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Effective length factors for solid round chord members of guyed towers.

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EFFECTIVE LENGTH FACTORS FOR SOLID ROUND CHORD MEMBERS OF GUYED TOWERS

by

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ABSTRACT

Chord members of twenty-five all-welded guyed-latticed communication steel tower sections were tested in the Structural Engineering Laboratory of the University of Windsor to determine the effective length factors of the chord members. Two different manufacturers, viz., Pirod Inc., Plymouth, Indiana, and ERI Inc., Chandler, Indiana, provided the test specimens. All tower sections were fabricated from solid round members and were triangular in cross section. Tower sections provided by ERI were 4.57 m (15.0 ft.) long with continuous diagonal bracings welded to the chord members, while those provided by Pirod were 6.09 m (20.0 ft.) long with the diagonal bracings cut and welded to the chord members. The diameters of the chord members varied from 38.1 mm (1.5 in.) to 69.85 mm (2.75 in.) while the diameters of the diagonal bracings varied from 12.7 mm (0.5 in.) to 22.3 mm (0.875 in.).

The tower sections were tested in a horizontal position. One chord member of the tower was cut and tested by applying a load at its center while the diagonal bracings remained attached to the chord members. Deflections were recorded manually while the applied load was recorded using a data acquisition system. The load-deflection data were used to calculate the effective length factors for the chord members which were found to be varying from 0.95 to 0.99. Good agreement was observed between the experimental deflections at the center of the chord member and those obtained from the computer analysis software package ABAQUS.

The stiffness contribution of diagonal bracings to the ends of the chord members was also computed numerically by structural analysis package STAAD/Pro and the effective length factors were calculated using the equilibrium equation given by the structural stability research council (SSRC, 1976). The effective length factor calculated by this method varied from 0.93 to 0.99. Based on the experimental and numerical results, it appears reasonable to assume an effective length factor of 1.0 in actual design practice.

DEDICATED
TO MY
LOVING PARENTS
DR. ABDUL HALIM QURESHI
AND
DR. SAKINA QURESHI

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NOTATION

A	=	Cross-sectional area
c	=	Uniformly distributed loaded length
D	=	Diameter of cross section
E	=	Young's modulus of elasticity
G_L	=	Relative joint stiffness ratio, lower
G_U	=	Relative joint stiffness ratio, upper
I	=	Moment of inertia
I_c	=	Moment of inertia of column/chord member
I_g	=	Moment of inertia of girder/diagonal bracing
K	=	Effective length factor
l, L_c	=	Panel length of column
L_g	=	Length of girder
M	=	Moment at ends of chord member
M_{FEM}	=	Fixed end moment
M_R	=	Restraining Moment
P	=	Concentrated load applied
q	=	intensity of load applied
δ	=	Difference in Deflection
Δ_E	=	Experimental deflection
Δ_X	=	Calculated Deflection
θ	=	Angle of Rotation

CHAPTER ONE

INTRODUCTION

1.1 GENERAL

With the world entering into the 21st century, the demand for an effective and useful telecommunication system is growing. This need has resulted in an increase in the production and development of telecommunication tools. Some of these telecommunication tools are mobile phones and pagers. To transmit and receive the signals properly, antennas are used, which are either parabolic or pole type. These antennas are usually mounted on steel communication towers.

1.2 CLASSIFICATION OF COMMUNICATION TOWERS

From the structural point of view, communication towers can be classified into three types:

(a) Monopoles, which are cantilevered tubes with heights up to 70 m,

(b) Self-supporting towers (Figure 1.1), commonly used for heights up to 120 m, and

(c) Guyed towers (Figure 1.2) which have been utilized for taller structures up to 620 m.

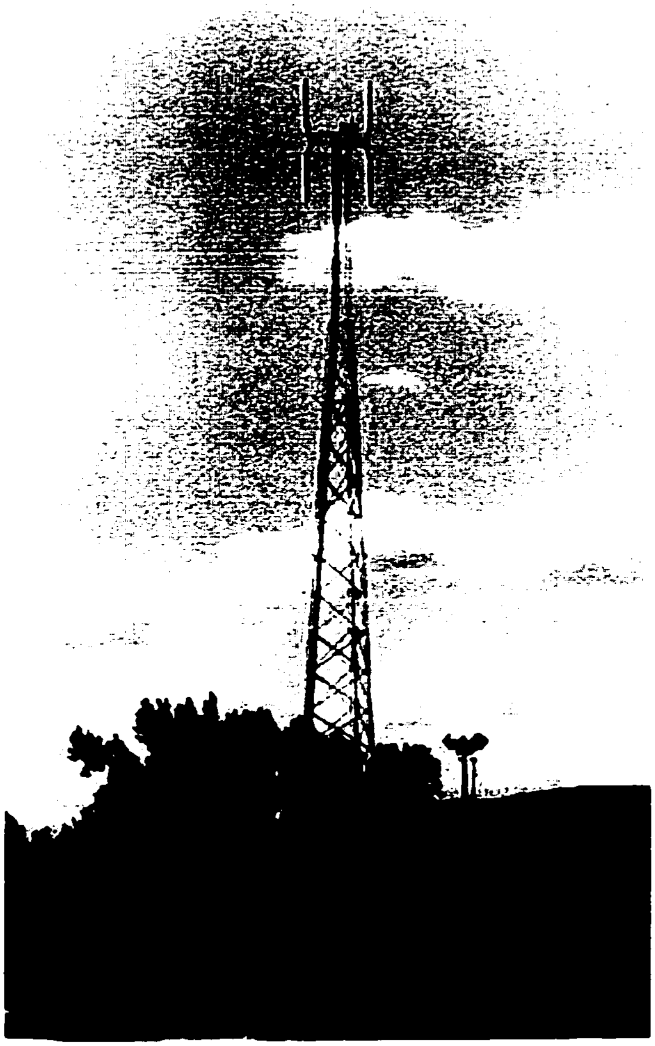


Figure 1.1 Self-supporting tower

(Source: Pirod Inc.)

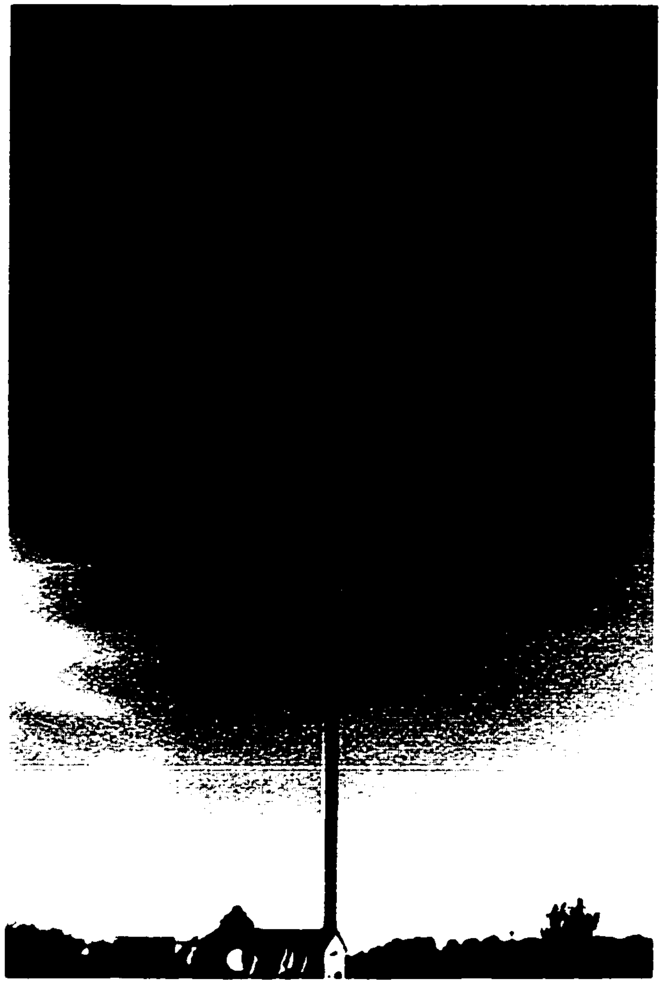


Figure 1.2 Guyed Tower

(Source: Pirod Inc.)

Guyed towers require more land area than the self-supporting ones. This is because of the guys that provide lateral support to the structure. These guys are steel cables made especially for this purpose. They are hooked up firmly with guy anchors that are anchored in the nearby land, thus occupying a huge surrounding area. Because of this disadvantage, guyed towers are generally not recommended for installation in densely populated residential areas or such places where availability of adequate space is a problem. Usually the number of guy levels holding a tower ranges from three to five (Wahba, 1999).

