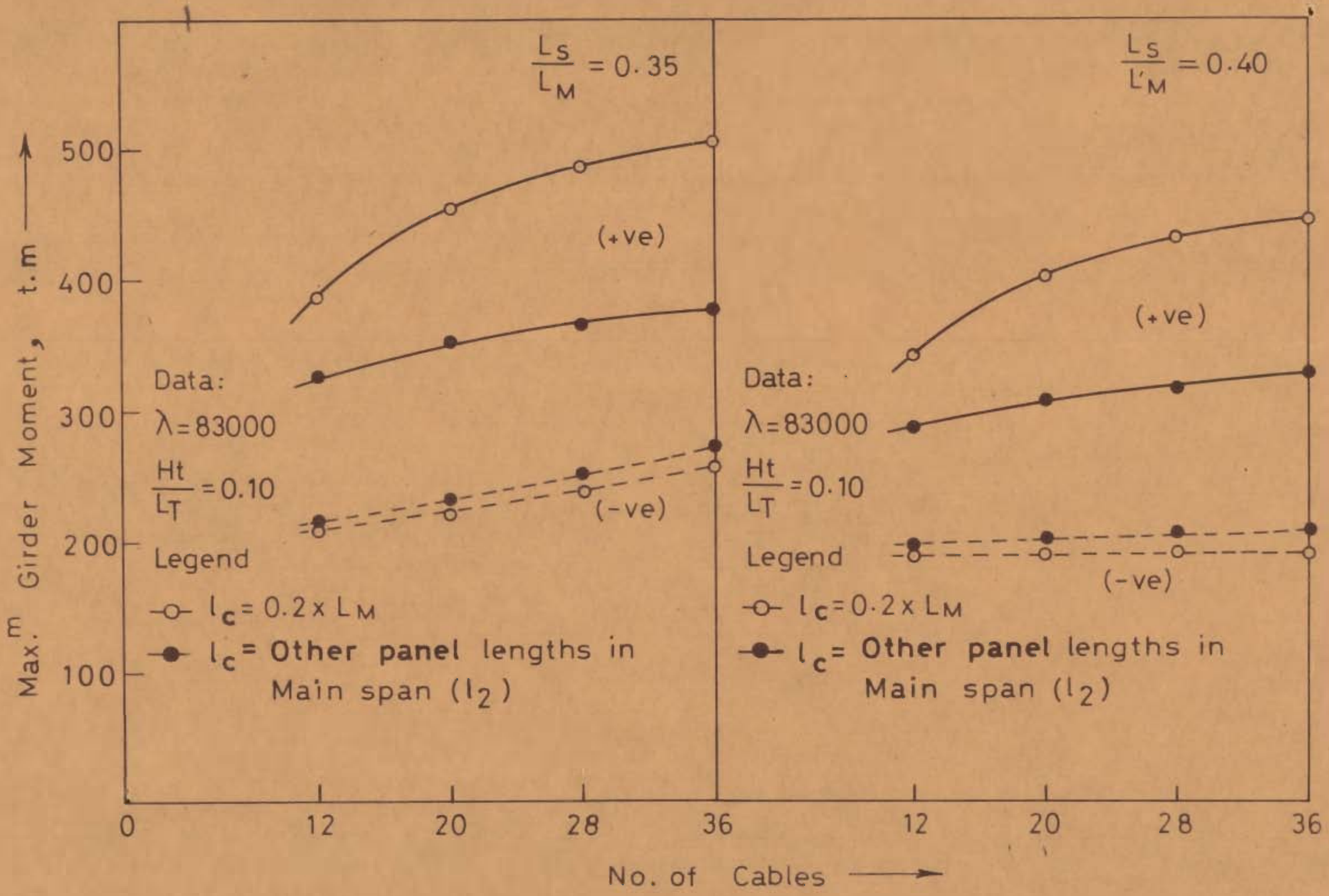


Fig. 6.13 (a) Effect of Number of Cables and Length of Central Panel on Maximum Moment in Longitudinal Girder.

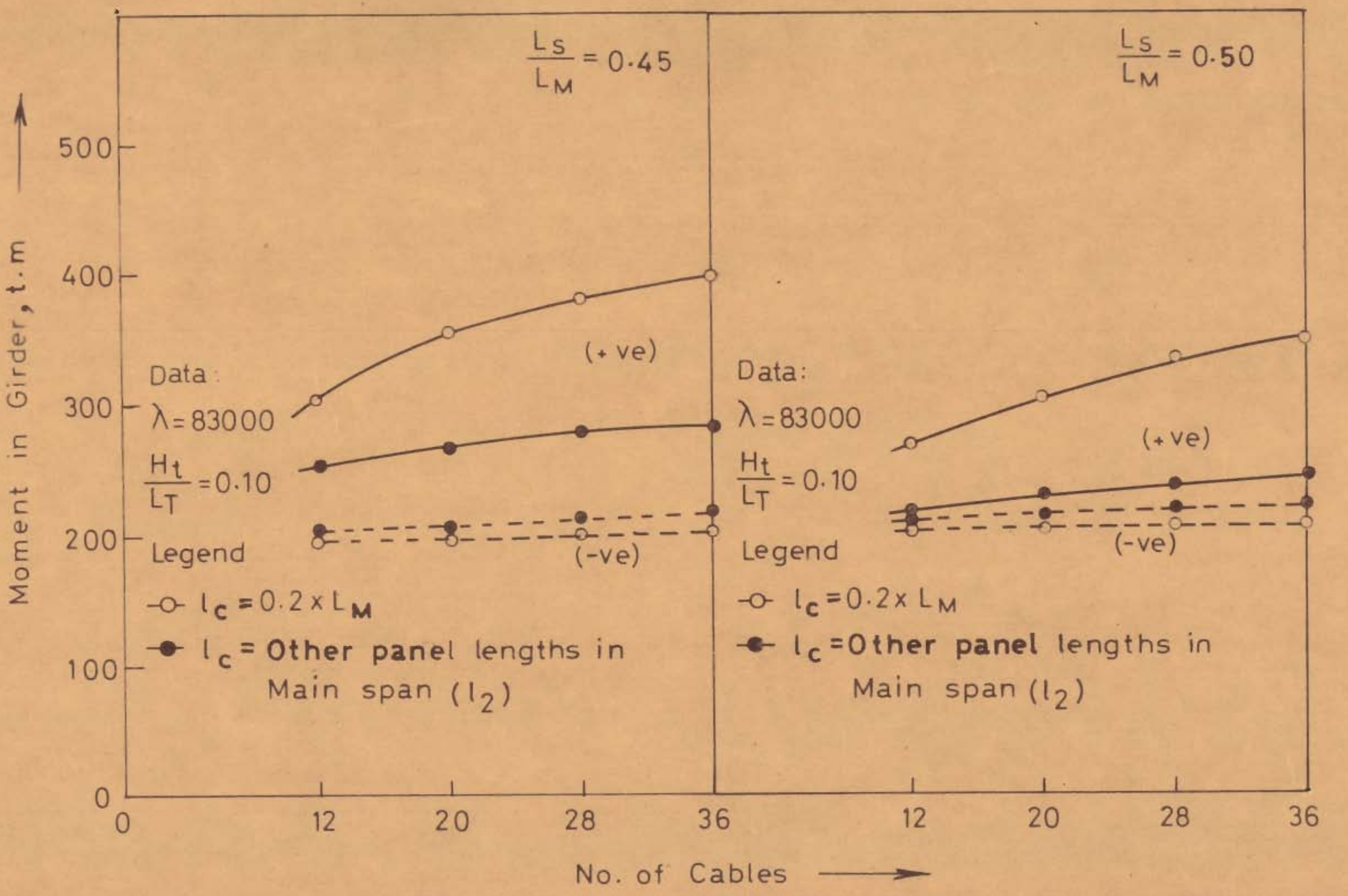


Fig. 6.13 (b) Effect of Number of Cables and Length of Central Panel on Maximum Moment in Longitudinal Girder.

length of central panel. The maximum decrease in sagging moment is in the case of the bridge system with 36 cables and 0.50 side span ratio.

In this study the parameter nxA, n being the number of cables and A being the x-sectional area of each cable is kept constant. The cables away from the tower are more effective in supporting the loads Figs.6.14(a) and (b). The cables with smaller rigidity develop smaller tensions. The above two figures show the cable tensions for bridges with 12,20,28 and 36 cables. The side span to main span ratio is taken as 0.40. These two figures are for the two cases of the central panel lengths. Figures 6.15(a) and (b) show the distribution of the bending moment in the girder for the two cases of the central panel length. These figures also indicate that the girder moments are larger in case of bridges with larger number of cables.

From the above study it may be concluded that the arrangement with equal lengths of all the panels in main span is more economical than the one with longer central panel length.

6.6 EFFECT OF HEIGHT OF TOWER

Three cases of tower height to total span ratios viz., 0.075, 0.100 and 0.125 have been considered. Only the radiating arrangement with the tower base fixed has been analysed. Four cases of side span ratios 0.35, 0.40, 0.45 and 0.50 have been investigated each with 12, 20, 28 and 36 cable system.The

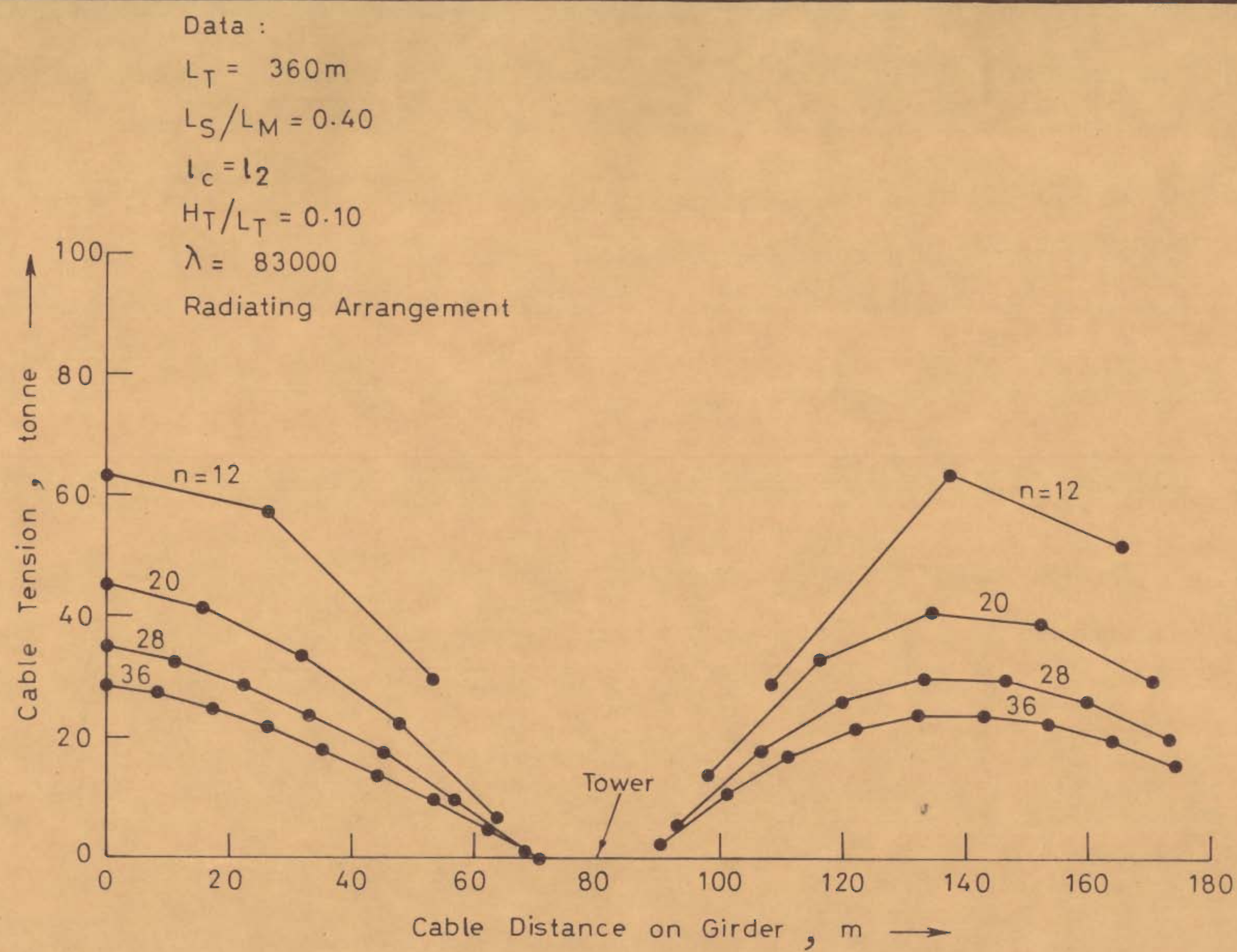


Fig. 6.14(a) Distribution of Tension in Cables ($l_c = l_2$)

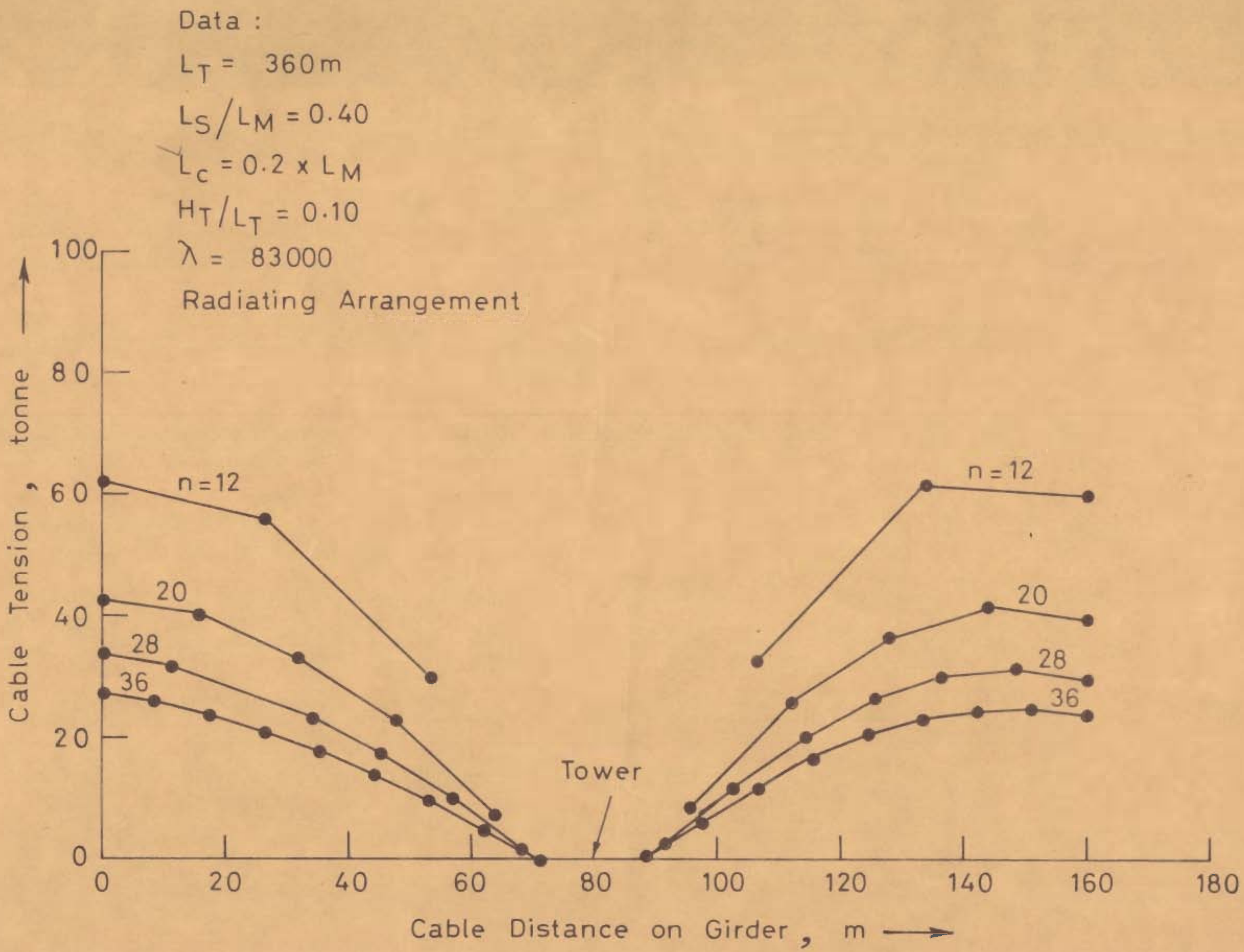


Fig. 6.14(b) Distribution of Tension in Cables ($l_c = 0.2 * L_M$)

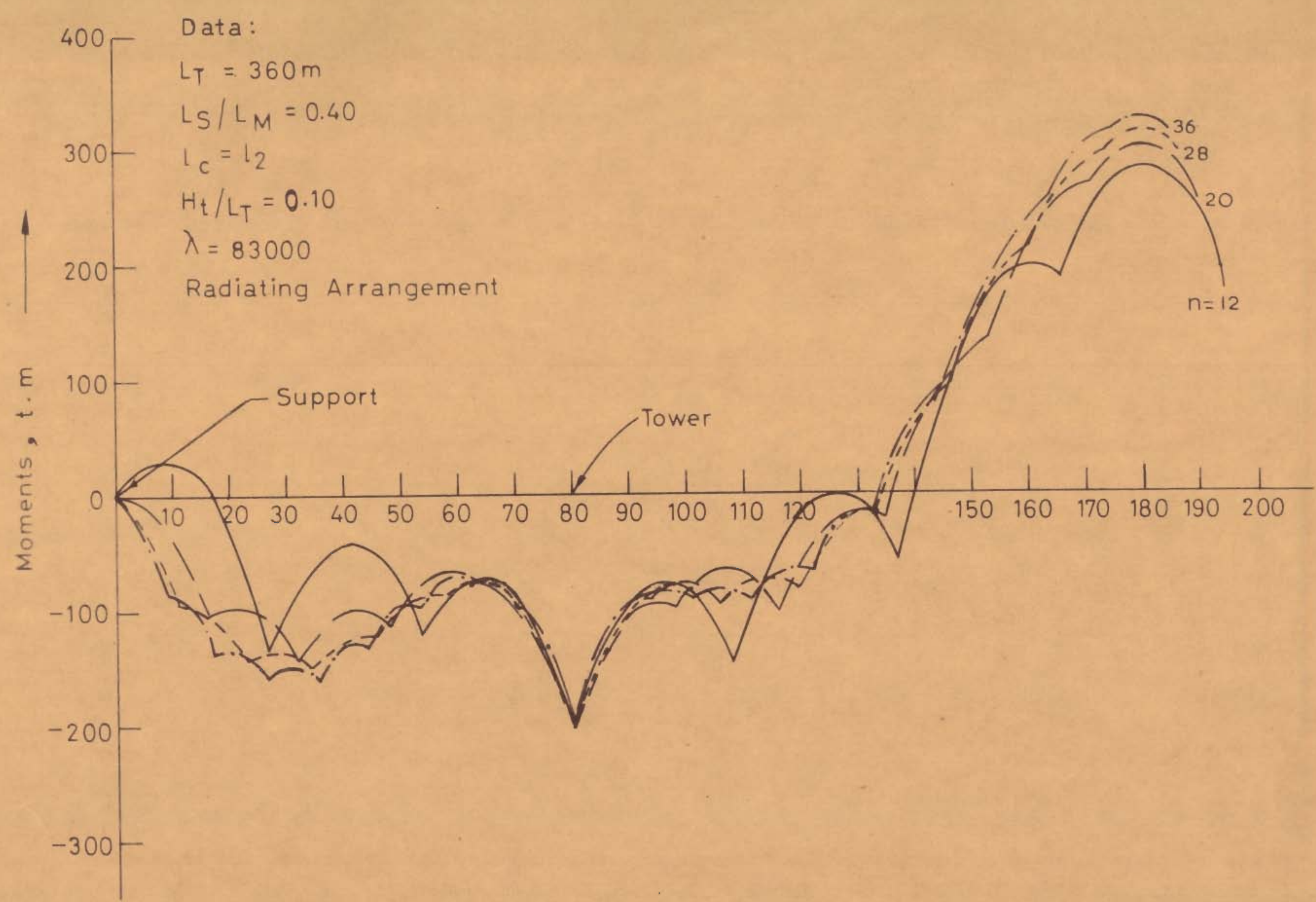


Fig. 6.15 (a) Distribution of Moments in Girder ($l_c = l_2$)

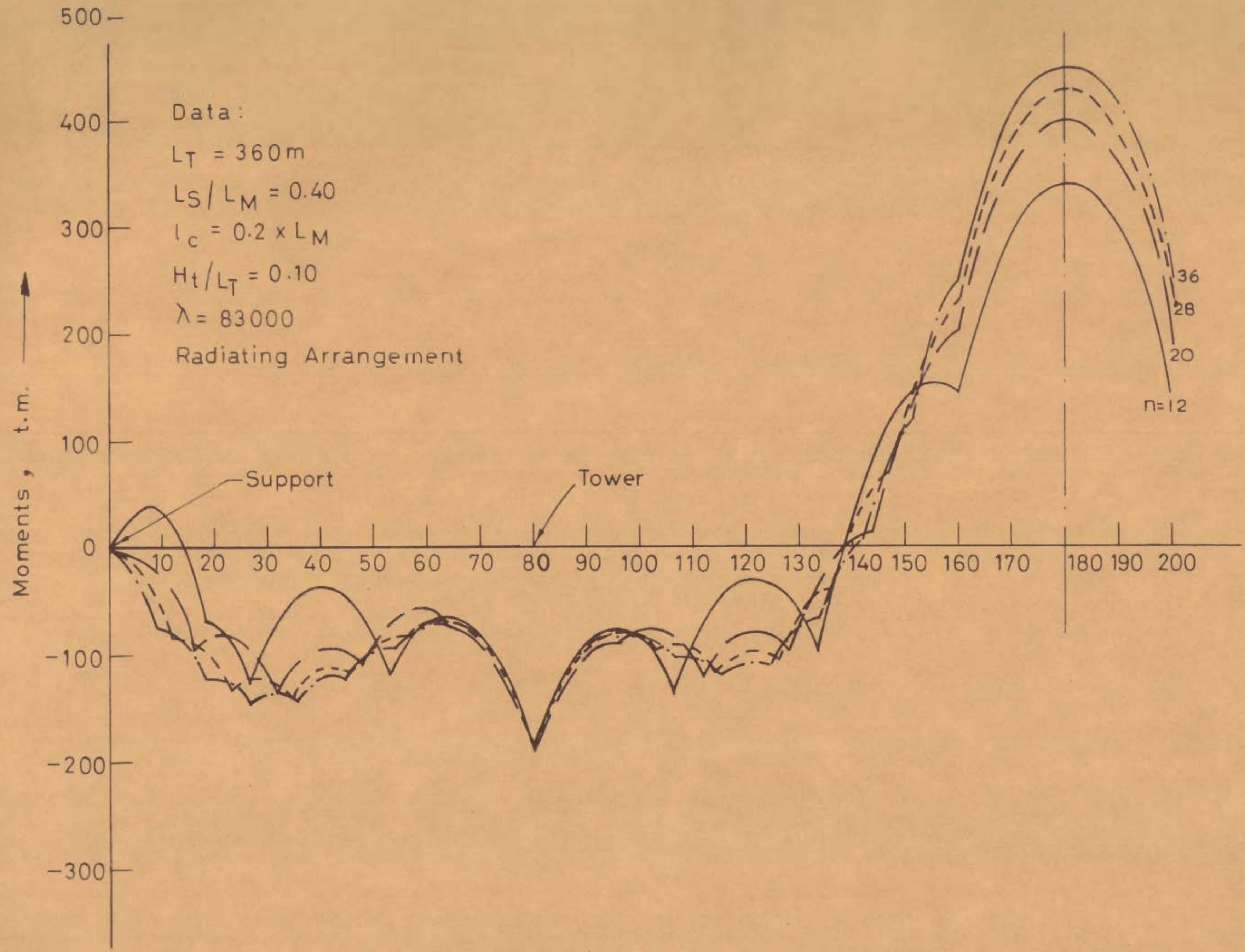


Fig. 6.15(b) Distribution of Moments in Girder ($l_c = 0.2 \times L_M$)

lengths of all the panels in the main span are taken equal. Other dimensions and sectional properties are shown in Fig.6.11.

The variation of cable tension with the tower height ratios is shown in Fig.6.16. This figure shows that the cable tension decreases with increase in the tower height. For example, for a span ratio of 0.40, the cable tension decreases by 26.66, 18.60, 16.36 and 25.48% for the varying number of cables when the tower height ratio is increased from 0.075 to 0.125.

Figure 6.17 shows the variation of girder moments with the tower height ratio for 12, 20, 28 and 36 cables. It is seen that the positive girder moments decrease with the increase in the tower height ratio. The decrease is 39.80, 39.25, 38.63 and 35.03% for 12, 20, 28 and 36 cable systems.

The negative moments decrease in 12 cable systems for all side span ratios. In this case, for a side span ratio 0.40 the decrease is about 7.17% . In case of 20, 28 and 36 cable systems the negative girder moments decrease with the increase in tower height ratio for smaller side spans, while for the larger span these moments increase. For a side span ratio 0.40, this increase in moments is 3.93, 3.11 and 2.32% respectively for 20, 28 and 36 cable systems.

6.7 COST STUDIES:

In this section the economics of harp and radiating arrangements has been compared. The choice is made because

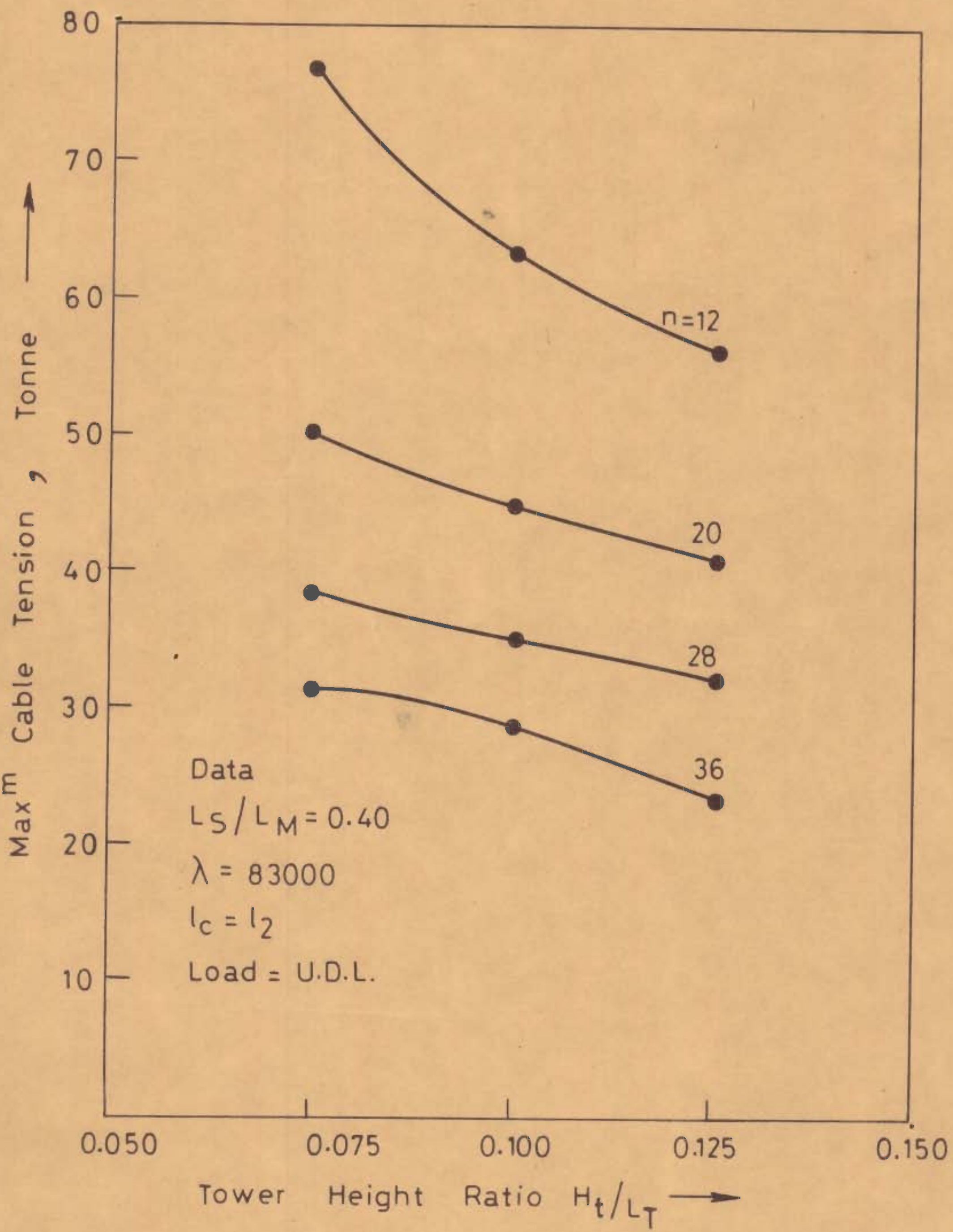


Fig. 6.16 Effect of Tower Height on the Maximum Tension in Cables.

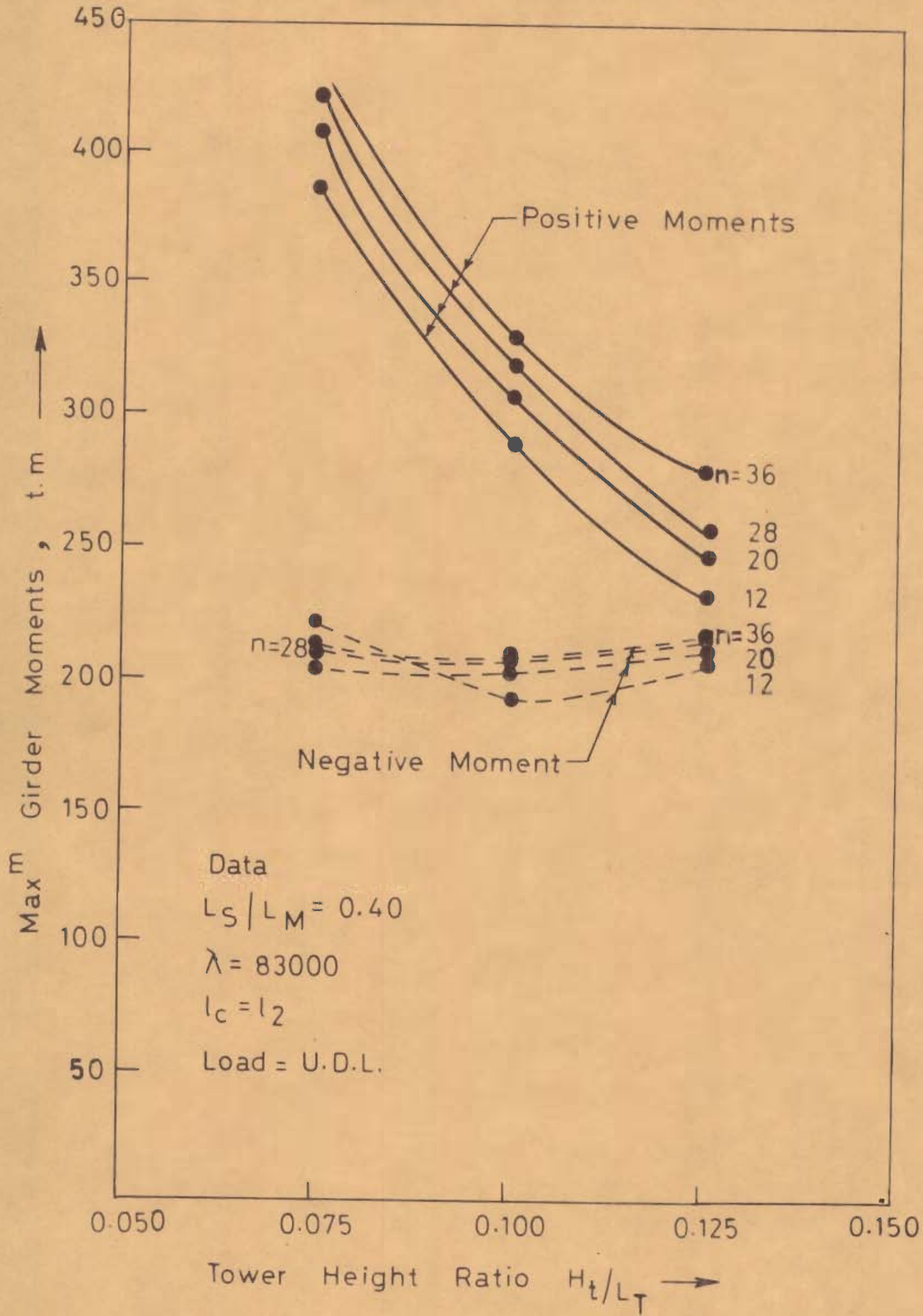


Fig.6.17 Effect of Tower Height on the Maximum Girder Moments.

these are the two commonly employed systems. The studies carried out in this section are mainly divided into two parts. In the first study the effect of the number of cables on the cost of the cable and girder in both harp and radiating bridges has been studied. In the second study the effect of the cable stiffness on the same cost parameters, has been studied. These studies are discussed in detail in arts. 6.7.1 and 6.7.2.

The steel requirement in the girder is evaluated on the basis of the computed moments. At sections, where the rigidity requirement falls below half the maximum value, a section with half of the maximum rigidity is deemed to be provided. It is further assumed that the web contains 1/3 the amount of steel contained in the flanges. The cables are designed on the basis of computed tension. It is assumed that the cable steel costs twice the girder steel.

6.7.1 Effect of Number of Cables on the Cost of Cables and Girders:

The girder moments and the cable forces are influenced by the number of cables in one vertical plane. Consequently it is expected that the cost of cables, girders and their combined cost will be affected by the number of cables. In this section the cost of harp and radiating arrangements with the number of cables in one plane taken as 12, 20, 28 and 36 have been investigated. The load for the above study is taken as 5 t/m over the whole open. The 28-cable system has been studied for the effect of certain other loading cases as well. The detailed discussion of this study is given below.

The separate cost of cables and girders as well as the total costs have been worked out, as described in Table 6.1 and plotted in Fig.6.18. The cost of cables is much smaller as compared to that of the girders. Further, the cost of cables is not influenced much by their numbers. As the number of cables is increased from 12 to 36, the cost of cables increases by 4.95 and 4.54% in the radiating and harp arrangements respectively. The cost of cables is 8.66% higher in the harp arrangement with 36 cables.

In both of the arrangements the cost of the girders as well as the total cost decreases sharply in the case of bridges with smaller number of cables. The decrease in the cost of the girders is 14.93 and 31.14% for radiating and harp arrangements when the number of cables is increased from 12 to 36. Comparable decrease in total cost is 11.26 and 24.95% respectively. The cost of girders and the combined cost of cables and girder are 17.81 and 16.20% higher in the harp arrangement with 12 cables while in case of 36 cables these costs are lower by 4.60 and 1.73% respectively. The combined cost of cables and girder is

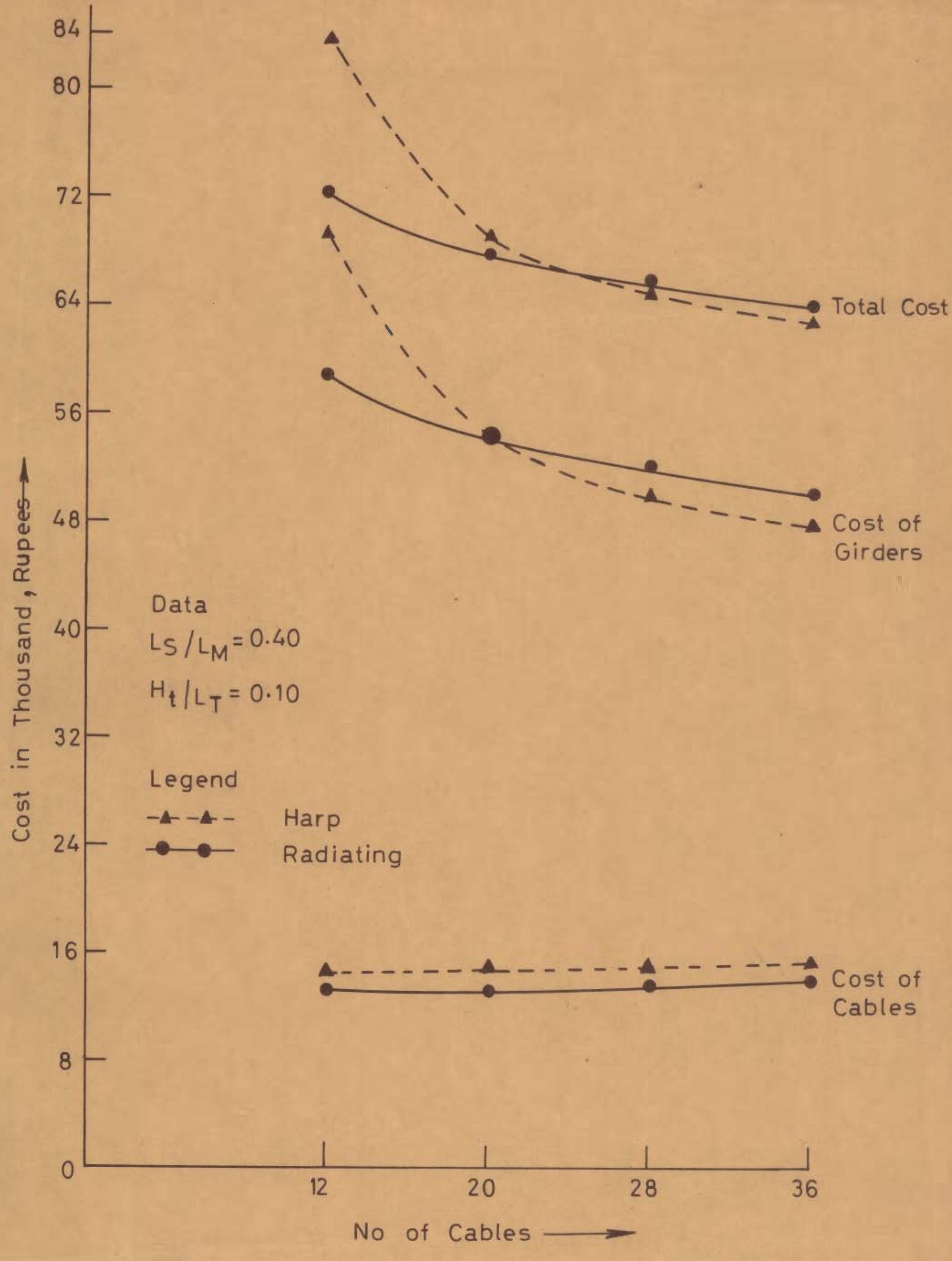


Fig. 6.18. Cost Comparison of Harp and Radiating Arrangement.

approximately equal in both arrangements with 28 cables. Hence, both harp and radiating arrangements with 28 cables are investigated in greater detail in the following study.