1.3 STRUCTURAL CONFIGURATION

A guyed tower consists of a mast usually of a constant triangular cross section. These towers have a latticed structure with all the joints welded or bolted. The triangular mast of a guyed tower has three vertical members (chords). These chord members are welded or bolted with horizontals and diagonals (bracing) to form a latticed structure. All the members are usually solid rounds with varying diameters. Tubes and angles are also sometimes used for these chords and bracings.

A guyed tower is fabricated in sections and is erected by mounting these sections one on top of the other in the field by a special method used to build tall communication structures with antennas fixed on it. Helicopters are also occasionally used to carry out this installation operation.

1.4 NEED FOR INVESTIGATION

The effective length factor of the chord members of triangular guyed-steel-communication towers is the main topic of this study. To the best of author's knowledge, most of the experimental investigation carried out so far on guyed-latticed communication steel towers, with reference to finding out the effective length factors, is on cross braced diagonals. So far, no investigation has been carried out on solid round chord members of these towers. Thus, there is a strong need to experimentally determine the effective length factors of chord members that are primary structural components of a steel communication tower.

Section 6.2 of the Canadian Standards Association S37-94, "Antennas, Towers and Antenna-Supporting Structures," (CSA 1994), deals with members under compression. It gives the effective length factor for the leg members of towers as unity. Most practicing engineers frequently use this value. However, there are some designers who feel that the bracing members provide rotational rigidity to the chord members and so take the effective length factor as 0.8.

Thus, there is a need to carry out the experimental investigation of the chord members and find out the value of the effective length factor. The research is based on the hypothesis that the diagonal bracings provide rotational restraint to the chord member with which they are attached, thus, resulting in an effective length factor which is less than unity.

1.5 OBJECTIVE OF THE PRESENT RESEARCH

The objective of the research is to determine, experimentally and theoretically, the effective length factors for solid round chord members of guyed-latticed steel communication towers.

1.6 OUTILINE OF THE THESIS

Chapter Two, BACKGROUND OF THE RESEARCH AND REVIEW OF LITERATURE, includes brief review of available research material on communication towers, effective length factors of members under compression and the recommendations given by different codes, standards and specifications in this regard.

Chapter Three, EXPERIMENTAL INVESTIGATION, describes in detail the experimental investigation carried out on chord members of tower specimens in the Structural Engineering Laboratory of the University of Windsor. The procedure is explained step-by-step and is accompanied by photographs taken at the time of experimentation.

Chapter Four, ANALYSIS OF LOAD - DEFLECTION DATA, deals with the analysis of the data obtained through experiments. The rotational stiffness at the ends of the chord members has been calculated from the load-deflection data and effective length factor of the chord member is determined. The deflection data of the chord member observed experimentally is also compared with the results obtained from the computer software ABAQUS (Version 5.8, 1998).

Chapter Five, NUMERICAL INVESTIGATION, describes the process for the determination of the relative joint stiffness ratio known as G-factor, by using structural analysis software STAAD/Pro (1998). This factor is used subsequently to obtain the effective length factor from the equilibrium equation (Equation of the Nomograph).

Finally, Chapter Six, CONCLUSIONS, gives the summary of the work done, and the conclusions reached.

CHAPTER TWO

BACKGROUND OF RESEARCH AND REVIEW OF LITERATURE

2.1 GENERAL

Guyed transmission towers are highly indeterminate three-dimensional structures with a relatively complex structural composition. These structures are among some of the tallest in the world. They stand as high as 1500 feet or 500 m above ground level. Due to the peculiar structural composition of guyed towers, much work has been done in this area by many researchers.

One general assumption that is usually made in analysis of guyed towers is that the different structural members meet at a single point and the joints have a pinned effect. However, this assumption is not satisfied in usual fabrication. When the joint is not pinned, there are chances that secondary stresses will arise in the member that may have a significant influence on the critical load. This discrepancy between design assumption and actual fabrication may undermine the ultimate load carrying capacity of the tower.

In the case of steel communication towers, the panel joints (end conditions) are not simple pin connections. The diagonal members of each panel provide some resistance to a moment. This restraining moment shifts the location of the points of inflection from the end points to inside, thus reducing the effective length of the member. The amount of reduction is proportional to the resistance provided by the restraining members.

In triangulated frame structures (trusses), the loads are usually applied at the joints. Thus if the joints are truly pinned, then the members are axially loaded. Deflections of the joints and the truss as a whole are caused by the axial deformation of the members. The angles between members meeting at a joint also change because of these deformations. If the members are connected together at the joints by welds or bolts, the angle changes cause secondary bending stresses. These have little effect on the buckling strength of the truss members. Because of the local yielding of extreme fibers of the members near the joints, as the truss is loaded to ultimate, the secondary moments gradually dissipate. (Galambos, 1988).

2.2 JOINT EFFECT IN TRANSMISSION TOWERS

Knight and Santhakumar (1993) studied the joint effects on behavior of transmission towers. A full-scale quadrant of the lowermost panel of a transmission tower designed as a pin-jointed truss was tested according to ICP (1978). The behavior of the tower was observed under normal loading conditions. The lowermost panel was chosen for experimental observation as the vertical, transverse and longitudinal loads were assumed to be the maximum on that panel. It was concluded that the failure of the chord members depends on the axial forces as well as the moments generated because of forces in the secondary members. Thus, it was concluded that the consideration of moments introduced by the secondary members is a necessity in order to arrive at a realistic estimate of the failure load of the tower and hence the consideration of joint effect is very important as it may result in a premature failure or inappropriate analysis of the whole tower.

2.3 EFFECTIVE LENGTH FACTOR IN BRACED FRAMES

With reference to the design of steel frames, the effective length concept is extensively used for finding out the effects of the interaction of other members (beams, columns or other) on the member under compression in a frame. Much work has been done with reference to the analysis of columns in partially and fully restrained frames.

Kishi, Goto and Komuro (1995) observed the effective length factor for columns in braced and flexibly jointed frames. Their discussion was with reference to AISC-LRFD specification (AISC 1993) which states that in order to design a partially or fully restrained frame, the bending moments of members are to be obtained estimating the second-order effects (P-Δ and P-δ effects) and the effective length factor-K of columns, considering the nonlinear moment-rotation characteristics of semi-rigid connections. They used the alignment chart approach to derive the governing equation for determining the columns K-factor in braced and flexibly jointed frames. They showed that alignment chart can be used to find the K-factor for a column by modifying the relative stiffness factors.

Duan and Chen (1988) studied the effective length factor for columns in braced frames. They derived the general effective length factor equations for columns in braced frames corresponding to five different boundary conditions of top and bottom columns. They concluded that the far end conditions of the columns above and below the column have significant effect on the effective length factor K of the column being investigated. A direct use of the alignment chart, without modifications, results in an effective length factor that can be either too conservative or even unsafe depending on the boundary conditions of the columns. They gave a modified improved alignment chart procedure which included the usual rigid far ends of the top and bottom column as a special case and also considered the cases of fixed or hinged far ends of the top and bottom columns.

Fraser (1983) gave a method for evaluating the effective length factor in braced frames. It comprises of an iterative procedure and is helpful in the analysis of structures in which the flexural stiffness of the restraining members is significantly reduced by the presence of axial forces. The whole procedure is much simplified by the use of linearized stability functions expressed in terms of effective length factors.

Cheong-Siat-Moy (1997) analyzed the possibility of the design charts used by practicing design engineers giving unconservative values for effective length factors for braced frames because they assume that the lateral restraints are infinite. The basis for this infinite value is the assumption that there is no sway in the member or in other words it is sway-prevented. It was concluded that the use of the assumed-sway-prevented K-factor in practical braced frames could sometimes be too unconservative.

Some researchers, taking into consideration the complex and indefinite procedure of determining the K-factor, went as far as proposing the design of steel frames without the consideration of effective length. White and Hajjar (1997) were among those researchers. Limits have been suggested in their paper for use of K=1. Also AISC-LRFD specifications are discussed with reference to neglecting P-Delta moments in design. General equations are presented in the paper that give the error in the AISC-LRFD beam-column interaction equations associated with the use of K=1. The influence of key variables on the error is analyzed. Suggestions are given when the design of steel frames by AISC LRFD may be based on K=1. The paper ends by comparing design strengths with and without effective length to the result from elastic-plastic hinge and rigorous plastic zone analysis for several 'maximum error' examples. The whole discussion provides an assessment of the accuracy of upper-bound error estimates, and of the implications of using K=1 relative to the theoretical inelastic frame behavior. The recommendations and discussions are applicable for any type of steel frame that includes frames with fully or partially restrained connections, and unbraced or partially braced frames.