Table 6.1

Cost analysis of harp and radiating arrangements
Costs in thousands of Rupees

Number of cables	Radiating Arrangement			Harp Arrangement		
	Cables	Girder	Total	Cables	Girders	Total
12	13.32	58.80	72.12	14.53	69.27	83.80
20	13.41	54.29	67.70	14.93	54.21	69.14
28	13.73	52.01	65.74	15.06	50.09	65.15
36	13.98	50.02	64.00	15.19	47.70	62.89

The second study includes the comparison of costs for different loading conditions but keeping the number of cables fixed as 28. In this case the following six loading cases are considered (Fig.6.19).

a. main span is fully loaded,

b. both side spans are fully loaded,

c. central $L_M/4$ length in mainspan is loaded,

d. main span except the central $L_M/4$ is loaded,

e. complete length of the bridge except the central $L_M/4$ length is loaded, and

f. complete bridge length is loaded.

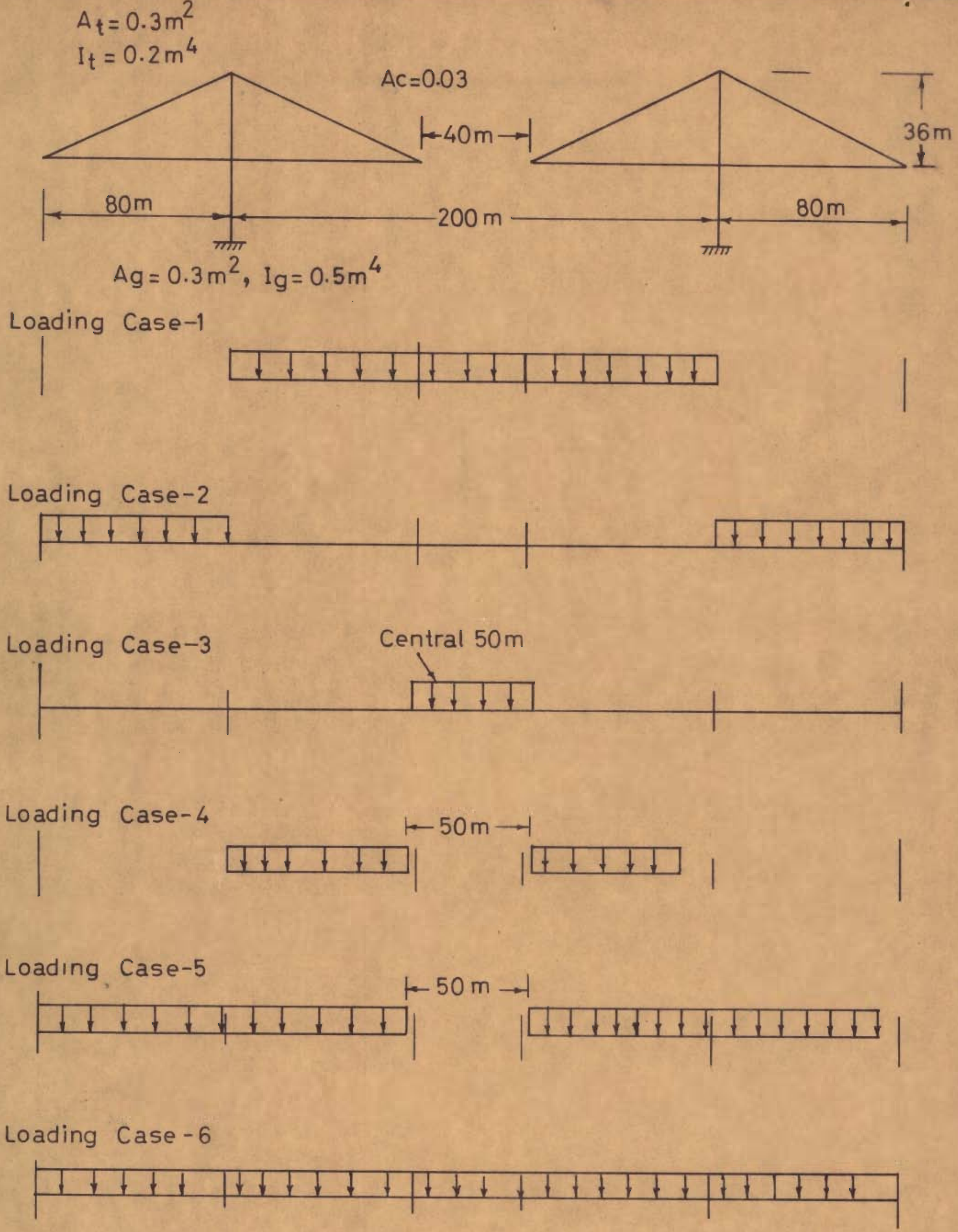


Fig.6.19 Loading Cases Considered for Cost Comparison of Radiating and Harp Arrangements.
(No. of Cables= 28)

The comparison of cost of cables, girders and their combined cost of harp and radiating arrangement is given in table 6.2. This study indicates that the cost is higher in case of uniformly distributed load over the main span in both of the above arrangements. From a comparison of this case with the uniformly distributed load over the whole span following results are obtained. The cost of cables, girders and their combined cost are 11.51, 35.03 and 30.12%. higher in radiating arrangement. In harp arrangement the cost of cables is lower by 12.68%. while the cost of girders and the combined cost of cables and girder are higher by 63.40 and 45.90%. respectively. For this loading case the cost of cables is smaller by 14.11%. in harp arrangement while the cost of girder and the combined cost of cables and girder is larger by 16.53 and 11.05%..

From the first study of the comparison of cost of the radiating and harp arrangements the following conclusions may be drawn:

a. In both the arrangements the cost of cables does not change appreciably with the increase of the number of cables.

b. In both the arrangements the cost of girder is much larger as compared to that of the cables.

c. The cost of the girder and the combined cost of cables and girder decrease sharply with the increase

Table-6.2

Comparison of Cost in Radiating and Harp Arrangements with 28 cables

(costs in thousands of rupees)

Loading case No*	Radiating Arrangement			Harp Arrangement			Remark
	Cables	Girder	Total	Cables	Girder	Total	
a	15.31	70.23	85.54	13.15	81.84	94.99	
b	5.06	44.06	49.12	6.74	71.23	77.97	
c	6.73	48.40	55.13	5.04	39.22	44.26	
d	9.42	37.68	47.10	9.42	63.74	73.16	
e	7.53	30.81	38.34	8.81	40.20	49.01	
f	13.73	52.01	65.74	15.06	50.09	65.15	

*

a. main span is fully loaded

b. both side spans are loaded

c. main $L_M/4$ is loaded

d. main span except the central $L_M/4$ is loaded

e. whole span except the central $L_M/4$ is loaded

f. whole span is loaded.

of the number of cables in both radiating and harp arrangements and it is almost equal in the two cases with 28 cables.

d. The cost of cables, girders and their combined cost of cables and girder is smaller in radiating arrangement as compared to harp arrangement.

In the second study, it may be concluded that the combined cost of cables and girder is maximum in both harp and radiating arrangements when only the main span is fully loaded. For this loading case the combined cost of cables and girder is higher in the harp arrangements as compared to the radiating arrangements.

6.7.2 Effect of the Cable Stiffness on the Cable and Girder Cost

The cost of cable stayed bridges could vary substantially with variation of the cable stiffness or flexural rigidity of girders.The girder moments vary linearly with variation of girder stiffness hence the cost will also vary proportionately. The girder moments in these bridges decrease sharply with the increase of the cable rigidity, and it is reasonable to expect that the cost of the girder will decrease with the increase of the cable cross section. The cost of cables will however increase with the increase in their size. It is therefore of interest to study the combined cost of material employed in the cables and the girders as the cable size is varied. The details of the structures analysed and the variation studied for this purpose are given in Art.6.4. The method of deducing the cost of the cable and the girder material on the basis of forces and moments obtained has been described already, in Art.6.7. The resulting influence on costs is discussed below.

Figure 6.20 shows the variation of cost of cables, girders and the total cost due to variation in cable size.

(i) The cost of cable material used varies linearly with change in cable cross section.

(ii) For the radiating arrangement the cost of cables changes by 7.8% as the area is increased from 0.0133 m^2 to 0.15 m^2 per cable.

(iii) The corresponding change in the harp arrangement is 20.3% .

(iv) The cost of cables in harp arrangement is higher by 1% at a cable area 0.0133 m^2 and 12.5% at 0.15 m^2 as compared to that in the radiating arrangement.

(v) The cost of girder and the combined cost of cables and girder decrease sharply with the increase of cable area.

(vi) For radiating arrangement the cost of girder and the combined cost of cables and girder at an area

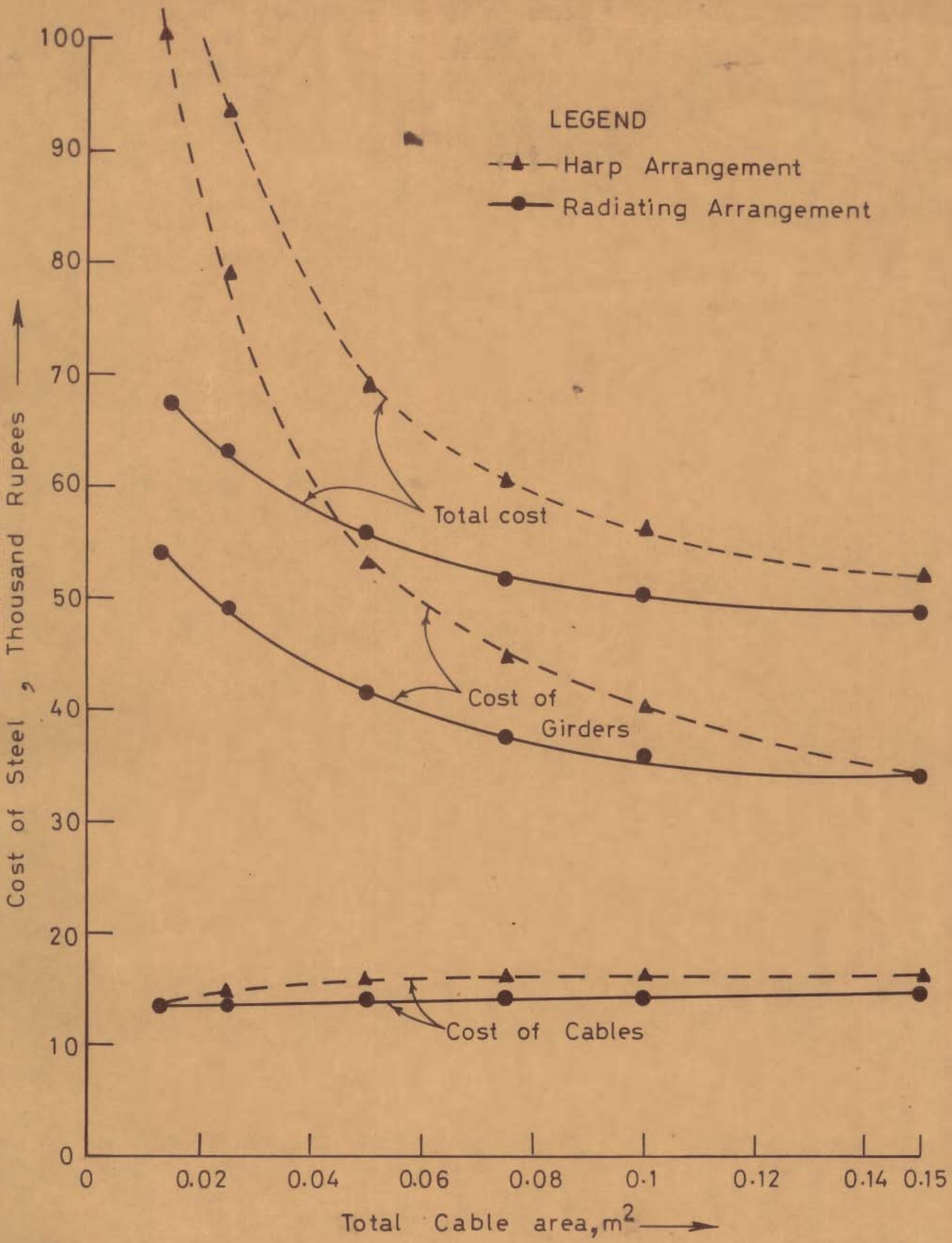


Fig. 6.20 Effect of Cable Area on Cost of Cable Stayed Bridge.

of 0.15 m^2 per cable are reduced to 0.63 and 0.72 per cable. In the harp arrangement these cost become 0.30 and 0.40 times.

(vii) In the harp arrangement the cost of girder and the combined cost of girder and the cables are higher by 118% and 95% as compared to those in radiating arrangement at an area 0.0133 m^2 per cable. The differences in these costs at an area 0.15 m^2 per cable are 5.3% and 7.4% respectively.

(viii) The combined cost of cables and girder does not change appreciably when the cable area is increased beyond 0.10 m^2 in the radiating arrangement and 0.15 m^2 in the harp arrangement.

6.8 PREPARATION OF DESIGN CURVES

The effect of various parameters on the structural behaviour of cable-stayed bridges has been considered individually in Arts. 6.2 to 6.7. In this section the combined effect of several parameters on girder moments and cable tensions has been studied. The results have been presented in the form of design curves. These curves will help the designer in making preliminary design estimates and in deciding the various parameters. Values of girder moments and cable tensions are presented in a dimensionless form.

Results are only given for the radiating type of arrangement which has been shown as being the most economical in Art6.7.

Overall dimensions and other parameters are the same as stated in Art.6.5. An attempt has been made to keep the various parameters considered in this study, within the practical range of values. The parameters and their values are given below:

1. Number of cables n = 12,20,28 and 36
2. Side span to main span ratio = L_s/L_M = 0.35,0.40, 0.45 and 0.50.
3. Tower height to total span ratio h_t/L_T = 0.075,0.100, 0.125.
4. Length of central panel in main span $-\ell_c = 0.2\ L_M$ and $\ell_c = \ell_2$.
5. Stiffness parameter

$$\lambda = \frac{E_c A_c L_T^2}{E_G I_G} = 62,000,\text{ and } 83,000.$$

where, L_s = side span, L_M = main span, L_T = total span,

ℓ_c = length of central panel in main span,

ℓ_2 = length of one panel in main span,

h_t = height of the tower,

E_c and E_G = modulus of elasticity of cables and girders respectively.

A_c = total area of cables and

I_G = girder moment of inertia.

The variation of maximum cable tension, and positive and negative girder moments has been evaluated in the following dimensionless form.

$$\text{Maximum cable tension, } T = \frac{\text{Maximum cable tension}}{wL_T}$$

$$\text{Maximum positive girder moment, } M_p = \frac{\text{Maximum positive girder moment}}{wL_T^2}$$

$$\text{Maximum negative girder moment, } M_n = \frac{\text{Maximum negative girder moment}}{wL_T^2}$$

where w = intensity of the uniformly distributed load over the whole span.

The variation of cable tension with side span to main span ratio is plotted in Figs.6.21(a) to 6.21(f), and the variation of girder moments, both positive and negative are plotted in Fig.6.22(a) to 6.22 (f). Data is available in these figures for all the parameter combinations stated above. These curves can be used very readily for arriving at preliminary dimensions of cable-stayed bridges and carrying out a quick cost analysis so as to decide the configuration for final checking.

Figure 6.22 indicates that the negative girder moments decrease sharply when the side to main span ratio increases from 0.35 to 0.40. For further increase in this ratio, the moments do not change appreciably. This effect is more pronounced in bridges with small tower heights and large number of cables. It is worth noting that the calculations for moments are done for side to main span ratios 0.35, 0.40, 0.45 and 0.50. If intermediate values between 0.35 and 0.40 were to be computed, the moment curves may be expected to smoothen out.