Wood (1974) studied the effective length of columns in multi-story buildings. He gave comprehensive design charts for effective length of columns with any local degree of end restraint, both for sway and non-sway conditions.

2.4 THE G-FACTOR AND EFFECTIVE LENGTHS OF COLUMNS

Framed members under compression interact with the restraining beams at member ends. They also come in contact with other members that are under compression above and below the member under consideration. This interaction is due to the connecting horizontal beams that are commonly in contact with both compression members. These interactions are complex and peculiar in behavior. It is, however, a common practice that to avoid more complexity in such analysis the compression members are studied as isolated, with the end restraints defined in terms of simple stiffness ratios, known as G-factors. The issue of the accuracy of the G-factor has been addressed by many researchers since it plays an important part in the overall design and assumptions made for a structure.

Bridge and Fraser (1987) gave an improved G-factor method for evaluating effective length of columns with reference to the Nomograph that is widely used by practicing engineers to find out the effective length factor. It requires the engineer to evaluate G at each end of the column. The value of G should always be positive. In the case of sway prevented structures, however, the effective length factors are always less than unity. It is practically possible that columns may have values of K greater than unity that corresponds to a value of G that is negative. The paper concludes that the failure to include the effects of axial load in adjacent members restraining a critical buckling column could result in an error in determining the elastic-buckling load. Such a calculation can be dealt with if the axial forces in restraining beams and in adjacent columns are taken care of properly. Such a procedure can be achieved by incorporating the negative G-factors. Hence, an improved G-factor method was given which takes into account all these factors.

Hellesland and Bjorhovde (1996) discussed the methods for determining effective lengths of members under compression in a frame in terms of exact results and general principles of mechanics of buckling. The conventional G-term (relative joint stiffness ratio) has been shown to be inaccurate and conceptually flawed. A novel concept of a restraint demand factor is introduced. This is done in order to allow for improved development of vertical interaction, which also includes effective length predictions

Hajjar and White (1995) summarized the objectives and contents of ASCE committee report entitled "Effective Length and equivalent imperfection approaches for assessing frame stability: Implications for American Steel Design". The discussion is with reference to the procedure used commonly for column design in the United States that is based primarily on the Nomograph effective length approach. The report discussed the procedure for the calculation of effective length within partially restrained (or semi-rigid) framing. Among other key issues, one was the issue of the validity of the effective length concept for framing in which the restraining elements, that includes beams and their connections, are relatively flexible and show significant non-linearity. For use in determining effective length, the selection of proper connection stiffness is also discussed, along with concepts for calculation of effective lengths in framed structures.

Hellesland and Bjorhovde (1996) also proposed a method which involved post-processing of effective lengths from isolated column analysis to arrive at improved, weighted mean values. As such, the method is termed as the "method of means". This method involves post-processing of effective lengths from isolated columns analyses to arrive at improved, weighted mean values. The approach is applicable to braced and to a range of unbraced frames.

Duan and Chen (1989) analyzed the effective length factor for columns in unbraced frames with reference to the specifications of AISC (1986) which makes use of the alignment charts to determine effective length factor for columns in both types of frames, braced and unbraced. It was found that the effective length factor of a framed column is not only dependent on the relative bending stiffness ratio of the jointed columns and girders, i.e., the G-factors at the ends of its unbraced length in an unbraced continuous frame, but is also dependent on the boundary conditions of far ends of restraining columns.

Kishi, Chen and Goto (1997) analyzed the effective length factor of columns in semi-rigid and unbraced frames. The paper is with reference to the engineering practice of evaluating the columns in frames with rigid and semi-rigid connections whereby the estimation of the effective length factor (K-factor) is necessary. This estimation is done considering the effects of the nonlinear moment-rotation characteristics of beam-to-column connections. The paper states that with a proper evaluation of the tangent connection stiffness for semi-rigid beam-to-column connections at buckling state and with the introduction of the modified relative stiffness factors, the alignment chart in the present American Institute of Steel construction – Load and Resistance Factor Design specification can be used to find the corresponding K-factor for columns in semi-rigid frames

2.5 CODES, STANDARDS AND SPECIFICATIONS

2.5.1 CSA S37-94 "ANTENNAS, TOWERS, AND ANTENNA-SUPPORTING STRUCTURES"

Section 6.2.1.1 of the Canadian Standards Association S37-94 "Antennas, Towers and Antenna-Supporting Structures", 1994, gives the effective length factor for leg members of towers as unity.

2.5.2 CAN/CSA-S16.1-94 "LIMIT STATES DESIGN OF STEEL STRUCTURES"

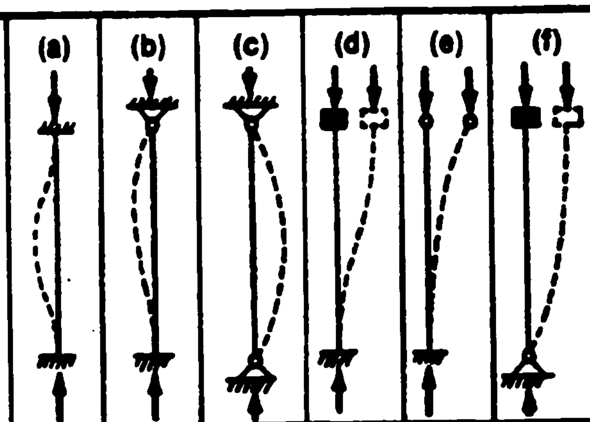
The standard states about the effective length of columns as:

"The effective length, KL, may be thought of as the actual unbraced length, L, multiplied by a factor, K, such that the product, KL, is equal to the length of a pin ended column of equal capacity to the actual member. The effective length factor, K, of a column of finite unbraced length therefore depends on the conditions of restraint afforded to the column at its braced location. A variation in K between 0.65 and 2.0 would apply to the majority of cases likely to be encountered in actual structures. Figure 2.1 illustrates six idealized cases in which joint rotation and translation are either fully realized or non-existent".

The standard also gives a Nomograph (Figure 2.2) which is based on the assumption that all the columns in the portion of the framework considered reach their individual critical load simultaneously. This may be used to determine the effective length factors for in-plane behavior of compression members of trusses designed as axially loaded members even though the joints are rigid. In this case, there should be no in-plane eccentricities and all the members of the truss meeting at the joint must not reach their ultimate load simultaneously. Further, the standard says that if it cannot be shown that all members at the joint do not reach their ultimate load simultaneously, then the effective length factor of the compression members shall be taken as 1.0.

2.5.3 AISC-LRFD "LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS"

Chapter E of the LRFD specification deals with columns and other compression-prismatic members that are subjected to axial compression through their centroidal axis. Section E1 deal with the Effective Length Factor – K and states that for structural members under compression it should not be taken as less than unity.

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.0	2.0

End condition code		
	Rotation fixed	Translation fixed
	Rotation free	Translation fixed
	Rotation fixed	Translation free
	Rotation free	Translation free

Figure 2.1 Effective Length Factors for six idealized conditions

(Source: SSRC, 1976)

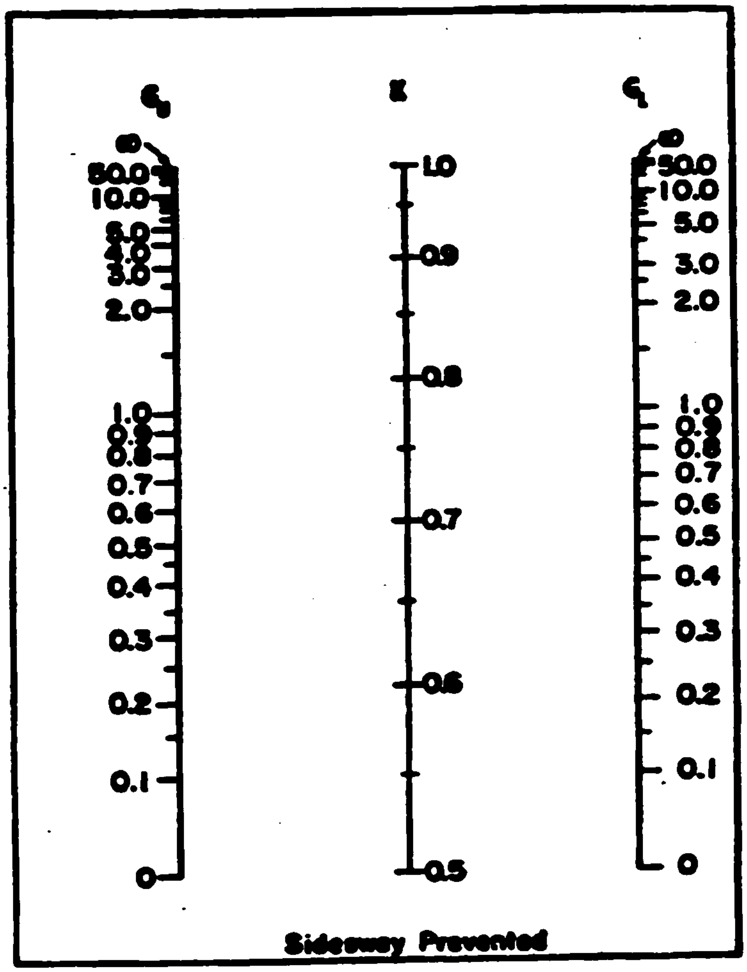


Figure 2.2 Nomograph

(Source: SSRC, 1976)

2.5.4 EUROCODE 3 PART 3.1: 1997 "DESIGN OF STEEL STRUCTURES TOWERS, MASTS AND CHIMNEYS –TOWERS AND MASTS,"

Sub-clause 5.6.2.2 of the code refers to single members and reads as:

"The following rules should be used for single angles, tubular sections or solid rounds where used for chord sections. For chords or chords with axial compression braced symmetrically in two normal planes, or planes 60^0 apart in the case of triangular structures, the slenderness should be determined from the system length between nodes.

"Where bracing is staggered in two normal planes or planes 60^0 apart in the case of triangular structures, the system length should be taken as the length between nodes on one face. For angle sections the radius of gyration about the minor axis should be used to calculate the slenderness".

Clause 5.7 refers to effective slenderness factor K and states that if the chord members are solid rounds then the value of K should be taken as 1.0. This value is for both types of towers having symmetrical or asymmetrical bracings.

CHAPTER THREE

EXPERIMENTAL INVESTIGATION

3.1 GENERAL

Investigations were carried out on fifteen specimens provided by Electronics Research, Inc., Chandler, Indiana, USA, and ten specimens provided by Pirod Inc., Plymouth, Indiana, USA. The specimens were actual tower segments fabricated by these two companies. The specimens provided by ERI Inc. were 4.57 m (15.0 feet) long while those provided by Pirod were 6.09 m (20.0 feet) long. Most of the specimens were representative samples of actual tower segments but some were specially fabricated for these investigations.

3.2 DETAILS OF SPECIMEN

3.2.1 ERI SPECIMENS:

Fifteen specimens were provided by Electronic Research Institute, Inc. Three different chord sizes, 69.9, 50.8 and 38.1 mm, were used with three different diagonal sizes, 15.9, 14.3 and 12.7 mm, respectively. The diagonal bracings were continuous and bent at the point of welding with the chord members. Refer to Figure 3.1.

Figure 3.1 ERI Tower – Weld arrangement

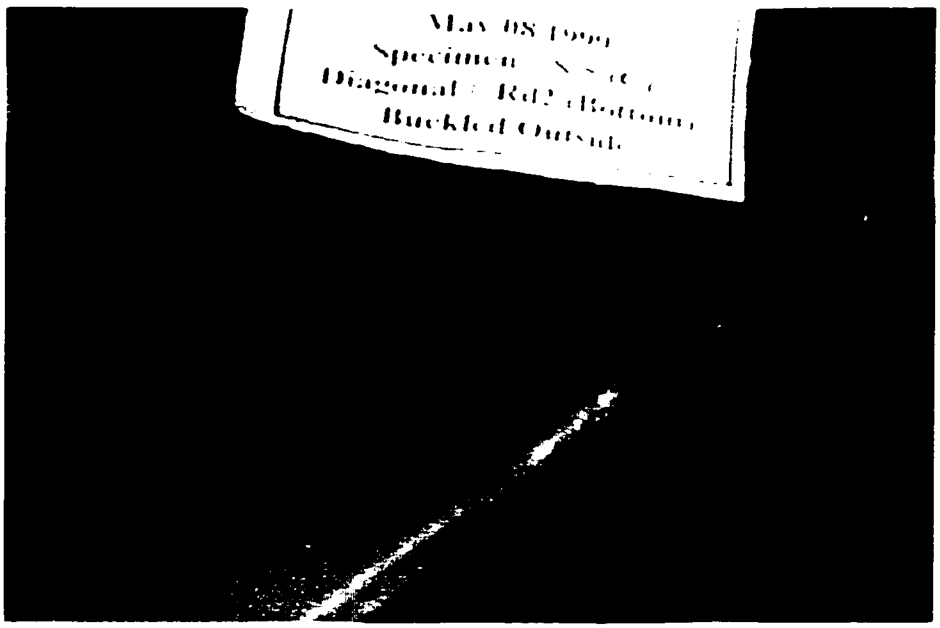


Figure 3.2 Pirod Tower – Weld arrangement

None of the specimens were galvanized. All the chord members were made from standard solid rounds with different diameters. The total length of each tower section provided by ERI Inc. was 4.57 m. Each tower section had six panels. All sections were of triangular cross section with chord, and diagonal members connected together by welding.

3.2.2 PIROD SPECIMENS

Ten specimens were provided by Pirod Inc. Three different chord sizes, 57.2, 50.8 and 38.1 mm, were provided with four different diagonal bracing sizes, 22.2, 19.1, 15.9 and 22.2 mm. The diagonal bracings were cut and welded to the chord member as shown in Figure 3.2. The total length of each tower section was 6.09 m. Figure 3.3 shows three Pirod towers section lying in the laboratory. Each section had eight panels. Refer to Table 3.1 for the details of the tower sections.

3.3 TEST SETUP

All the tower specimens were tested in a similar fashion at the Structural Engineering Laboratory of the University of Windsor. They were first placed in a horizontal position resting on two roller supports, 1219 mm long and 152.4 mm in diameter. One panel from the bottom chord was chosen for the test and was loaded by the hydraulic jack that was attached to the loading frame in the laboratory

TABLE 3.1 DETAILS OF TOWER SECTIONS

S.No.	Supplier	Specimen I.D	Total Length mm	No. of Panels	Panel Length mm	Face Width mm	Dia. of chords mm	Dia.of diagonals mm
1	ERI	S-1 (A)	4572	6	762	914	38.1	12.7
2	ERI	S-1 (B)	4572	6	762	914	38.1	12.7
3	ERI	S-2 (A)	4572	6	762	914	38.1	12.7
4	ERI	S-2 (B)	4572	6	762	914	38.1	12.7
5	ERI	S-2 (C)	4572	6	762	914	38.1	12.7
6	ERI	S-3 (A)	4572	6	762	914	50.8	14.3
7	ERI	S-3 (B)	4572	6	762	914	50.8	14.3
8	ERI	S-4 (B)	4572	6	762	914	50.8	14.3
9	ERI	S-4 (C)	4572	6	762	914	50.8	14.3
10	ERI	S-5 (A)	4572	6	762	914	69.9	15.9
11	ERI	S-5 (B)	4572	6	762	914	69.9	15.9
12	ERI	S-5 (C)	4572	6	762	914	69.9	15.9
13	ERI	S-6 (A)	4572	6	762	914	69.9	15.9
14	ERI	S-6 (B)	4572	6	762	914	69.9	15.9
15	ERI	S-6 (C)	4572	6	762	914	69.9	15.9
16	Pirod	P1 (A)	6096	8	711	914	38.1	15.9
17	Pirod	P1 (B)	6096	8	711	914	38.1	15.9
18	Pirod	P2 (B)	6096	8	711	914	38.1	15.9
19	Pirod	P3 (A)	6096	8	711	914	50.8	19.1
20	Pirod	P3 (B)	6096	8	711	914	50.8	19.1
21	Pirod	P4 (A)	6096	8	711	914	50.8	19.1
22	Pirod	P6 (A)	6096	8	711	914	57.2	22.2
23	Pirod	P6 (B)	6096	8	711	914	57.2	22.2
24	Pirod	P8 (A)	6096	8	711	914	38.1	22.2
25	Pirod	P8 (B)	6096	8	711	914	38.1	22.2