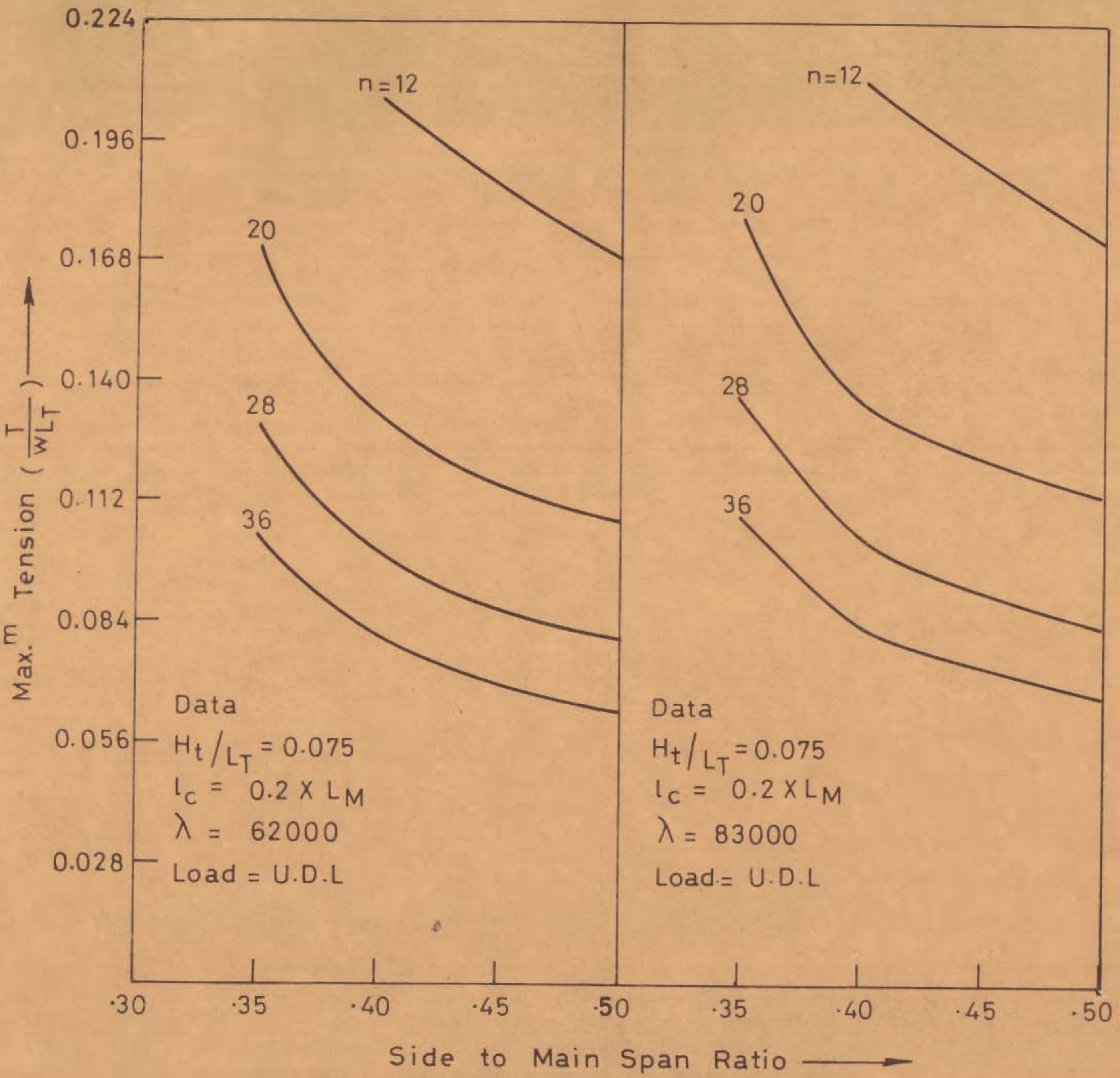


Fig. 6.21(a) Cable Tension

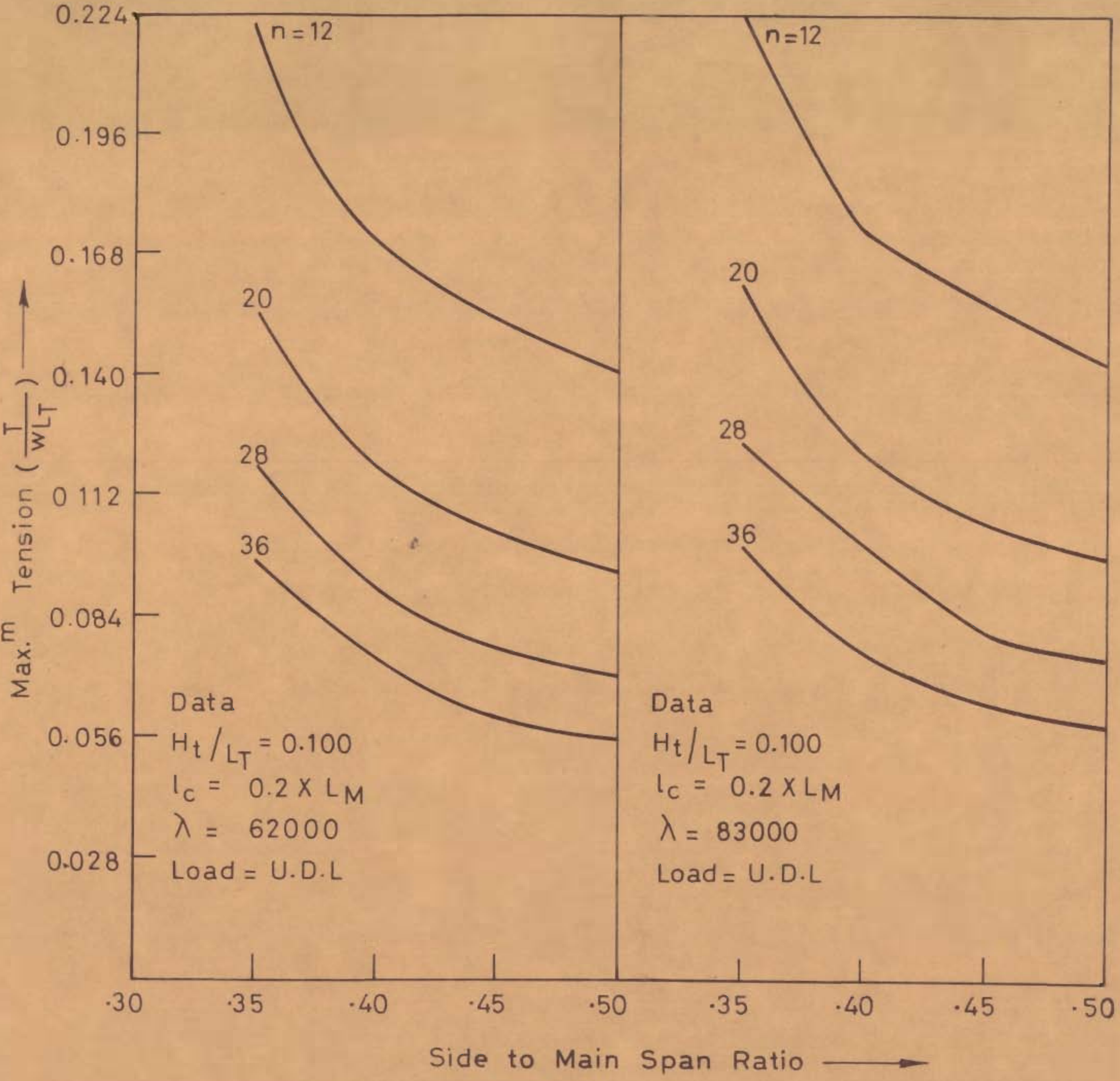


Fig. 6.21 (b) Cable Tension

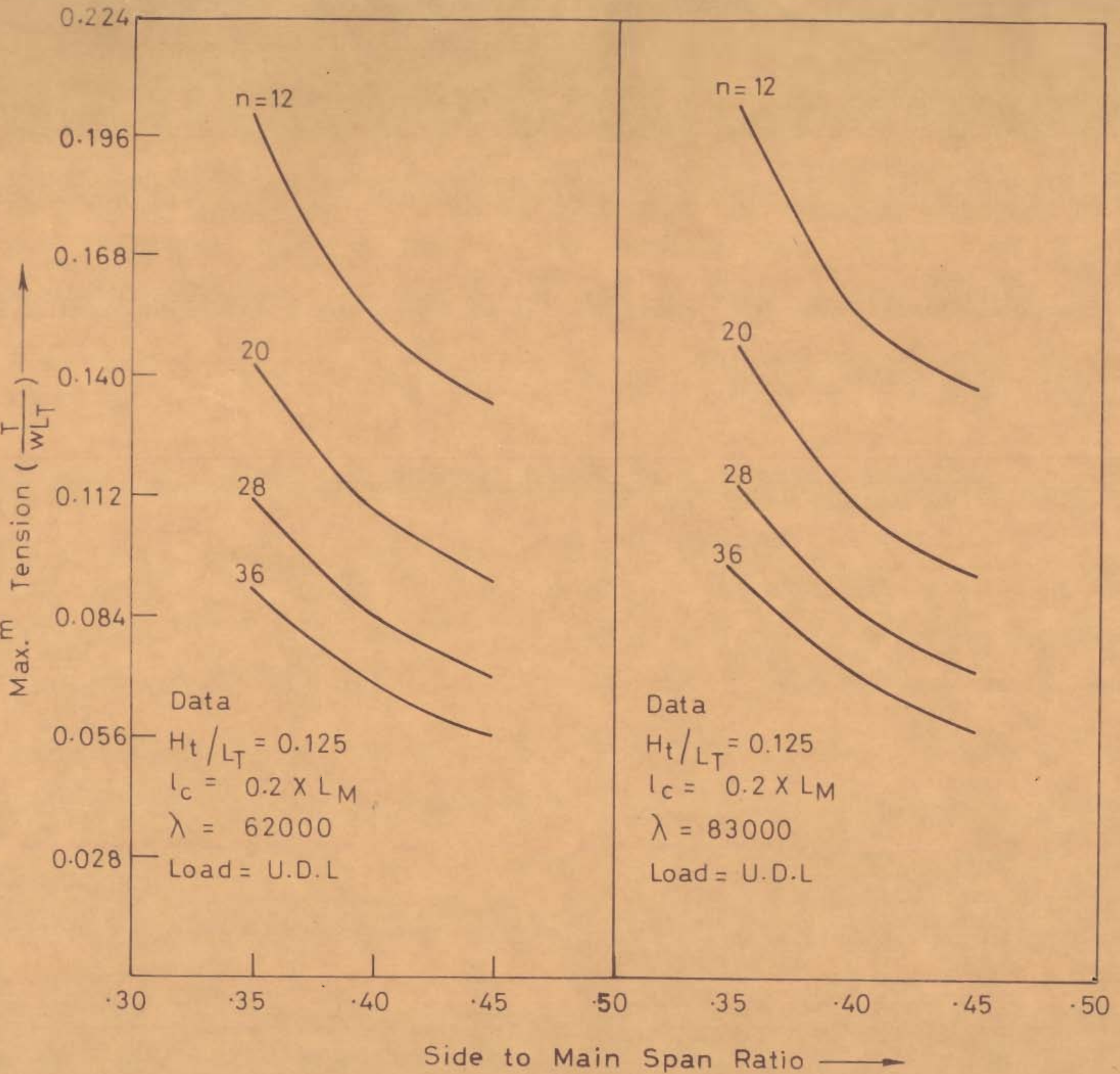


Fig. 6.21(c) Cable Tension

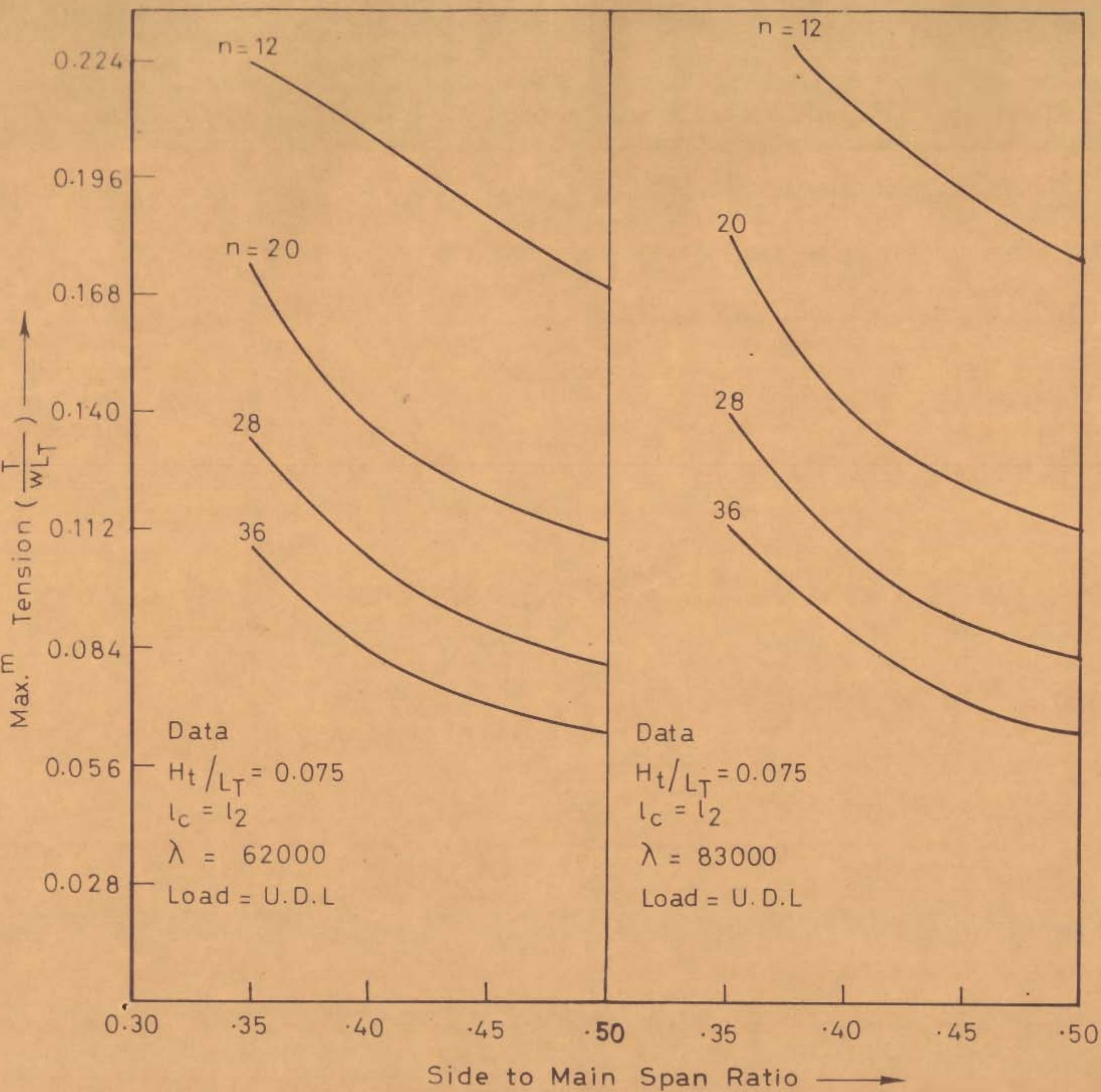


Fig. 6.21(d) Cable Tension

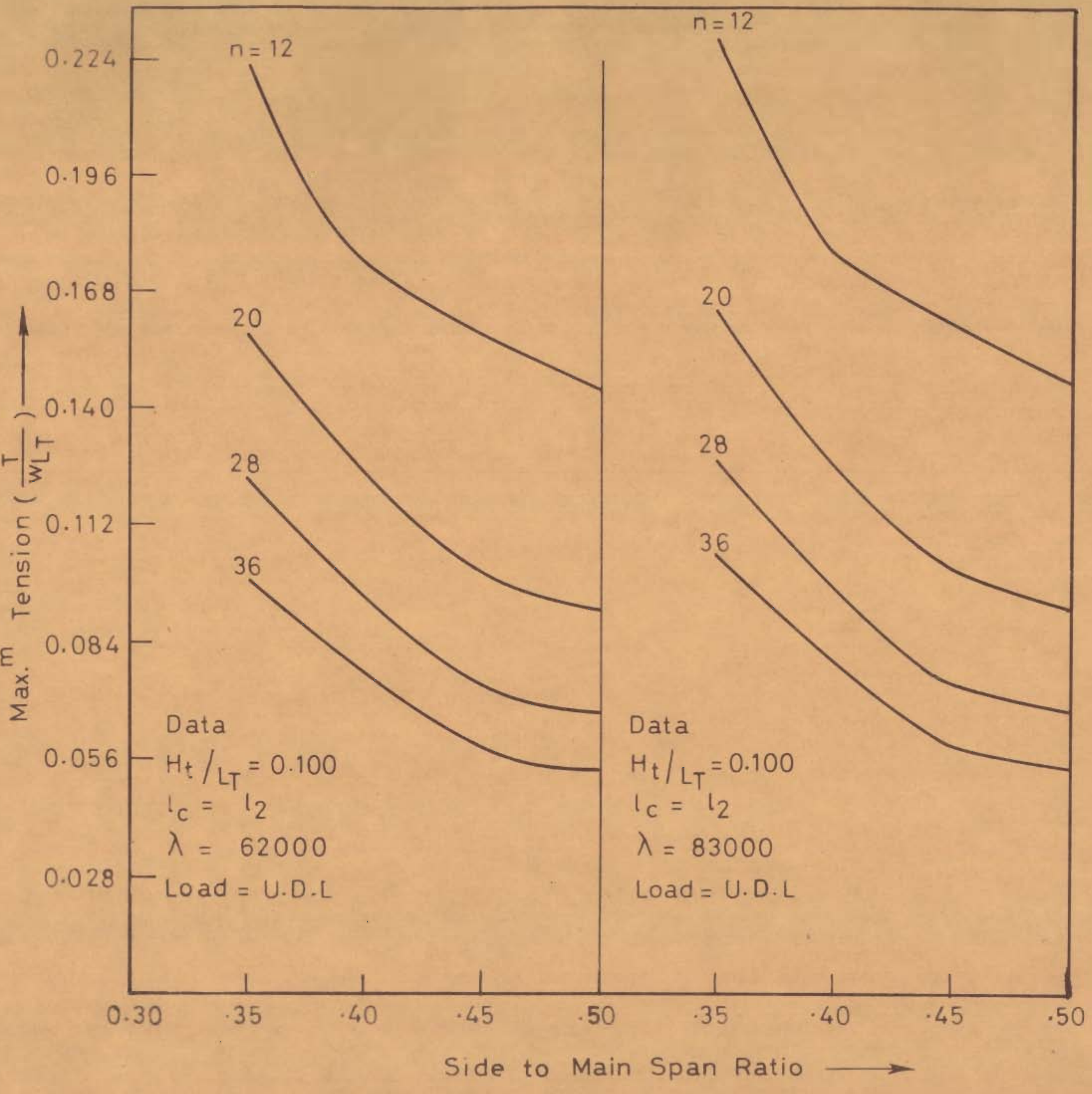


Fig. 6.21 (e) Cable Tension

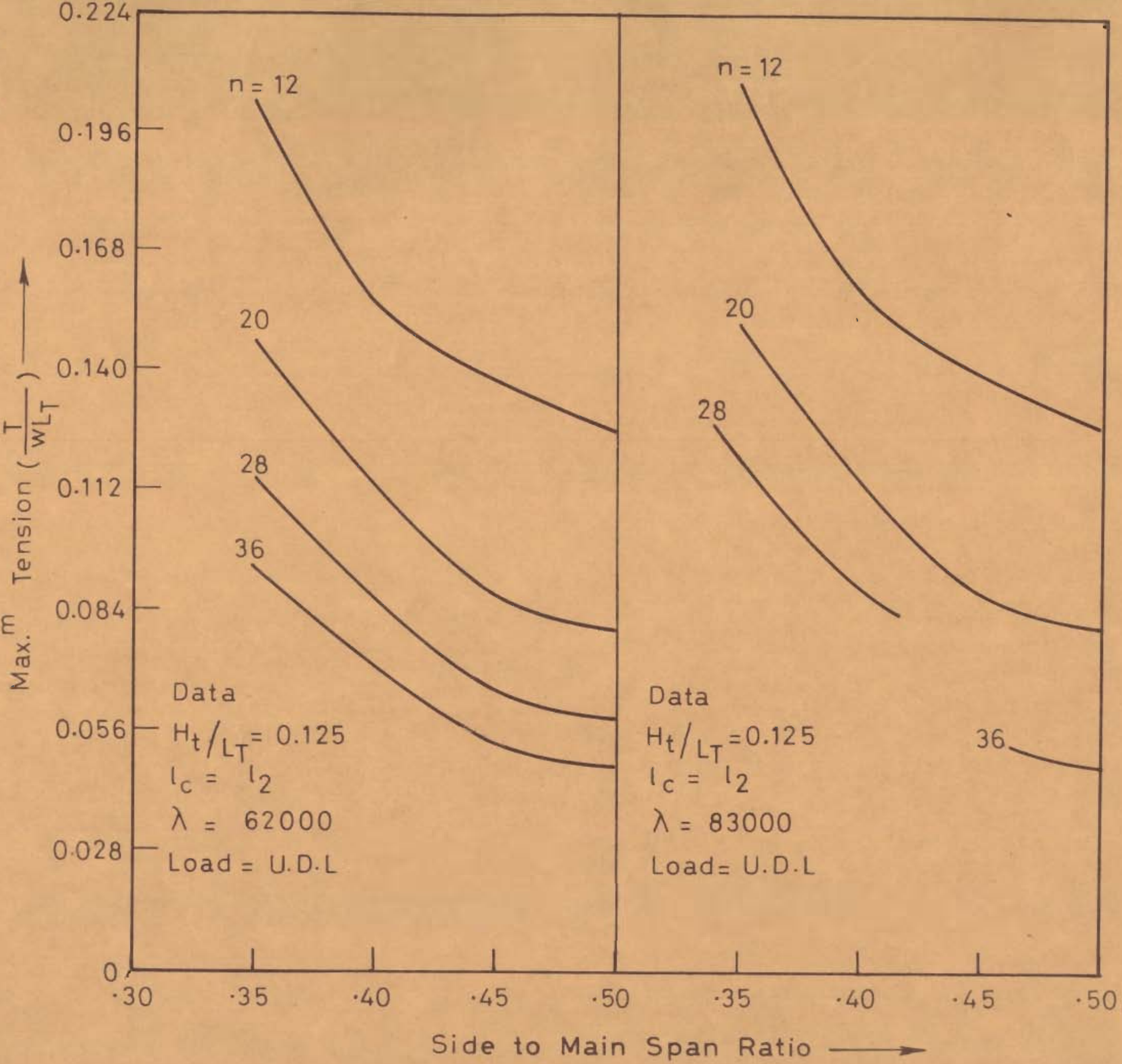


Fig. 6.21(f) Cable Tension

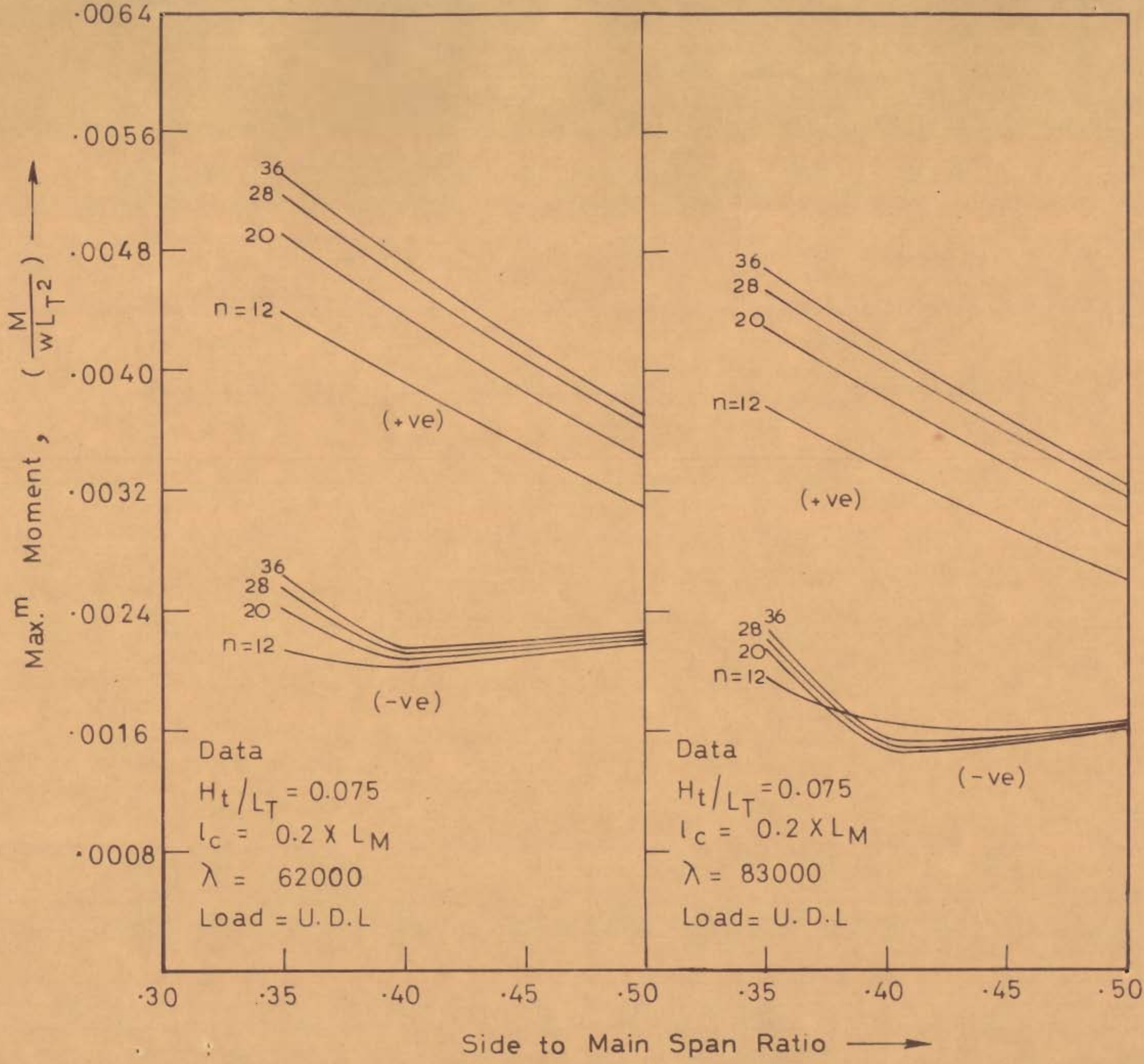


Fig. 6.22(a) Girder Moments.