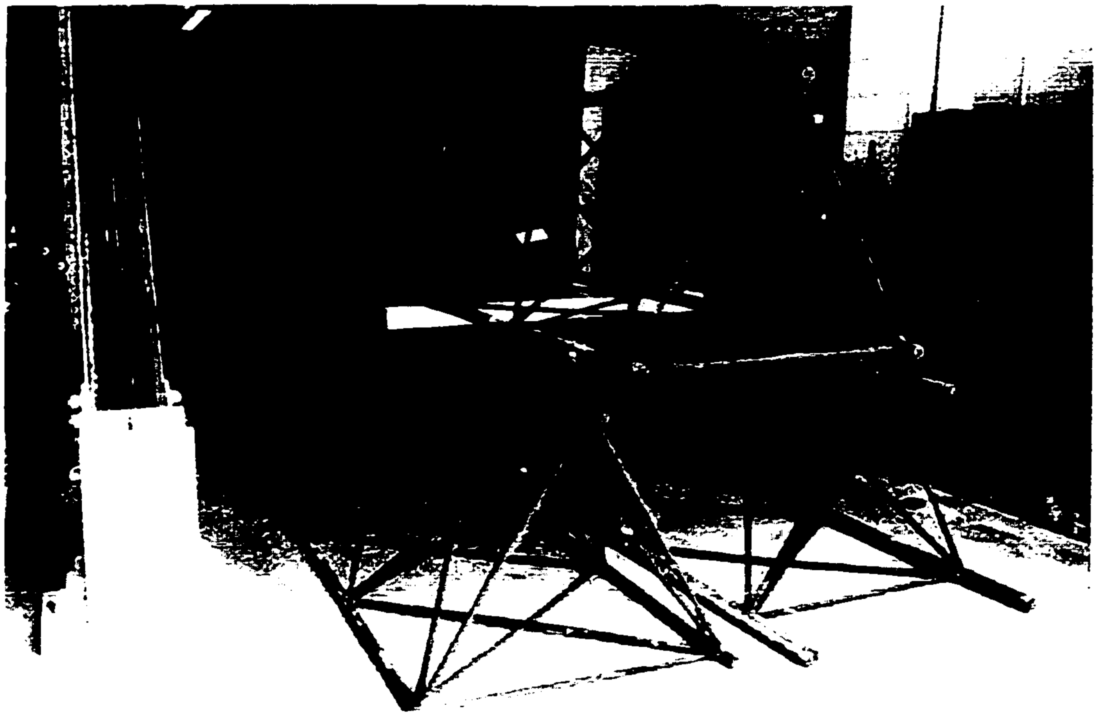


Figure 3.3 Pirod Towers lying in the Laboratory

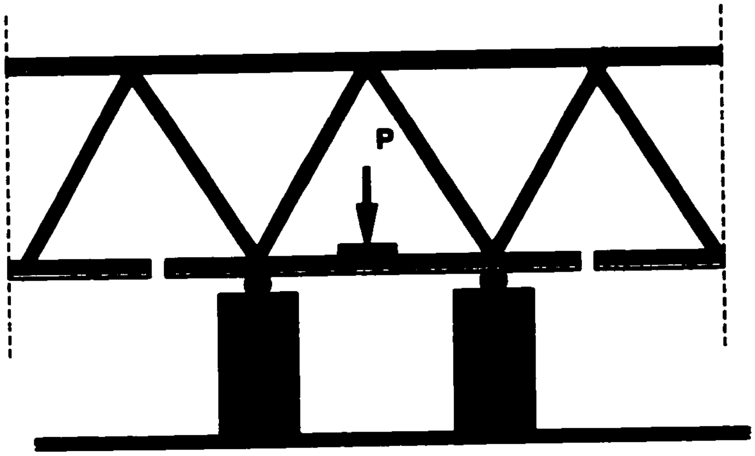


Figure 3.4 Chord member cut and loaded

The chord member was cut beyond its panel points/joints on either side of the test section as shown in Figure 3.4. The diagonal bracings, however, remained attached to the chord member. The cutting was done either with the help of a welding torch or an electric power saw. The panel, being isolated from both sides , was then simply supported on either of its sides on two small rollers, 127 mm long and 38.1 mm in diameter. Load was applied on that panel at the mid-point through a loading block as shown in the test setup in Figure 3.5

3.3.1 SUPPORT ASSEMBLY

A support assembly that consisted of several built up steel sections supported the tower at the ends. These steel sections were assembled together to furnish enough height to be able to test the specimen using the testing frame available in the laboratory.

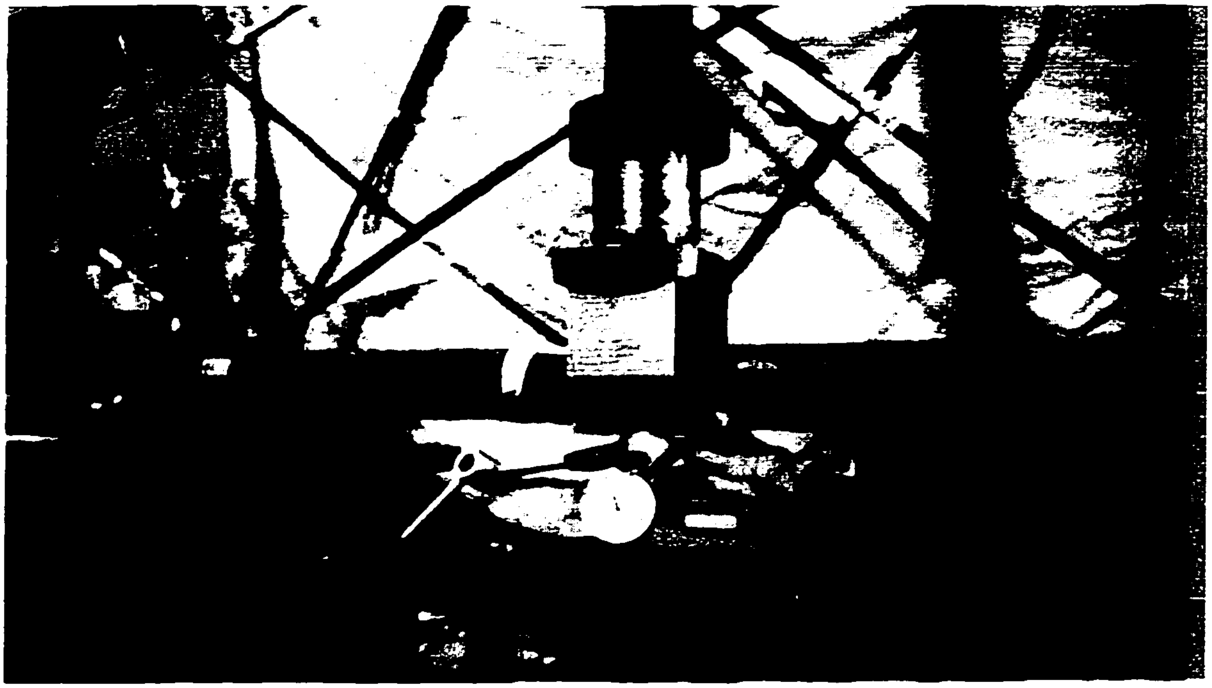


Figure 3.5 Test setup

3.3.2 LOAD APPLICATION

The load was applied through a hydraulic jack attached to the testing frame. A 448 kN (100 kip) load cell was screwed to the bottom of the hydraulic jack. An extension was provided to the load cell in the shape of a long solid round cylindrical column, 76.2 mm in diameter and 1220 mm long. This extension was needed to furnish the difference between the heights of the load cell attached to the roof beam and the chord member of the tower specimen. Refer to Figure 3.6.

The load was applied through loading blocks that were specially machined and grooved to sit properly on the chord members having varying diameters. The blocks measured 101.6x76.2x192.08 mm, 120.65x76.2x192mm and 139.7x76.2x192 mm to sit properly on the chords having diameters 38.1, 50.8 and 69.9 mm respectively. In between the load application column and the load block, a ball-and-socket joint, as shown in Figure 3.5, was introduced in order to take care of any eccentricity of the applied loading. The load was applied using a mechanical pump connected to the hydraulic jack. To measure the applied load, a data acquisition system was used which gave readings in kilo-newtons.

3.3.3 DATA ACQUISITION SYSTEM

An automatic data acquisition system was used to record the load applied by the hydraulic jack to the chord member. This was a Datascan 7000 series of Data Acquisition Modules. A Type 7321 measurement processor was used that provided directly 8 analog

Figure 3.6 Test setup showing the loading column and the support assembly

inputs, including channel excitation and full local channel expansion capability. A type 7021 analog expansion scanner module was used which provided 8 analog input channels per scanner, with excitation for transducers. In addition to that, a type 7036 digital expansion scanner module was used which provided 8 digital input and 8 digital output channel capability. One among the 40 channels was used to know the applied load on the chord member. The system was hooked up with a 486 DX2 host computer (Figure 3.7). Proper settings were made and the load cell was configured to the system before the start of the actual test.

3.3.4 TESTING OF CHORD MEMBER

A small load was first applied and released to make sure that the whole assembly is perfectly seated and well placed. In order to measure the deflection due to the applied loading, a dial indicator was fixed exactly below the point where the load was being applied to the chord member. The indicator was firmly fixed to the adjacent steel section, which was lying on the floor. No movement was permitted to ensure undisturbed readings of the dial indicator once the actual testing procedure was started.

The load was applied in smaller increments. To make sure that the material does not yield, the maximum load applied was kept within the elastic range of the chord member. The maximum load applied varied for each chord depending upon its diameter. For each load increment, the dial gauge reading and the corresponding applied load were recorded.

Load deflection data were thus obtained for all these specimens. Graphs for all twenty-five specimens are given in Appendix A.

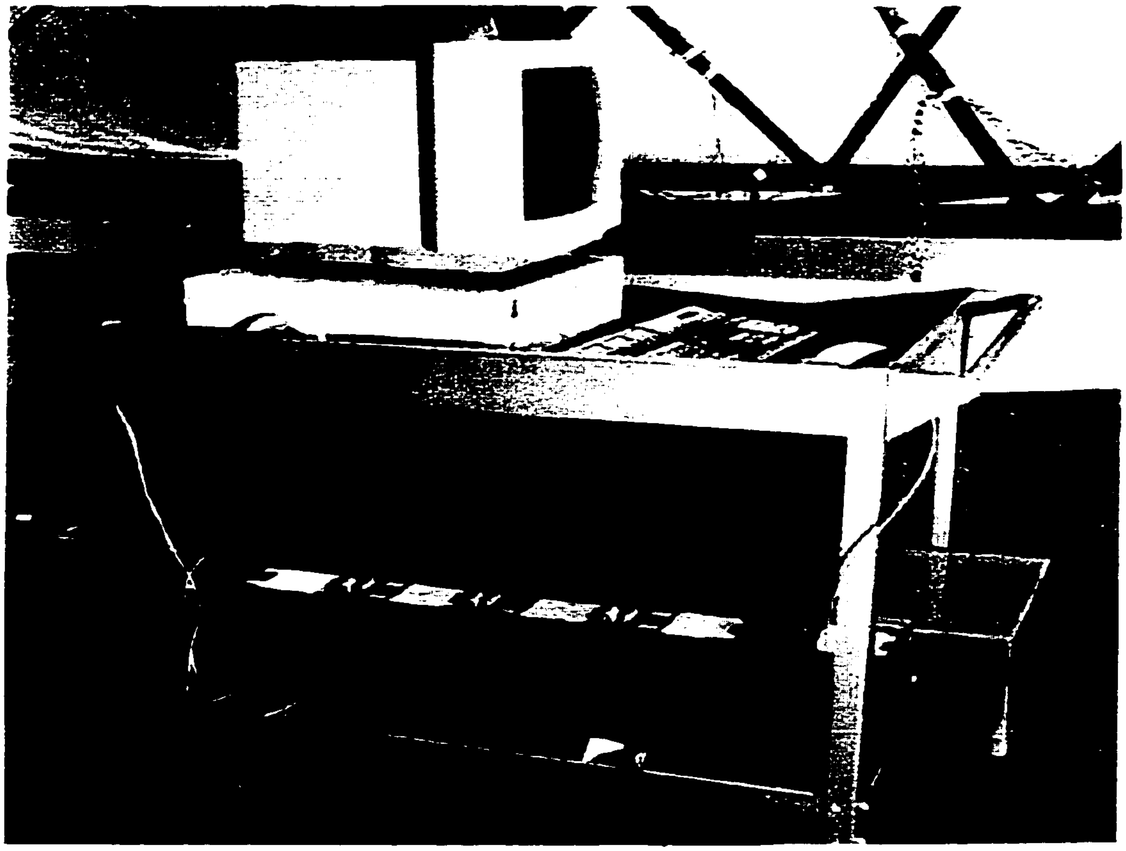


Figure 3.7 Data acquisition system

CHAPTER FOUR

ANALYSIS OF LOAD-DEFLECTION DATA

4.1 GENERAL

The load deflection data gathered through the experimental procedure were analysed to determine the effective length factor K. The deflections of the chord member of the tower were also compared with the values obtained numerically by using the commercial software package ABAQUS.

4.2 CALCULATION OF CHORD ROTATION AT THE ENDS

The deflections, Δ_x, of the chord member of the tower were first calculated assuming both ends to be completely fixed as shown in the Figure 4.1. The downward deflection at mid-span of the beam, fixed at both ends and loaded with a uniformly distributed load of length 'c' at its center is given by:

$$\Delta_x = \frac{qcl^3}{384EI}(2 - 2\gamma^2 + \gamma^3) \tag{4.1}$$

where q is the intensity of loading, l is the panel length of the tower, c is the uniformly distributed loaded length, E is Young's modulus of elasticity, I is the moment of inertia and $\gamma = c / l$.

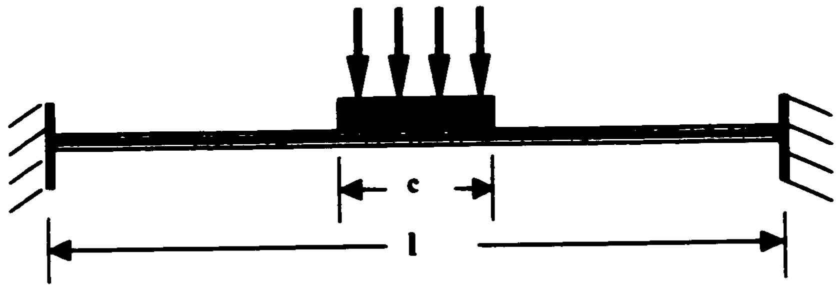


Figure 4.1 Chord member fixed at both ends

Equation 4.1 gives the deflection of a beam that is completely fixed on both ends and is acted upon by a partial uniformly distributed load. This equation was chosen to take into account the setup in the laboratory where a rectangular block was placed on the chord at its center (refer to Figure 3.4). It may be noted that the parameter 'c' in Equation 4.1 is taken as the width of the rectangular block (76.2 mm) whereas '*l*' is the panel length (varying for both types of towers).

The load versus deflection data was calculated for all the towers with the help of Equation 4.1. The graphs are plotted by the side of the experimental deflections and are given in Appendix A. The difference in the deflection in both the cases is given by:

$$\delta = \Delta_E - \Delta_X \tag{4.2}$$

Δ_E is the experimental and Δ_X is the calculated deflection of the chord member. The difference occurs because the ends of the beam are not totally restrained in real situation.

For a beam partially restrained at both ends, the slope at each end, θ, is given by:

$$\theta = M\,l\,/\,2\,E\,I \tag{4.3}$$

or

$$M = (2\,E\,I\,/\,l)\,\theta \tag{4.4}$$

where, M is the moment at both ends of the chord member. Deflection, δ, at mid span is given as:

$$\delta = M\,l^2\,/\,8\,E\,I \tag{4.5}$$

From Equation 4.4 and 4.5, slope θ, at the ends of the beam is given by:

$$\theta = 4\,\delta\,/\,l \tag{4.6}$$

where δ is the differential displacement as calculated in Equation 4.2. The end slope calculated through Equation 4.6 is given in Table 4.1. Load of 6.0 kN was chosen arbitrarily for all the tower sections provided by both the fabricating companies. At this load the chord member remained elastic and did not yield.