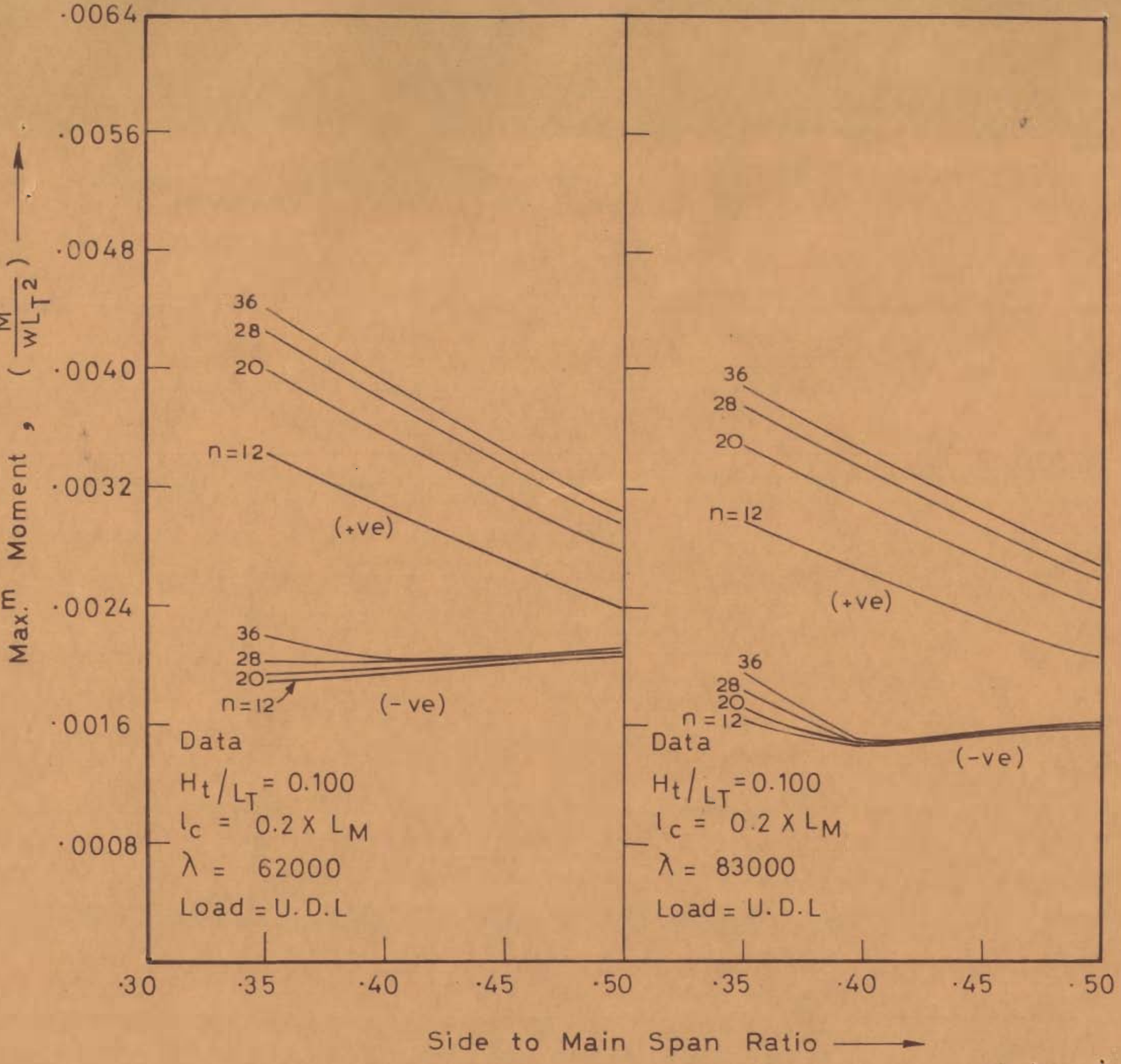


Fig. 6.22 (b) Girder Moments

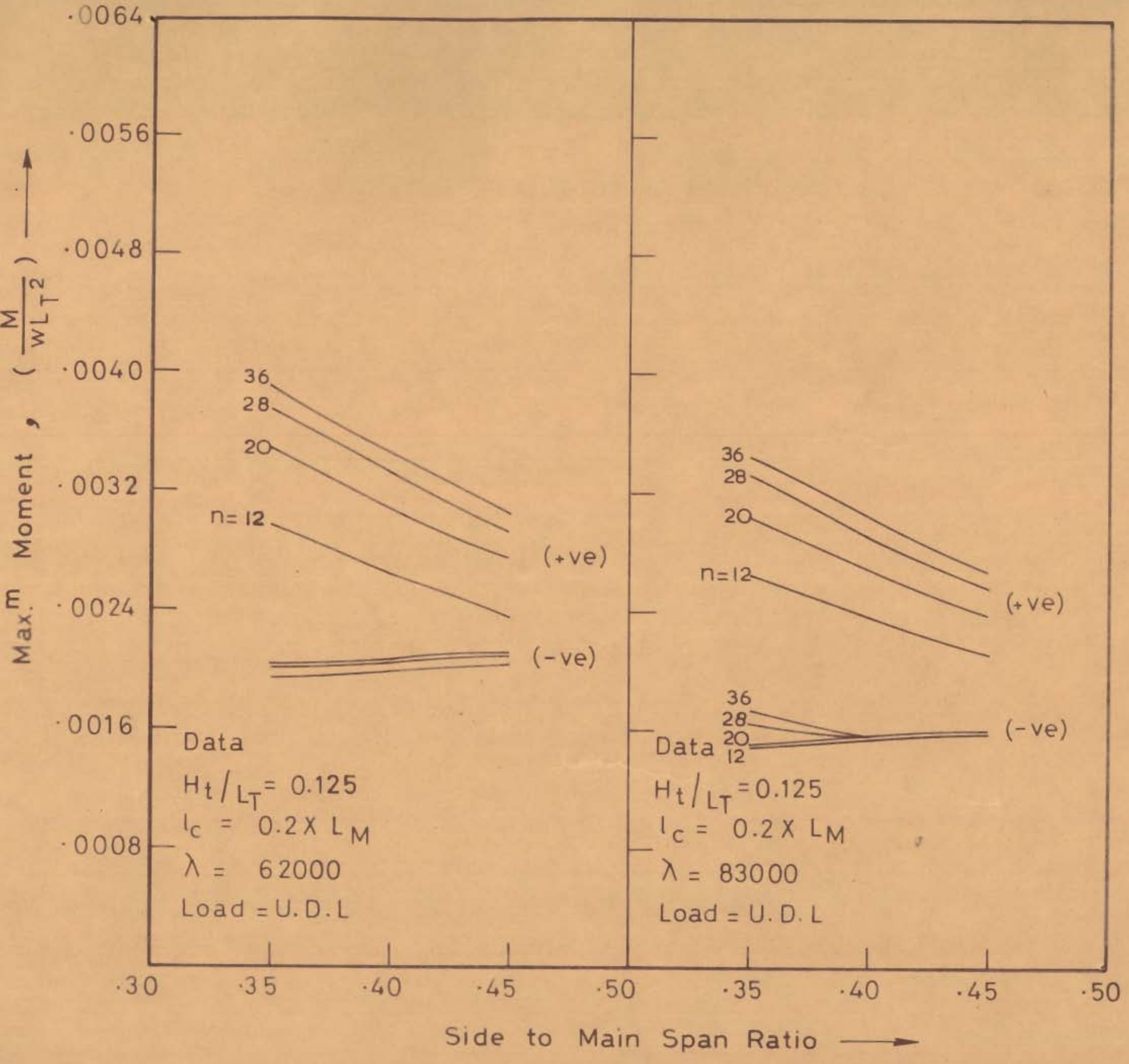


Fig. 6.22(c) Girder Moments

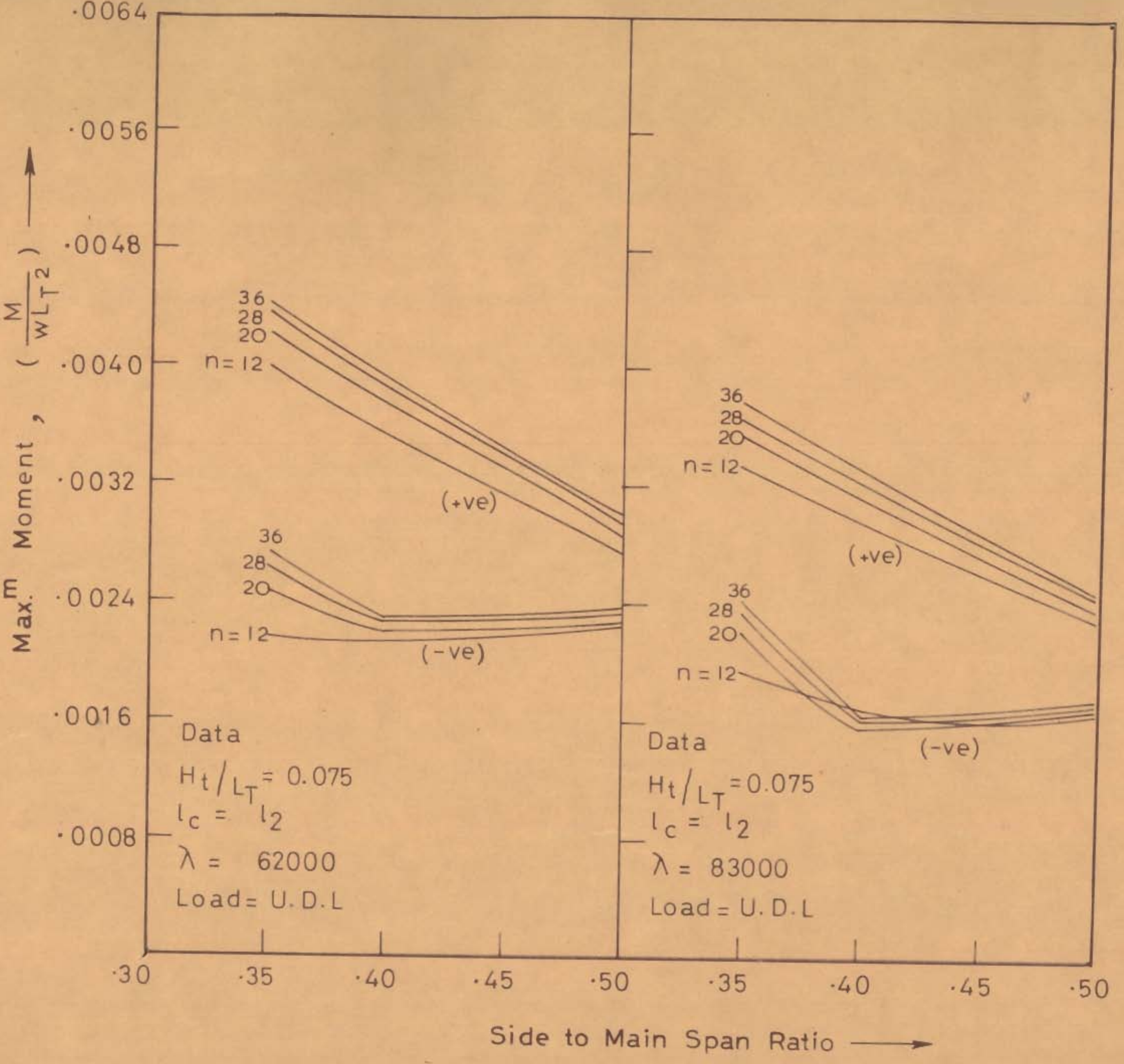


Fig. 6.22 (d) Girder Moments

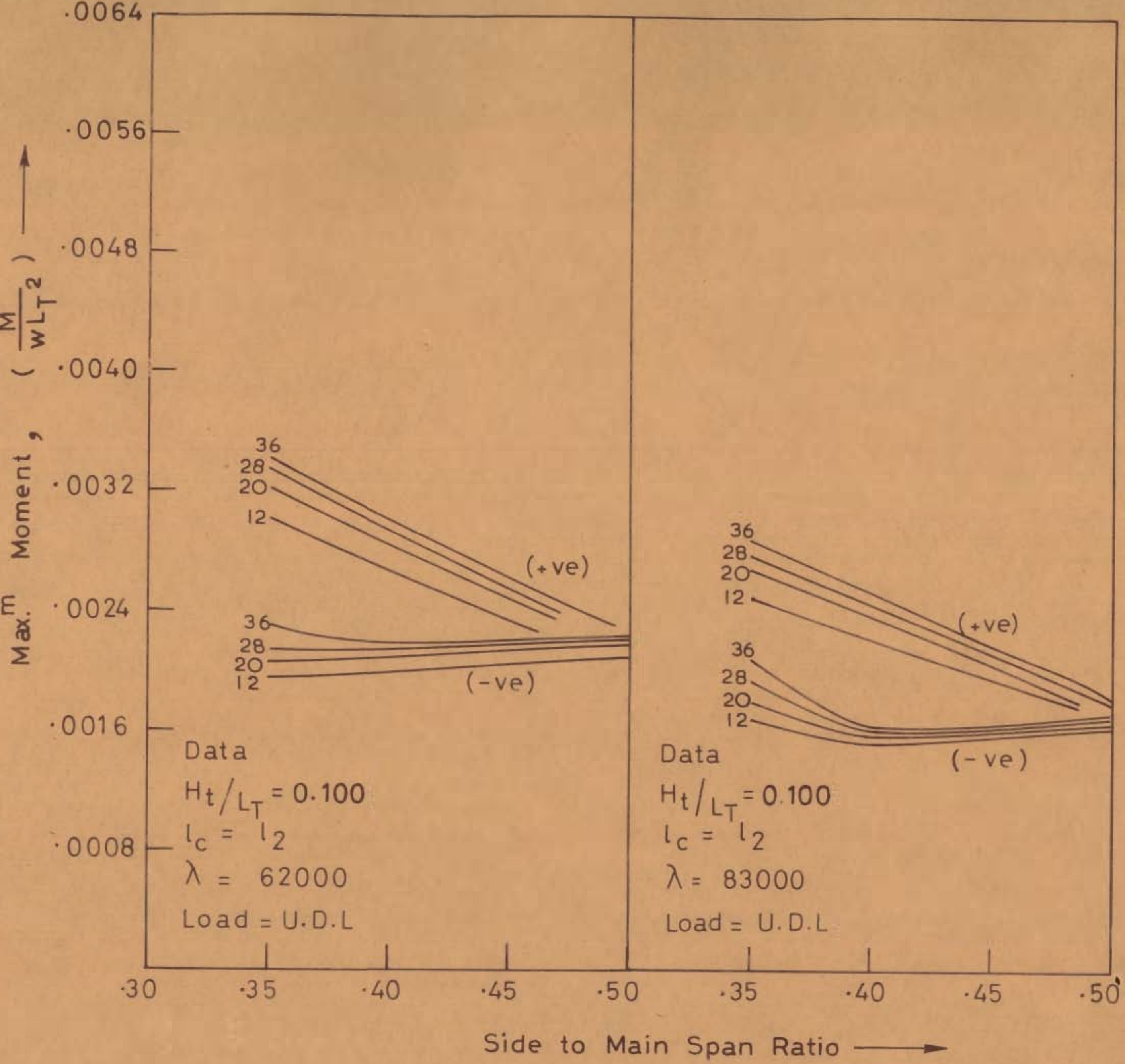


Fig. 6.22 (e) Girder Moments

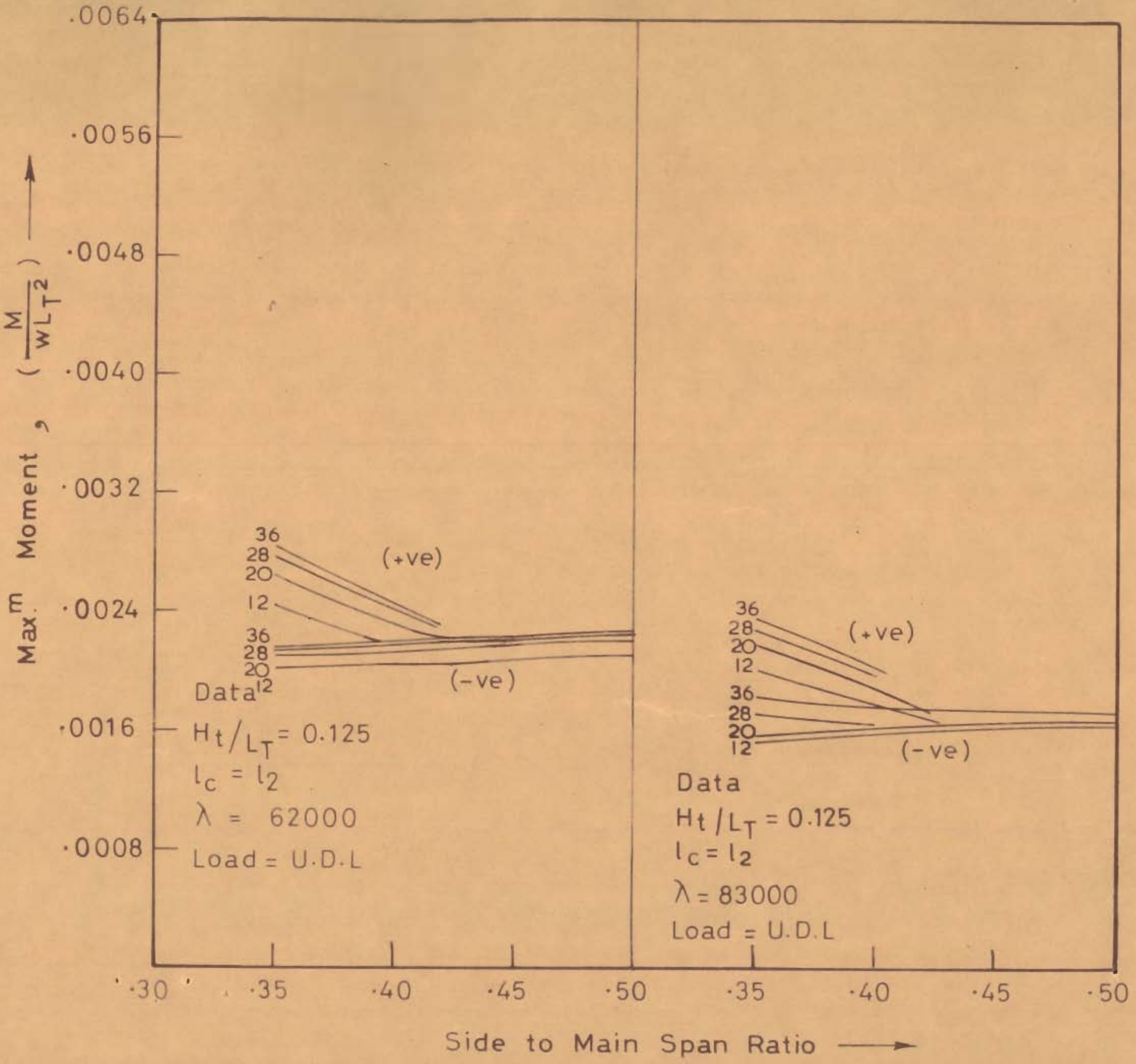


Fig. 6.22(f) Girder Moments.

CHAPTER VII

EXPERIMENTAL WORK

7.1 GENERAL

Properly conducted field tests on prototypes provide excellent data for understanding the behaviour of structures. However, economic problems either eliminate full scale field testing or severely reduce the scope. Properly planned and controlled model tests provide a good alternative. Model tests can be used to check the validity of the assumptions made in the analysis. Further they help in predicting the physical behaviour of the structure. There are not many reports on model tests carried out on cable-stayed bridges in the current literatur. This chapter deals with the details of the experimental work carried out. A perspex model to a scale of 1/200 of a proposed cable stayed bridge across River Hooghly at Calcutta had earlier been fabricated for dynamic studies in the School of Research and Training in Earthquake Engineering at the University of Roorkee. This model was suitably modified for static tests in the present study.

The objective of the model tests was to study the correlation between theoretical and experimental results, and thus to verify experimentally the validity of the assumptions made in theoretical analysis which was carried out both by linear and nonlinear methods and the structure was approximated to a plane frame in some cases, and treated as a space frame in others.

7.2. DESCRIPTION OF PROPOSED SECOND HOOGHLY BRIDGE AT CALCUTTA

The design for the proposed second Hooghly bridge at Calcutta consists of six traffic lanes, three in each direction with a physical separation between them. The total width of the deck including footpaths is 34.5 m. There are two longitudinal girders with a 3 m x 2 m box section, the centre to centre distance between them being 27.5 m. The longitudinal girders have a varying cross section. Cross beams are provided at a spacing of 7.5 m, in the form of I-shaped plate girders.

The main span between the two towers which are made of steel, is 457.2 m. In addition it has got two side spans on each side. Each side span is of 91.44 m length. The longitudinal girder is supported on rollers at the piers except left pier at which it is hinged. The height of the towers is 105.3 m. The towers are fixed at the base and are continuous over the piers. The longitudinal girders are supported on rollers at the towers. The towers are connected by means of three cross beams in the transverse direction. Two cross beams are above the deck level and one below. High tensile steel wire ropes are to be used as the cables. In each plane 62 cables have been provided. Fig.2.1 shows details of this bridge.

7.3 DETAILS OF THE MODEL

As already mentioned in Section 7.1 a perspex model of the second Hooghly Bridge was already available and was suitably modified for the tests reported herein. This model was to a scale of 1/200 and consisted of the complete bridge including piers and wells. The scale factor was chosen on the considerations of facilities of testing equipments available, cost of fabrication and time required in fabrication. Perspex was used to facilitate easy fabrication. Jointing of perspex sheets was achieved by using chloroform. Members with large thickness were formed by sandwitching available sheets of appropriate thicknesses. Spring steel wires were used to represent cables. The details of the model are shown in Fig. 7.1. The dimensions of the model components were worked out to represent the flexural rigidities of towers, deck, piers as well as wells taking into account the actual materials in the prototype and perspex in the model. Also the masses were scaled properly according to model laws for natural frequency determination and additional masses were pasted at the deck in the form of little brass cylinders. The sizes and other details of components of bridge model are discussed below. (See Appendix A)

7.3.1 Deck

The model deck consists of longitudinal girders, cross beams and the deck plate. The longitudinal girders

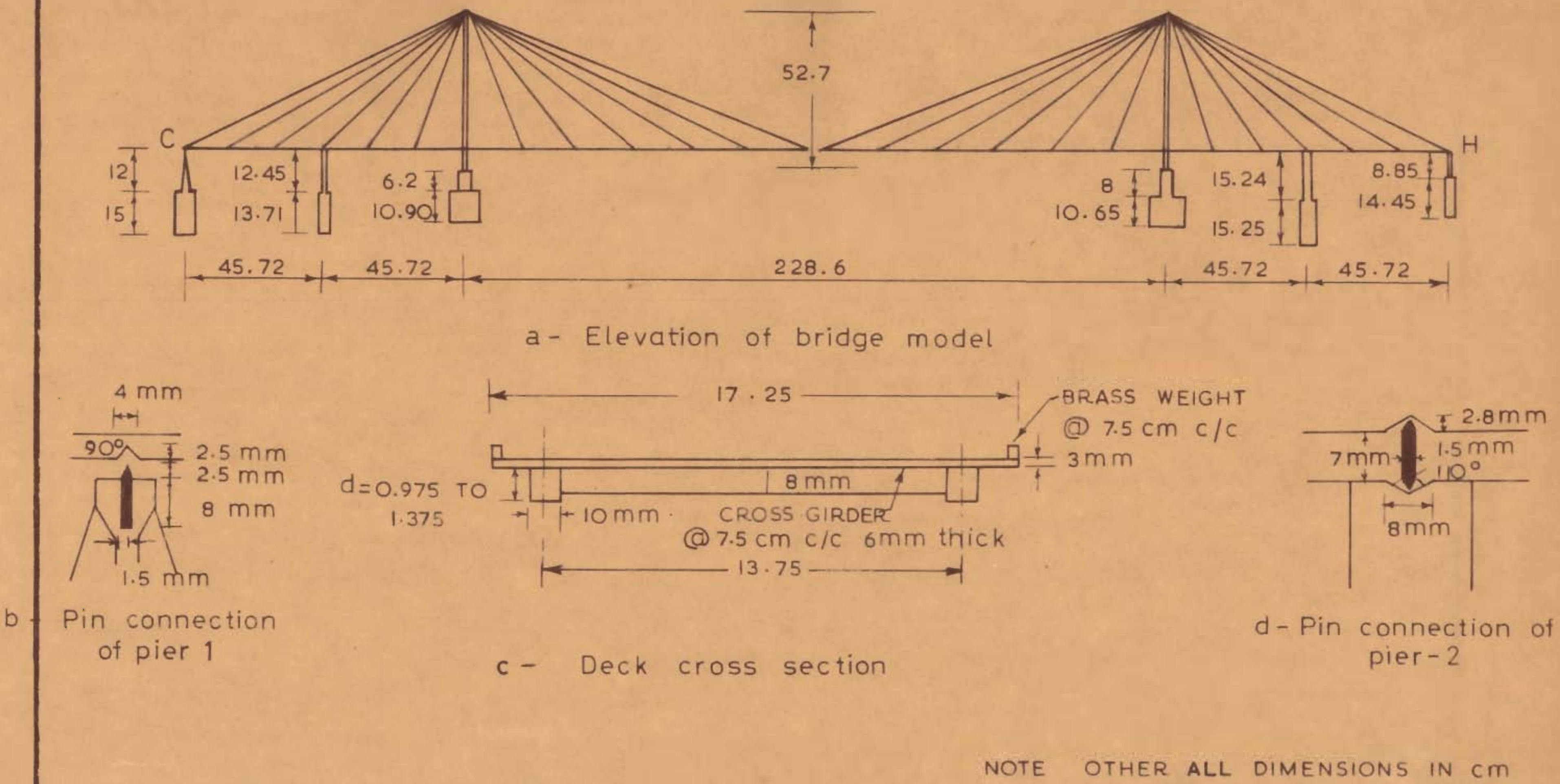


Fig. 7.1 Details of Perspex Model of Hooghly Bridge.

are placed at a centre to centre distance of 13.75 cm. The width of the girders is 1.0 cm and depth varies between 0.975 cm to 1.375 cm.

The deck plate is made with a 3mm thick perspex sheet. The total width of the deck is 17.25cm. This deck is supported on two main longitudinal girders and cross beams.

The main span between the two towers is 228.6 cm long. There are two side spans on each side of the main span, each of length 45.72 cm. Vertical pin supports free to rotate at both ends are provided to the longitudinal girders at piers and towers. These pins do not provide restraints to the horizontal displacements and rotations, but do restrain vertical movement. The vertical lift of the girders is restricted by clamping the girder to the support by a thin wire.

The deck is supported on cross beams which are supported on longitudinal girders. The size of these cross beams is 8 mm x 6 mm and they are provided at a spacing of 7.5 cm centre to centre. Same stiffness and mass ratios are maintained between the prototype and the model between the various elements.

7.3.2 Towers

Two towers are provided to support the main and side spans. The height of the towers above the deck level is

52.7 cm. The cross section of the tower legs tapers from 20 mm x 20 mm at the deck level to 15 mm x 15 mm at the top. The towers are connected by three cross beams in transverse direction. Two cross beams, 10 mm x 19 mm, are provided above the deck level and one 10 mm x 23 mm, below the deck.

The towers are continuous above the piers which are fixed to the wells. Hollow piers and wells are made by turning the thick members. The wires are anchored individually at the top of the tower to a brass anchor. The details of the anchorage are shown in photograph No. 1. The size of this anchorage block is 16 mm x 16 mm x 25 mm.

7.3.3 Cables

Spring steel wires are used as cables. The number of wires is reduced to 24 in one plane, to facilitate the prestressing and measurement of change of cable tension. Cable diameters and their spacing is correspondingly changed. The closeness of the diameters of wires required in the model and actually provided is based on the availability of these wires. In calculating the theoretical results, actual diameter of the wires is taken. The wires are anchored below the deck. A prestress adjusting barrell is provided between the longitudinal girder and the grip. The holes through which these pass are drilled at an angle to coincide with the slope of the wire. Load cells are provided in wire nos. 4, 12, 13, 21 in both planes. These wires are chosen to maintain symmetry.

7.3.4 Grips

To anchor the wires to the deck, brass grips are used. At the anchor point a kink is formed in the wire which prevents slippage of the wire. Spacial grips with hemispherical heads are used to grip the wires at the load cells. Hemispherical heads are chosen to maintain the same contact area between the grip and the load cell, since the failure to do so would alter the strain measurements.

7.4 PROPERTIES OF MATERIALS USED IN THE MODEL

An important factor affecting the correlation of theoretical results with experimental results is the accuracy with which material properties are determined. The behaviour of the structure in the elastic range is mainly affected by the modulus of elasticity of the materials used. The materials used in the model are perspex and spring steel. Other materials used for grips, additional weights etc. are brass and aluminium which are estimated not to affect the structural behaviour of the model in any major way. The methods adopted for determining the modulus of elasticity of spring steel wires and perspex are presented below.

7.4.1 Modulus of Elasticity of Spring Steel Wires

These wires represent cable stays in the model. The value of E was determined with the help of Searle's standard apparatus shown in Photo. 2. The average modulus of elasticity of spring steel wires was found to be 2.11×10^6 kg/cm^2. A typical set of readings and stress-strain relationship for these wires is shown in Fig.7.2.

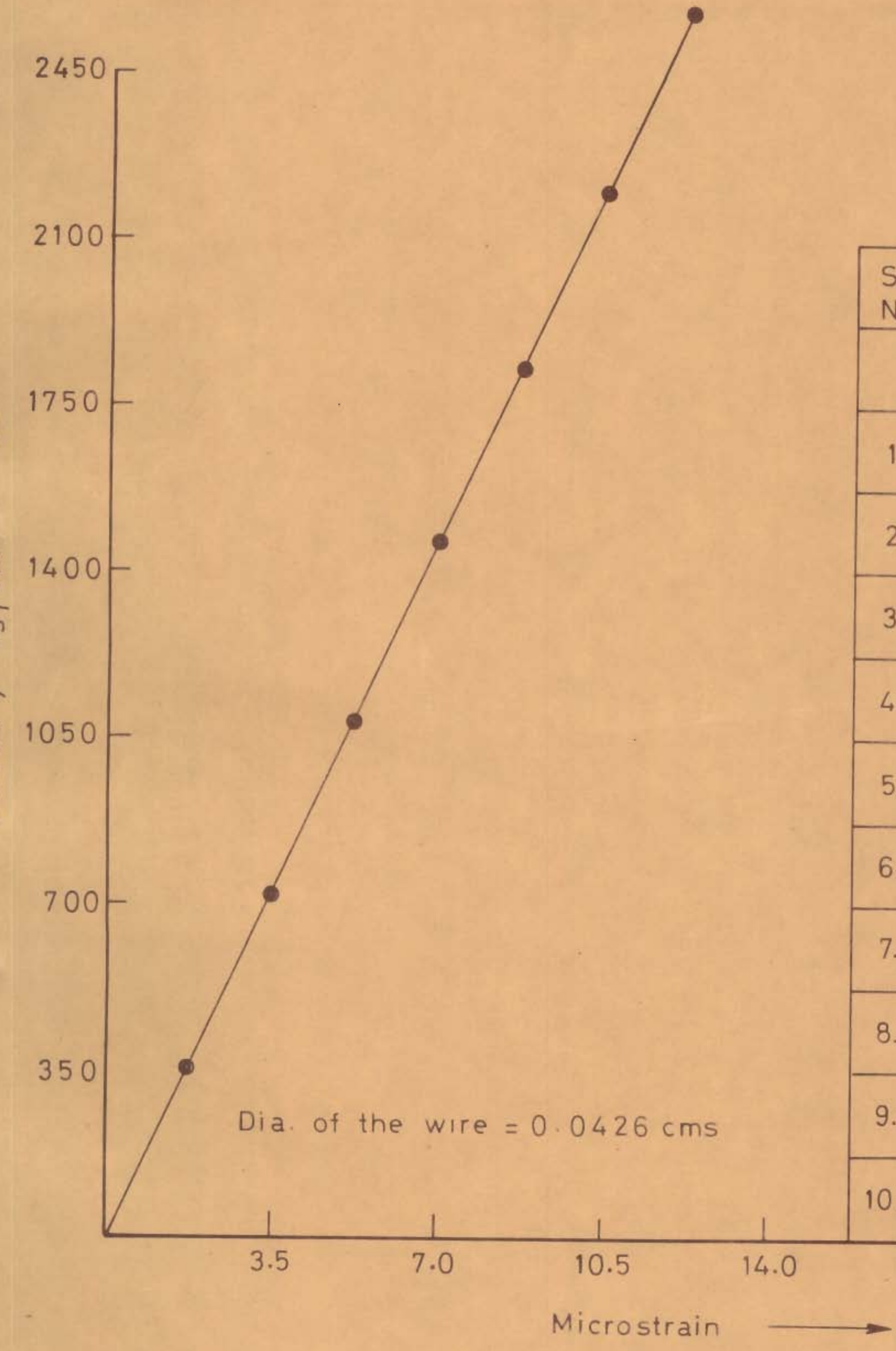
7.4.2 Modulus of Elasticity of Perspex

The members made up of perspex are mainly subjected to flexural stresses. Hence the modulus of elasticity of perspex was determined in bending. For this purpose two simply supported beams of cross-sections 4.005 cm x 1.195 cm and 3.98 cm x 1.235 cm respectively were tested. The first beam was a single piece cut from the perspex sheet and the second was formed by joining two perspex sheets with chloroform. The later specimen was tested to simulate the properties of such members formed by joining two or more sheets used in the model. The length of each of these beams was 60 cm and they were placed on two knife edges at a centre to centre distance of 55 cm. Deflections of the central point were measured with the help of dial gauge. , under the central point load increasing with intervals of 200 grams each. Photograph No.3 shows the experimental set up.

E is given by

$$E = \frac{PL^3}{48\ I\Delta}$$

where, P is the applied central load,



SL No.	Load in Kgs.	Strain X 10^{-4}		Avg. strain
		Load incr.	Load decr.	
1.	0.5	1.75	1.75	1.75
2.	1.0	3.50	3.50	3.50
3.	1.5	5.25	5.25	5.25
4.	2.0	7.00	7.00	7.00
5.	2.5	8.75	8.75	8.75
6.	3.0	10.50	10.50	10.50
7.	3.5	12.25	12.25	12.25
8.	4.0	14.0	14.0	14.0
9.	4.5	15.75	15.75	15.75
10.	5.0	17.50	17.50	17.50

Fig. 7.2 Determination of Modulus of Elasticity of Spring Steel Wire.