4.3 CALCULATION OF EFFECTIVE LENGTH FACTOR

Referring to Figure 4.1, the equation for fixed-end moment is given as:

$$M_{FEM} = \left[\frac{qcl}{24}\left(3-\gamma^2\right)\right] \tag{4.7}$$

The restraining moment M_R, which is caused by the diagonal bracings at the point of weld with the chord member is equal to the fixed end moment less the moment released due to a rotation θ of the member. Mathematically, from Equations 4.4 and 4.7, M_R can be defined as:

$$M_R = \left[\frac{qcl}{24}\left(3-\gamma^2\right)\right] - \frac{2EI}{l}\theta \tag{4.8}$$

The rotational stiffness provided by the diagonal bracings to the ends of chord member can be calculated by dividing the restraining moment with the end slope as given in Equation 4.9 as:

$$\text{Rotational Stiffness} = M_R / \theta \tag{4.9}$$

Equation 4.9 is solved by taking values from Equations 4.6 and 4.8. The results are given in Table 4.1. The table also gives the value of the restraining moment calculated with Equation 4.8.

TABLE 4.1 ROTATIONAL STIFFNESS OF CHORD MEMBERS

S.No.	Specimen ID	Dia. of chord	Expt. Deflection (for 6 kN)	Fixed beam def. (for 6 kN)	Difference	End Slope (Eq. 4.6)	Restraining Moment (Eq.4.8)	Rot. Stiffness (Moment/Slope) (Eq. 4.9)
		mm	mm	mm	mm	Radians	N-mm	N-mm/radian
1	S-1 (A)	38.1	2.39	6.62E-01	1.73E+00	9.07E-03	7.71E+04	8.50E+06
2	S-1 (B)	38.1	2.30	6.62E-01	1.64E+00	8.60E-03	1.03E+05	1.19E+07
3	S-2 (A)	38.1	2.60	6.62E-01	1.94E+00	1.02E-02	1.72E+04	1.69E+06
4	S-2 (B)	38.1	2.64	6.62E-01	1.98E+00	1.04E-02	5.83E+03	5.61E+05
5	S-2 (C)	38.1	2.63	6.62E-01	1.97E+00	1.03E-02	8.68E+03	8.40E+05
6	S-3 (A)	50.8	0.78	2.09E-01	5.71E-01	2.99E-03	5.57E+04	1.86E+07
7	S-3 (B)	50.8	0.81	2.09E-01	6.01E-01	3.15E-03	2.86E+04	9.08E+06
8	S-4 (B)	50.8	0.80	2.09E-01	5.91E-01	3.10E-03	3.76E+04	1.21E+07
9	S-4 (C)	50.8	0.83	2.09E-01	6.21E-01	3.26E-03	1.06E+04	3.26E+06
10	S-5 (A)	69.9	0.23	5.84E-02	1.72E-01	9.01E-04	1.56E+04	1.73E+07
11	S-5 (B)	69.9	0.23	5.84E-02	1.67E-01	8.74E-04	3.17E+04	3.63E+07
12	S-5 (C)	69.9	0.22	5.84E-02	1.62E-01	8.48E-04	4.79E+04	5.64E+07
13	S-6 (A)	69.9	0.21	5.84E-02	1.52E-01	7.96E-04	8.02E+04	1.01E+08
14	S-6 (B)	69.9	0.21	5.84E-02	1.52E-01	7.96E-04	8.02E+04	1.01E+08
15	S-6 (C)	69.9	0.20	5.84E-02	1.42E-01	7.43E-04	1.12E+05	1.51E+08
16	P1 (A)	38.1	1.72	5.37E-01	1.18E+00	6.65E-03	1.44E+05	2.17E+07
17	P1 (B)	38.1	1.70	5.37E-01	1.16E+00	6.54E-03	1.51E+05	2.30E+07
18	P2 (B)	38.1	2.05	5.37E-01	1.51E+00	8.51E-03	3.61E+04	4.24E+06
19	P3 (A)	50.8	0.61	1.70E-01	4.37E-01	2.46E-03	7.89E+04	3.21E+07
20	P3 (B)	50.8	0.59	1.70E-01	4.22E-01	2.37E-03	9.48E+04	3.99E+07
21	P4 (A)	50.8	0.25	1.70E-01	7.60E-02	4.28E-04	4.53E+05	1.06E+09
22	P6 (A)	57.2	0.40	1.06E-01	2.91E-01	1.64E-03	4.76E+04	2.91E+07
23	P6 (B)	57.2	0.38	1.06E-01	2.74E-01	1.54E-03	7.61E+04	4.94E+07
24	P8 (A)	38.1	1.35	5.37E-01	8.13E-01	4.57E-03	2.65E+05	5.80E+07
25	P8 (B)	38.1	1.28	5.37E-01	7.43E-01	4.18E-03	2.88E+05	6.89E+07

At buckling, the forces that act on a column with elastically restrained ends, are shown in the free body diagram Figure 4.2.

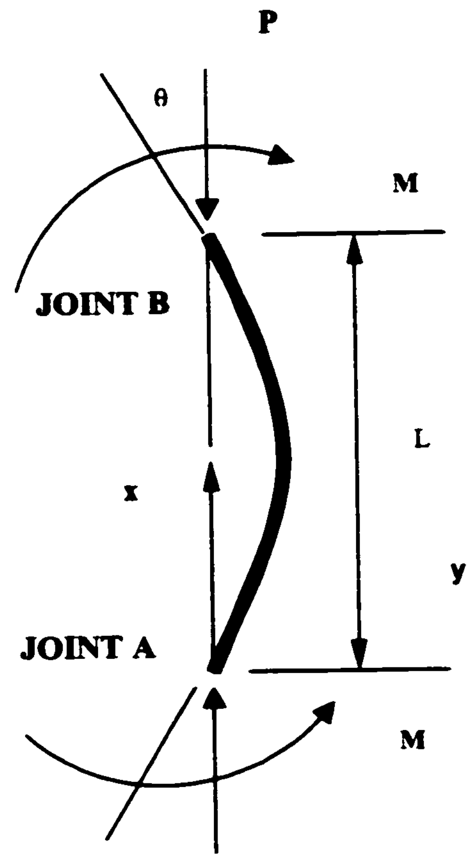


Figure 4.2 Free body diagram of a column

The differential equation of the column acted upon by a load P as shown in Figure 4.2 can be written as:

$$y'' + \alpha^2 y = \frac{M}{EI} \tag{4.10}$$

where M is the end moment induced as a result of buckling of the column due to the applied loading. The solution to Equation 4.10, with consideration of deflection boundary conditions, is given by:

$$y = \frac{M}{P}\left(1 - \tan\frac{\alpha L}{2}\sin\alpha x - \cos\alpha x\right) \qquad (4.11)$$

and

$$\frac{dy}{dx} = \frac{M\alpha}{P}\left(-\tan\frac{\alpha L}{2}\cos\alpha x + \sin\alpha x\right) \qquad (4.12)$$

At x = 0,

$$\frac{dy}{dx} = \theta = \frac{M\alpha}{P}\left(-\tan\frac{\alpha L}{2}\right) \qquad (4.13)$$

From equation 4.13

$$\frac{M}{\theta} = \frac{P}{\alpha\left(-\tan\left(\frac{\alpha L}{2}\right)\right)} \qquad (4.14)$$

The left-hand side of Equation 4.14 is the rotational stiffness as calculated through equation 4.9, where

$$\alpha = \sqrt{\frac{P}{EI}} \qquad (4.15)$$

Euler's equation for elastic buckling of columns is given as:

$$P = \frac{\Pi^2 EI}{(KL)^2} \tag{4.16}$$

Equations 4.14 was solved for P, which was substituted subsequently in Equation 4.16 to find out the effective length factor, K. Young's modulus of elasticity was taken as 200 000 MPa while I, the moment of inertia, and *l*, the panel length, varied depending on the type of tower under consideration. The values of the effective length factor (Experimental) so obtained are given in Table 4.2.