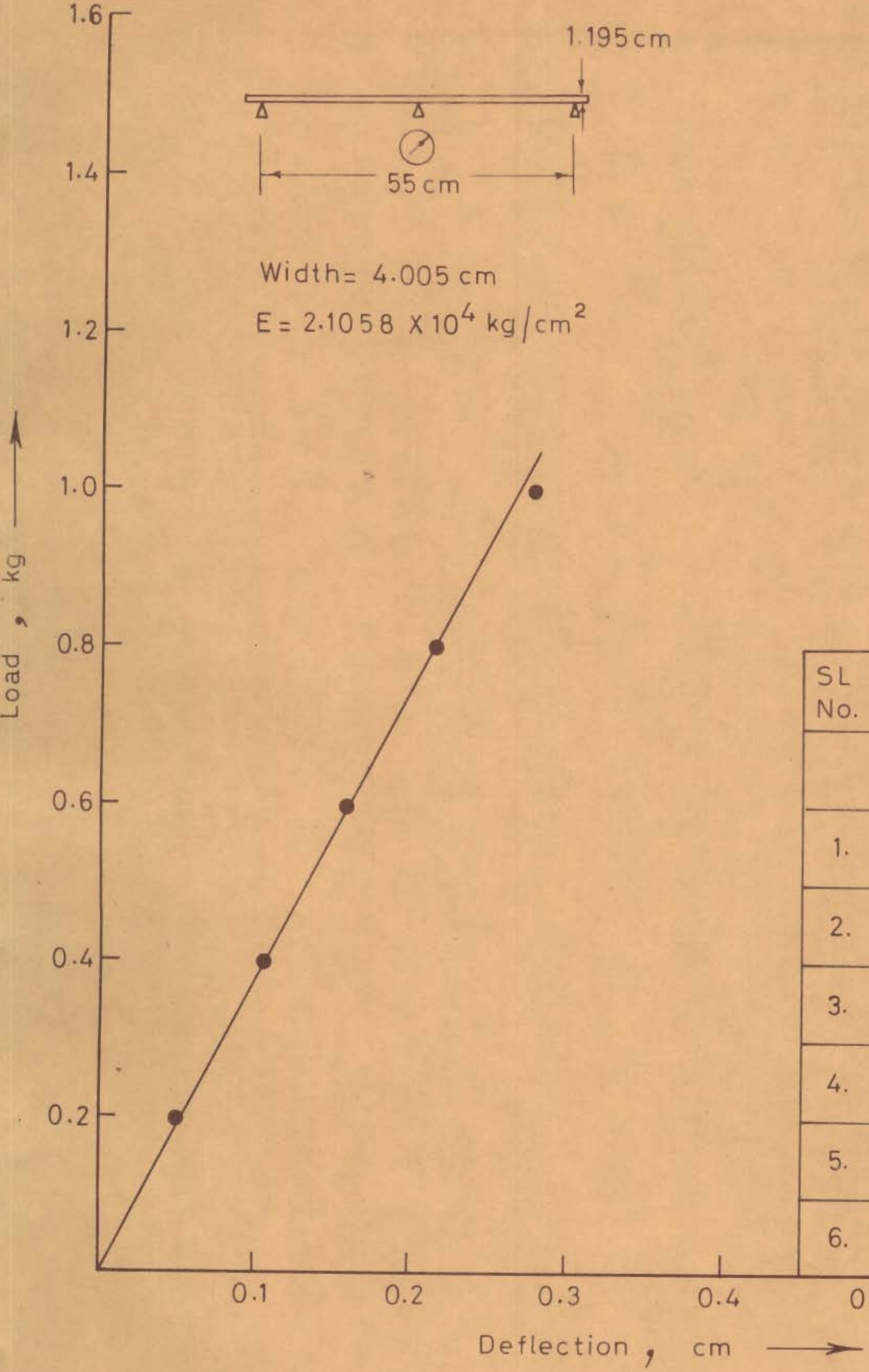
Δ the central deflection, L the span, I the moment of inertia and E is the modulus of elasticity of the material.

The average value of modulus of elasticity of perspex, obtained from the tests carried out on two beams, was found to be 2.10 x 10^4 kg/cm^2. Figure 7.3 shows the load-deflection relationship of the perspex beam made out of single piece. A table showing the deflections for load intervals as well as a line diagram of the apparatus are also shown in the same figure.

7.5 FABRICATION AND CALIBRATION OF LOAD CELL

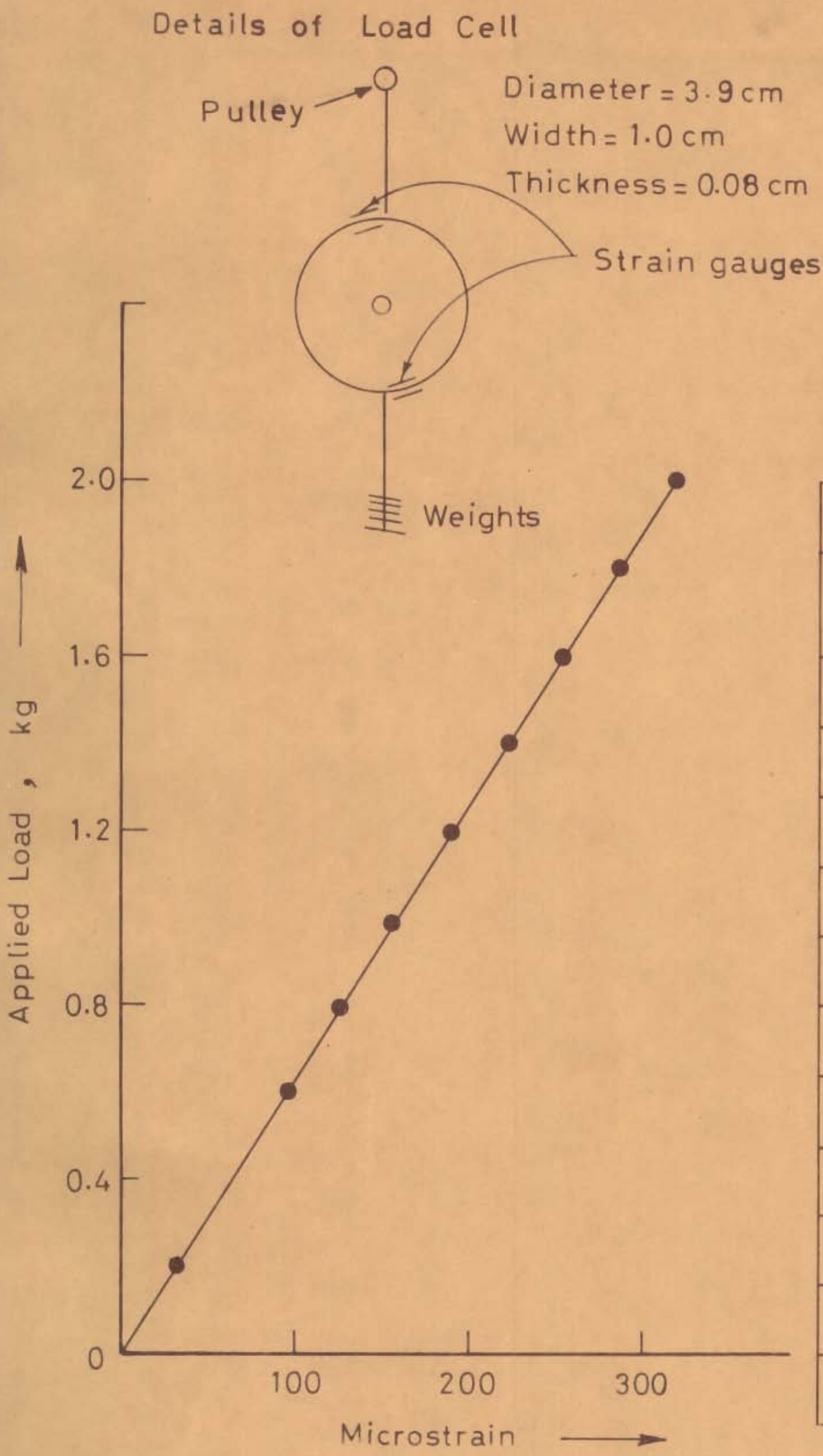
The measurement of tension in thin wires is not a straight forward problem. Specially designed and manufactured load cells were used for this purpose. Details of the cells are shown in Fig. 7.4 and Photo. 4. These load cells were in the form of circular rings of external diameter 3.9 cm. The width of the rings was 10 mm and thickness varied from 0.8 mm to 1.0 mm. These load cells were introduced in the wires by intercepting them near the longitudinal girder and were clamped with the help of hemispherical grips. The contact area between the grip and the ring was always same. Load cells were positioned close to the longitudinal girder in order that their weight did not lead to any appreciable effect on the wire geometry or tension.

These load cells were fabricated by machining a



SL No.	Load in kgs.	Deflection in cms.		Avg. Defl. in cms.
		Load incr.	Load decr.	
1.	0.2	0.049	0.051	0.050
2.	0.4	0.104	0.102	0.103
3.	0.6	0.160	0.158	0.159
4.	0.8	0.217	0.219	0.218
5.	1.0	0.281	0.279	0.280
6.	1.2	0.344	0.346	0.345

Fig. 7.3 Determination of Modulus of Elasticity of Perspex.



S.No	Load in kgs.	Bridge Readg.		
		Load incr	Load decr.	Avg.
1	0·0	820	820	820
2	0·2	850	850	850
3	0.4	885	880	882.5
4	0·6	920	910	915.0
5	0·8	950	940	945.0
6	1·0	980	975	977.5
7	1·2	1010	1010	1010
8	1·4	1045	1040	1042·5
9	1·6	1075	1075	1075
10.	1·8	1110	1105	1107.5
11	2·0	1140	1140	1140

Fig. 7.4 Calibration of Tension Ring D_2

galvanised iron pipe of suitable diameter and thereafter annealed under graphite powder at a temperature of 835°C to release stress concentration if any. Two strain gages each of 120 ohm resistance, gage factor 2.11, 3 mm grid length were pasted on each ring, one on the outer surface and the other on the inner surface. Since stresses and strains are maximum near the points of attachment of wire, these gages on the opposite side of the metal are subjected to tensile and compressive stresses respectively when the ring bends. These gages were connected to the strain measuring bridge such that the values of strains in both the gages were additive, thus giving greater sensitivity. Also one gage acted as active and the other as the compensating gage. These gages were pasted as close to the attachment point as possible.

7.5.1 Calibration of Load Cells

Calibration of the load cells was done through direct loading. Strains were measured at a loading interval of 200 grams. The calibration was done for both increase and decrease in load. Several sets of such readings were taken for each load cell to check repeatability. The average of these readings was taken. Values of the calibration constant, i.e. strain per kilogram load obtained by these tests are given for various load cells in Table 7.1. Figure 7.4 shows a typical calibration curve for the load cell D_2. A table showing the calibration values of this load cell is also shown on the same figure.

Table - 7.1

Calibration Constants for Load Cells.

Load cell No.	Microstrains per kg
D_1	165
D_2	160
D_3	138
D_4	184
D_5	223
D_6	187
D_7	268
D_8	195

7.6 PRESTRESSING OF WIRES

The cables in cable stayed bridges are to be prestressed to reduce the girder moments due to dead load. The prestress would also prevent slackening if compressive forces develop under live load in any of the cables.

It was decided to introduce prestress in the wires of the model equal to the dead load tensions. Cotton bags, containing weights equal to estimated dead weight tension in the respective wire, were suspended from hooks connected to the wires at their bottom ends. The wires were clamped at the bottom, and the prestressing barrel adjusted to bring the grip flush against the girder. The bags were then removed.

7.7 TESTING OF THE MODEL

The perspex model described in Art. 7.3 was tested under concentrated loads placed at various points as given below.

(i) Central point of the main span.

(ii) Central point of the outer side span.

(iii) Central point of the inner side span.

In each case of loading, weights were directly placed on the model and increased at an interval of 0.5 kg. The displacements and tension changes were recorded in each case. The vertical displacements were measured with the help of dial gauges fixed at the centre of main and side spans and also at the quarter points of the main span. Horizontal displacements of the tower tops were measured by fixing dial gauges parallel to the longitudinal axis of the model. The change of tension in wires was measured with the help of pre-calibrated load cells as described in Art. 7.5. These were attached to cable nos. 4, 12, 13 and 21 in each cable plane. The readings of displacements and change of tensions were taken at laod intervals of 0.5 kg. The results are discussed in detail below.

Figure 7.5 (a) shows the variation of maximum deflection with load when the main span is centrally loaded. This figure indicates that the load-deflection relationship is nonlinear. Initially, for smaller loads, the deflections obtained by model tests are larger than the theoretical

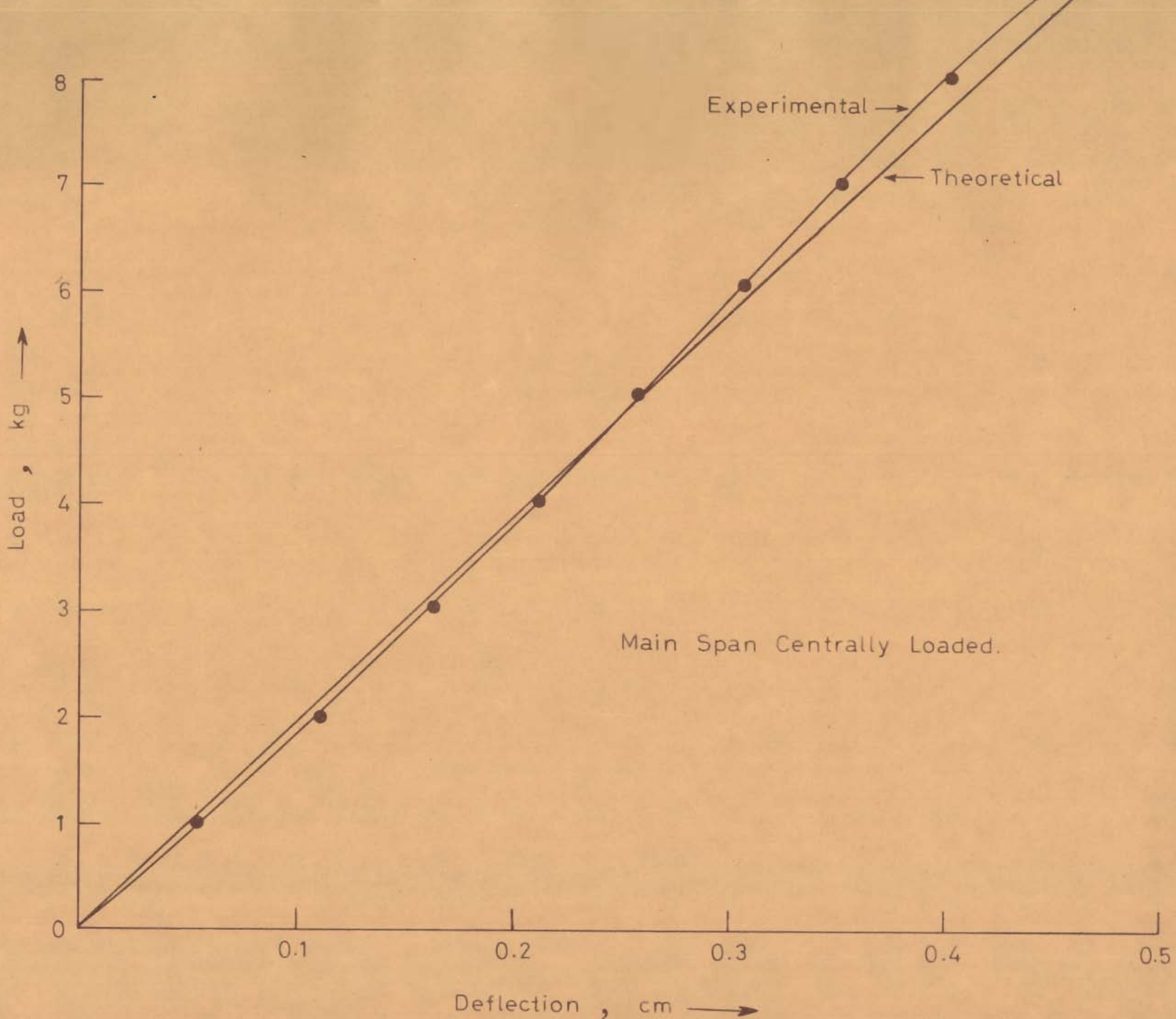


Fig.7.5(a) Load-Deflection Relationship for the Model.

deflections. In the initial stages of loading some of the wires were slack. Kinks in the wires disappeared as tension increased. It may be concluded that in the initial stages of loading some of the wires did not effectively support the deck. After a load of 5 kgs the experimental values of deflections are smaller than those obtained by the analysis. The maximum difference between the theoretical and experimental deflections is 1% at a 3 kg load.

Figure 7.5(b) shows the variation of maximum deflection with load, both theoretical and experimental values of deflections being plotted for the two cases of outer and inner side span loaded centrally. The maximum deflection in each case is under the load. The experimental deflections are larger than those obtained by the analysis for both cases. This difference varies approximately between 15 to 20% for the load on the inner side span and about 4.5% for the other case. In the case when the outer side span is loaded the values of deflections are larger than those obtained for the case of inner side span loaded. This is due to the support condition at the end. It may be observed further that the load-deflection curve deviates from linearity by only small degrees, or not at all. The observed nonlinearity as of the hardening type.

The tension changes are measured with the help of precalibrated load cells. In general the theoretical values of tension changes are smaller than those obtained

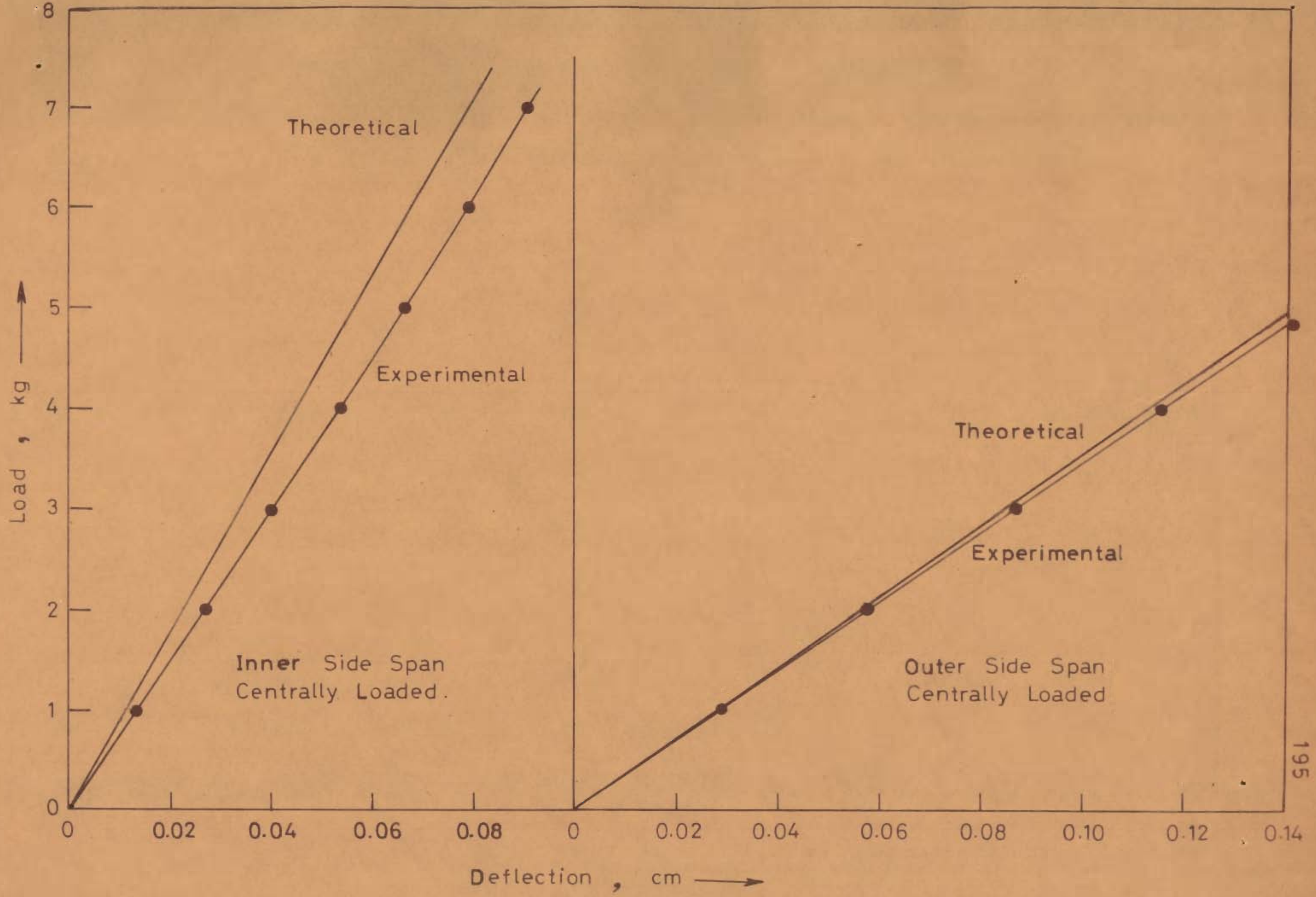


Fig. 7.5 (b) Load - Deflection Relationship for the Model.

by the model tests. The difference between the theoretical and the experimental values of tension changes vary between 15 to 25%..

The experimental results helped to verify the general behaviour pattern obtained theoretically and the effort proved most useful. It may be observed that differences between the measured and computed values are not small in all cases. This could be attributed to several factors.

(i) There were some discrepancies between the stipulated wire tensions and those actually obtained. These were estimated to be between 10 to 15%..

(ii) There were also some discrepancies in the obtained geometry.

(iii) As in most other cases the measurement technique could not be considered totally free of blemish.

Photo.1. Details of Anchorage of Wires at the Top of Tower.

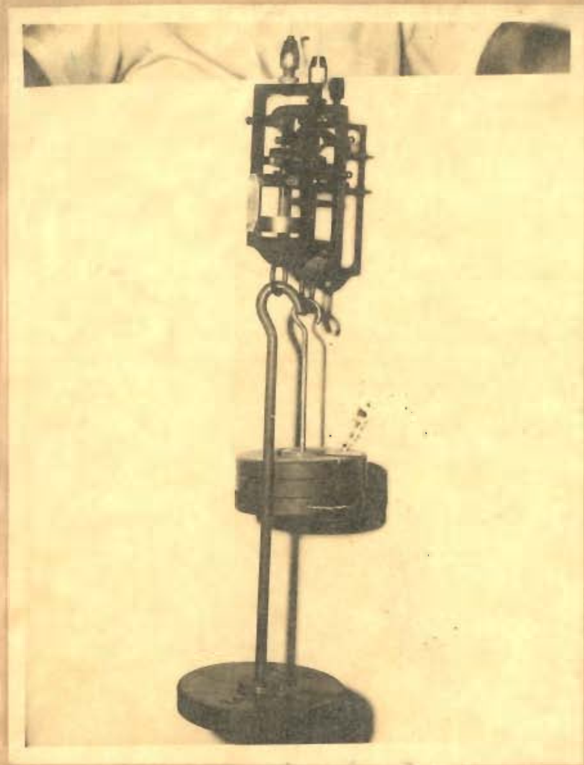


Photo.2. Searle's Apparatus.

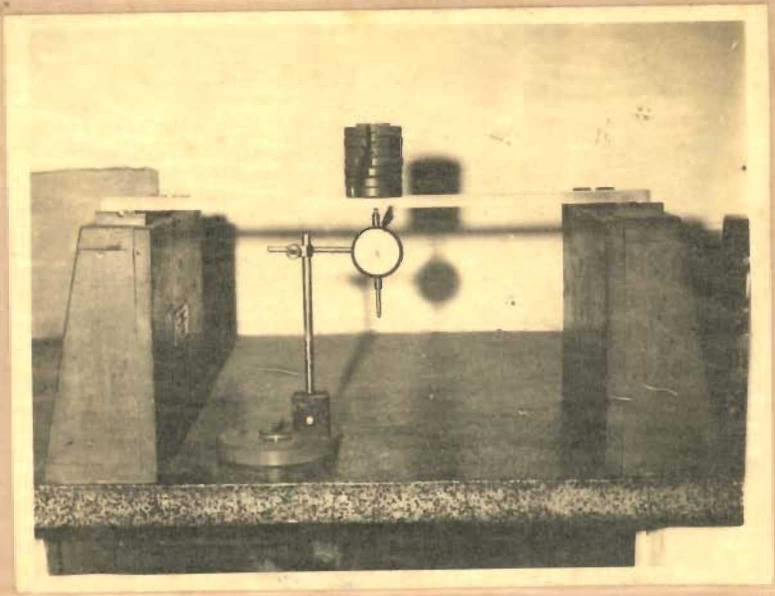


Photo. 3. Determination of Modulus of Elasticity of Perspex.

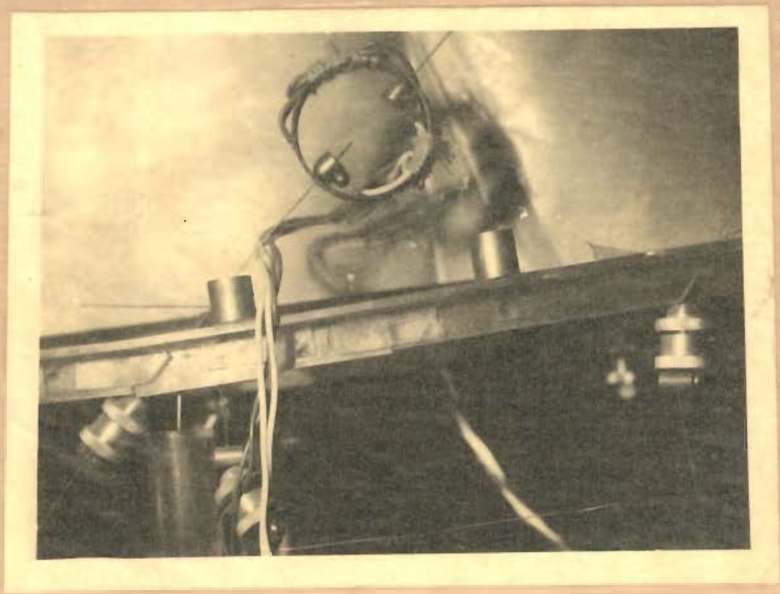


Photo. 4. Load Cell

CHAPTER VIII

SUMMARY AND CONCLUSIONS

8.1 GENERAL

A cable-stayed bridge is a highly indeterminate structure. The behaviour of longitudinal girders in cable stayed bridges is similar to that of a continuous beam supported on elastic foundation. The elastic support is provided by the cables at the points of cable attachment. In the construction of existing cable stayed bridges various geometric configurations have been utilised. The two important longitudinal cable configurations are the radiating and the harp type and these are the ones studied herein.

The following studies have been carried in this thesis and presented earlier in Chapters 3 to 7.

(a) Linear elastic analysis of the bridge under static vertical and horizontal loads. The results of of two-dimensional (plane frame type) and three dimensional (space frame type) analyses are compared.

(b) Investigation of large deflection non-linearity effects with reference to a long span cable-stayed bridge.

(c) Investigation of erection stresses as would develop in the double-cantilever method of construction and how they can be controlled by prestressing of the cables simultaneously as the erection proceeds.

(d) Experimental study using a perspex model for verification of analytical results.

(e) Parameteric analytical study to study the influence of variation of individual parameter values on the important stress resultants in the bridge.

(f) Development of design charts for preliminary design of cable stayed bridge cables, girders and towers.

The conclusions obtained from these studies are described in the following articles.

8.2 LINEAR ELASTIC ANALYSIS

Amongst the various methods of linear elastic analysis of structures, the direct stiffness matrix method was considered to be best suited for the analysis of the cable stayed bridges. Although the structure is basically a space frame due to the action of towers, cables and the deck, it is not uncommon to treat it as plane frame through suitable assumptions. The difference in the results using these two approaches has been investigated herein. Two computer programs have been developed, one treating the bridge as a plane frame and the other treating it as a space frame. For comparison of the results, the cable stayed bridge **of radiating type with 8 cables in one**

at Calcutta has been considered for analysis. It has a main

plane (Fig. 3.3) has been considered for analysis. It has a main span of 200 m and one side span of 80 m length on each side.

For different loading cases were considered which included symmetrical, unsymmetrical as well as lateral loading on the bridge. The main conclusions arrived at are the following:

(i) Under symmetrical loads on both the longitudinal girders, the results obtained by the two dimensional and the three dimensional analyses are exactly the same and the two dimensional analysis can be used.

(ii) Under unsymmetrical loads, the two dimensional analysis gives higher values of forces and deflections in main girders as compared to those obtained by the three dimensional analysis but the latter exhibits moments and torsions in deck elements which the former does not.

(iii) For static wind loads the linear analysis both two and three dimensional, does not give the actual behaviour of the bridge.

8.3 GEOMETRICAL NONLINEARITY ANALYSIS

The nonlinear behaviour of the cable stayed bridges may be due to various reasons, such as (i) Large deflections, (ii) Axial forces in the various elements of the bridge, and (iii) Non-linear behaviour of the material. In this study the non-linearity due to large deflections has only been investigated by the 'plane frame' approach. For this purpose an iterative approach has been used. The two dimensional non-linear analysis included the effect of the type of support at the tower bases and the flexibility of the girder. From this study, the following conclusions can be drawn:

i) The nonlinearity due to large deflections is very small.

ii) Nonlinearity increases with the increase of girder flexibility.

iii) Nonlinearity is larger in case of bridges with hinged tower base as compared with fixed tower base.

iv) Nonlinearity is larger in case of unsymmetrical loads.

v) In general the nonlinear analysis values of forces and deflections are smaller than the linear values, indicating that the bridge has a 'hardening' type load-deflection characteristic in the vertical plane.

8.4 ERECTION STRESSES

As stated earlier, the erection stresses have been worked out for the method of construction using double cantilevers. This study has two main aspects:

i) Effect of the length of the panels on the girder moments when the cables are just installed to support the panels without prestress, and

ii) Control and reduction of girder moments by pre-stressing the cables during erection.

The main results obtained in these cases are presented below:

8.4.1 Effect of Panel Lengths on the Girder Moments

In this part of the study, bridges with radiating arrangement with 36 cables were taken and in one case the panel lengths were taken all equal while in the other case the panel lengths were made unequal increasing in length away from the tower supports. The following results are obtained:

i) The girder moments increase at every stage of erection and finally very large moments are developed in the girder.

ii) In the initial stages of erection, the girder moments in unequal panel case are smaller than those in the equal panel case, but in the final stages, situation reverses. Ultimately unequal panel case results in larger girder moments.

iii) A similar behaviour is observed for the maximum cable tension and girder deflection as for moments in ii) above.

8.4.2 Reduction of Girder Moments by Prestressing the Cables

The aim of prestressing was set to achieve at the final stage the state of the girder just like that of a continuous beam resting on non-deflecting supports. For this investigation bridges with 12,20,28 and 36 cables have been investigated. The following conclusions can be obtained from this study:

i) At the final stage of erection, the tensions in the cables can be made almost equal to the forces obtained from the support reactions of an equivalent continuous beam.

ii) At the final stage of erection, the girder moments at all the cable points except near the cantilever ends are approximately equal to $w\ell^2/12$ i.e. the support moments in the continuous beam having the span length ℓ.

iii) The moment in the girder at the second exterior most point is approximately equal to $w\ell^2/9$, that is, quite close to that in a propped cantilever.

iv) The maximum girder moment in this case with controlled prestressing is reduced to about 0.016 of the maximum moment in the unprestressed equal panel case.

8.5 PARAMETRIC STUDY

In this study the effect of various parameters on the critical stress-resultants in the cable stayed bridges has been investigated. This investigation includes the behaviour of the harp and the radiating arrangements. To arrive at the effect of a parameter, its value was changed keeping the others constant in a base structure. The following parameters were included:

i) Flexural rigidity of towers

ii) Flexural rigidity of girders

iii) Extensional stiffness of cables

iv) Number of cables, length of the central panel and side to main span ratio,

v) Height of the tower, fixity condition at base of tower.

Following conclusions are obtained from these investigations:

i) The maximum values of forces and deflections in the harp arrangement are larger as compared to those in the radiating arrangement.

ii) The effect of the fixity conditions at the tower bases is very small except the maximum moment in the tower itself.

iii) The tower moments increase with the increase in the stiffness of the towers, but the girder moments and deflections and cable tensions are not much affected by the change in the flexural rigidity of the towers.

iv) The effect of the flexural rigidity of the girders on the cable tensions, tower moments and the girder deflections is small. But the girder moments, both positive and negative, sharply increase with the increase in the stiffness of the girders.

v) The cable tensions increase with the increase in the extensional stiffness of cables, that is, increase in cable area, but the change is small. The girder deflections as well as moments decrease sharply for the initial increase of cable rigidity and then the decrease becomes gradual.

vi) The cable tensions and negative girder moments are not much affected by the changes considered in the central. panel length, but effect of the length of the central panel on the positive moments is appreciable. The positive moments are larger in bridges with larger length of the central panel.

vii) The effect of the tower height on the positive moment and the cable tension is appreciable. These forces decreases with the increase of the tower height. However, the effect of the tower height on the negative girder moment is small.

viii) The cost of steel in cables is much smaller compared to that of girder (Art. 6.7).

ix) The change in the number of cables does not affect appreciably the cost of steel in cables.

x) For smaller number of cables the total cost of harp arrangement is larger as compared to that of radiating arrangement. The cost of these arrangements with 28 cables is approximately equal.

xi) In both the above arrangements the total cost is largest when the main span is fully loaded.

xii) The effect of cable area on the cost of steel in cables is small.

xiii) The effect of cable area on the cost of girders and the total cost is large for the small values of the cable area and then change becomes gradual.

xiv) The effect of cable area on the combined cost of cables and girder of radiating and harp arrangement is very small after a cable area 0.10 m^2 and 0.15 m^2 respectively.

8.6 MODEL STUDY

Experimental study was carried out on a perspex model of the proposed second Hooghly Bridge at Calcutta. The experimental work included determination of modulus of elasticity of perspex and the spring steel wiresand fabrication and calibration of load cells .. The prestressing of the wires was done in the model. The prestress which could actually be provided came to within ± 10% of the desired value. The load deflection relationship was in general found to be linear except when main span was loaded in which case the characteristic was seen to be hardening type. This corroborates qualitatively with the non-linear analytical results. The theoretical values of deflections are found to be approximately 5% smaller than the experimental values and those of cable tensions 5 to 25% smaller than the experimental values. As a whole it may be concluded that the theoretical and experimental results are in close approximation.

8.7 DESIGN CHARTS

The design charts in the dimensionless form, incorporating the effect of various parameters have been prepared and presented. These charts are useful in making the preliminary design estimates. The side to main span ratios of 0.35, 0.40, 0.45 and 0.50 and the tower height to total span ratios of 0.075, 0.100 and 0.125 are considered. Two values of lengths of central panel are taken. In one case the length of central panel is taken as 0.2 times the main span and in the second equal to the length of other panels of the main span. Two values of the parameter λ (62000, 83000) are considered.

8.8 PROBLEMS FOR FURTHER RESEARCH

In the present study an attempt is made to cover the several aspects of study of cable stayed bridges. Many other problems could however not be investigated and there is need to carry out further research work in the following areas.

The analysis of radiating type bridges during erection is carried out. It may be extended to other bridge forms. With the sequence of prestressing used the girder moments are reduced to a minimum in the final stage. However, the moments in the intermediate stages are still large. An attempt should be made to reduce the girder moments in intermediate stages too, by improving the prestressing sequence.

Certain other areas of further research could be:

1. Extent of nonlinearity due to sag of cables and beam-column interaction of the members.

2. Effect of interaction between the foundation soil and the foundation.

3. Dynamic behaviour of the bridge due to wind and earthquake loads.

4. Behaviour of bridge under static loads.

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APPENDIX - 'A'

SCALING OF MODEL

A. 1 CHOICE OF MODEL SCALE

A scale factor of 200 is chosen with the considerations of facilities of testing equipments available, cost of the model and time required in the fabrication of the model. It is felt that the perspex is suitable for the deck, towers etc and the spring steel for the cables. The linear dimensions are scaled geometrically. The details of the scaling of the dimensions are given below:

A.1.1 Span Lengths

$$L_m = \frac{L_p}{n} \qquad \ldots \quad (A.1)$$

where, L_m is the span length in the model, L_p is the span length in the prototype and n is the scale factor.

A.1.2 Stiffness Ratio for Deck and Tower

$$\frac{K_p}{K_m} = \frac{1}{n^3} \; \frac{E_{p2}\, I_{p2}}{E_m\, I_{m1}} \qquad \ldots \quad (A.2)$$

$$= \text{Stiffness ratio for substructure}$$

$$= n\,\frac{E_p}{E_m}$$

where, E_{p2} is the modulus of elasticity of steel in deck or tower, I_{p2} is the moment of inertia of the steel member in deck or tower in prototype, E_m is the modulus of elasticity of the model material, and I_{m1} is the moment of inertia of the model member. Equation (A.2) gives the value of I_{m1},

$$I_{m1} = \left(\frac{1}{n^4} \right) \left(\frac{E_{p2}}{E_{p1}} \right) I_{p2} \qquad \ldots (A.3)$$

where, E_{p2}/E_{p1} is the ratio of modulus of elasticity of steel to concrete of substructure, assumed as 14. Equation (A.3) will ensure same stiffness ratio in deck and tower. The depths of sections are so adjusted in the model deck as to achieve the desired stiffness ratio.

A.1.3 Stiffness Ratio for cable

$$\frac{K_p}{K_m} = \frac{E_{p3}\, A_{p3}\, L_m}{E_{m2}\, A_{m2}\, L_p} \qquad \ldots (A.4)$$

= Stiffness ratio for substructure

$$= n \frac{E_p}{E_n} \qquad \ldots (A.5)$$

where, $E_{p3}\, A_{p3}$ and $E_{m2}\, A_{m2}$ represent the axial rigidity of cable in prototype and model respectively, L_p and L_m represent the length of cable in prototype and model respectively. The cross-sectional area of cable in model is obtained from Eq. (A.5),

$$A_{m2} = \left(\frac{A_{p3}}{n^2} \right) \left(\frac{E_n}{E_p} \right) \left(\frac{E_{p3}}{E_{m2}} \right) \qquad \ldots (A.6)$$

The model is designed such that the ratios of K_p / K_m and M_p / M_m are kept same for every component of the bridge. Where M_p and M_m are the masses of components in the prototype and the model. The mass of the deck is increased by pasting the small brass cylinders. The mass ratio is important for the dynamic tests.

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