TABLE 4.2 EFFECTIVE LENGTH FACTORS (EXPERIMENTAL)

S.No	Specimen ID	Dia. of chord	Dia. of Diagonal	Panel Length	M of I Chord Member	M of I Diagonal Bracing	K
		mm	mm	mm	mm4	mm4	
1	S-1 (A)	38.1	12.7	762	103 000	1280	0.96
2	S-1 (B)	38.1	12.7		103 000	1280	0.96
3	S-2 (A)	38.1	12.7		103 000	1280	0.96
4	S-2 (B)	38.1	12.7		103 000	1280	0.95
5	S-2 (C)	38.1	12.7		103 000	1280	0.96
6	S-3 (A)	50.8	14.3		327 000	2050	0.98
7	S-3 (B)	50.8	14.3		327 000	2050	0.98
8	S-4 (B)	50.8	14.3		327 000	2050	0.97
9	S-4 (C)	50.8	14.3		327 000	2050	0.98
10	S-5 (A)	69.9	15.9		1170 000	3140	0.99
11	S-5 (B)	69.9	15.9		1170 000	3140	0.98
12	S-5 (C)	69.9	15.9		1170 000	3140	0.99
13	S-6 (A)	69.9	15.9		1170 000	3140	0.99
14	S-6 (B)	69.9	15.9		1170 000	3140	0.98
15	S-6 (C)	69.9	15.9		1170 000	3140	0.99
16	P1 (A)	38.1	15.9	711.2	103 000	3140	0.96
17	P1 (B)	38.1	15.9		103 000	3140	0.96
18	P2 (B)	38.1	15.9		103 000	3140	0.97
19	P3 (A)	50.8	19.1		327 000	6530	0.97
20	P3 (B)	50.8	19.1		327 000	6530	0.97
21	P4 (A)	50.8	19.1		327 000	6530	0.98
22	P6 (A)	57.2	22.2		525 000	11900	0.98
23	P6 (B)	57.2	22.2		525 000	11900	0.98
24	P8 (A)	38.1	22.2		103 000	11900	0.95
25	P8 (B)	38.1	22.2		103 000	11900	0.95

4.4 COMPARISON OF DEFLECTIONS WITH ABAQUS RESULTS

Numerical investigation was carried out with the commercial computer software package ABAQUS in order to compare theoretical results with the deflections of the chord members obtained experimentally. A model of a typical six-panel tower section, ERI specimen, was generated with 40 nodes as shown in the Figure 4.3. The boundary conditions were defined in the same manner as the tower was placed in the laboratory for testing.

The element library in ABAQUS contains several types of beam elements. Different beam elements in ABAQUS use different assumptions. The beam elements chosen for the model definition of the tower are the Timoshenko (shear flexible) beams. These beams allow for transverse shear deformation. They are used since they provide useful results for such beams that are made from uniform material. ABAQUS assumes that the transverse shear behaviour of Timoshenko beams is linear elastic with a fixed modulus and, thus, independent of the response of the beam sections to axial stretch and bending. Refer to Appendix B for INPUT FILES FOR ABAQUS.

It may be noted that in the model, Figure 4.3, with reference to the chord member on which the load is applied the adjacent chord members are not provided. This is to resemble the situation in the laboratory whereby the chord member was cut beyond the panel junction. In order to apply a central concentrated load at the middle of the chord member an extra node was defined at that point.

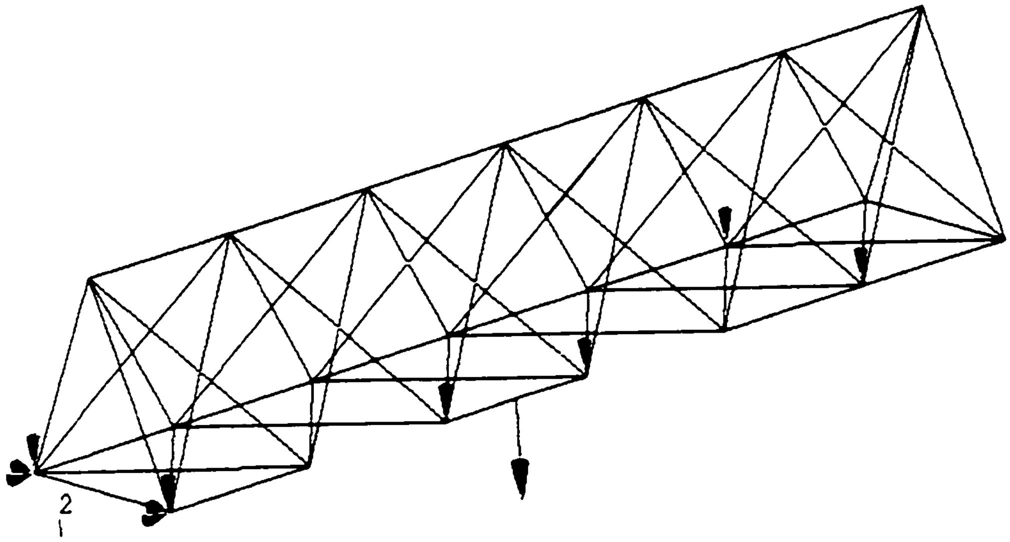


Figure 4.3 Model generated by ABAQUS

Analysis was carried out keeping the magnitude of the applied load equal to 6 kN, for which the experimental deflections were also available. The results are compared with those obtained experimentally and are shown in Figures 4.4, 4.5 and 4.6. The comparison shows the experimental and numerical deflections of tower sections S-1 (A), S-3 (A) and S-5 (A), having diameters of the chord member 38.1, 50.8 and 69.9 respectively.

4.5 DISCUSSION OF RESULTS

As expected, the effective length factors for both types of towers came out to be less than unity. This proved that the diagonal bracings were providing resistance (rotational rigidity) to the chord members, though, in most cases not very significant. The effective length factors varied from 0.95 to 0.99. It was also observed that the thicker the chord member the more closer the effective length factor was to unity. The difference in the weld arrangement of two towers did not have a significant effect on the results. Thus, it can also be said that the diagonals do not provide significant rotational restraint to the chord member, though they do provide lateral restraint. The results obtained through commercial software package ABAQUS gave deflections that were a little less than the experimental ones. This may be due to the reason that the model generated in the computer was stiffer than the one tested experimentally and had ideal loading and supporting conditions. The error between the numerical and experimental results was observed to be less than 5%.

Tower ID	Chord Dia.	Diagonal Dia.	LOAD APPLIED	EXPERIMENTAL DEFLECTION (Average)	DEFLECTION BY ABAQUS
	mm	mm	kN	mm	mm
S-1	38.1	12.7	6.0	2.39	2.35

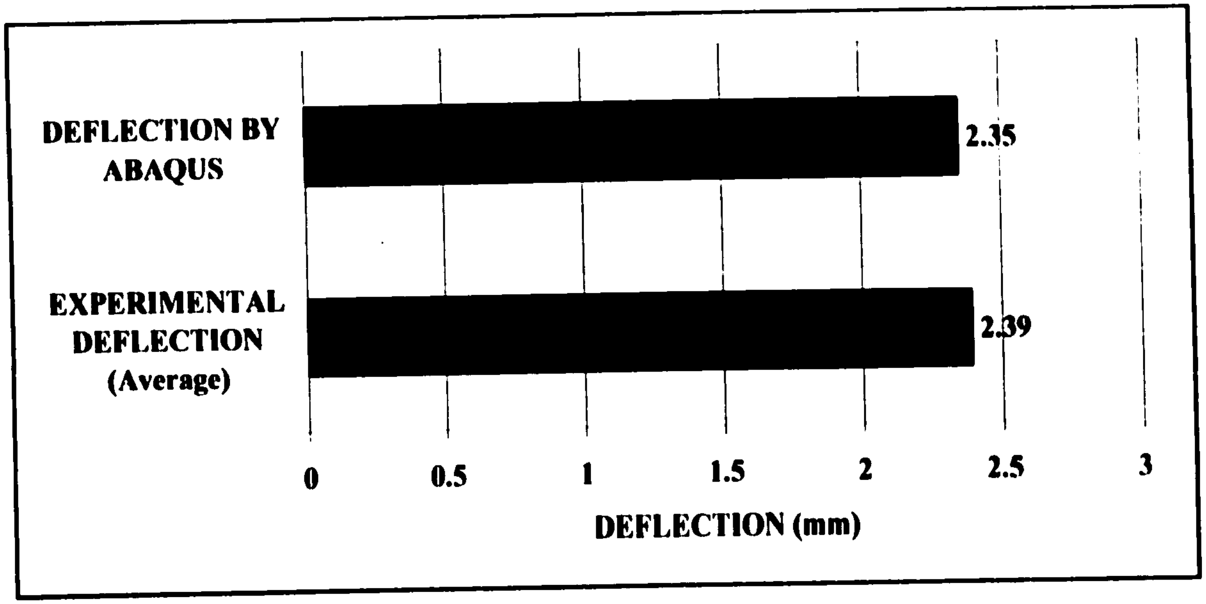


FIGURE 4.4 GRAPHICAL COMPARISONS FOR DEFLECTIONS - SPECIMEN S-1 (A)

Tower ID	Chord Dia.	Diagonal Dia.	LOAD APPLIED	EXPERIMENTAL DEFLECTION (Average)	DEFLECTION BY ABAQUS
	mm	mm	kN	mm	mm
S-3	50.8	14.3	6.0	0.79	0.77

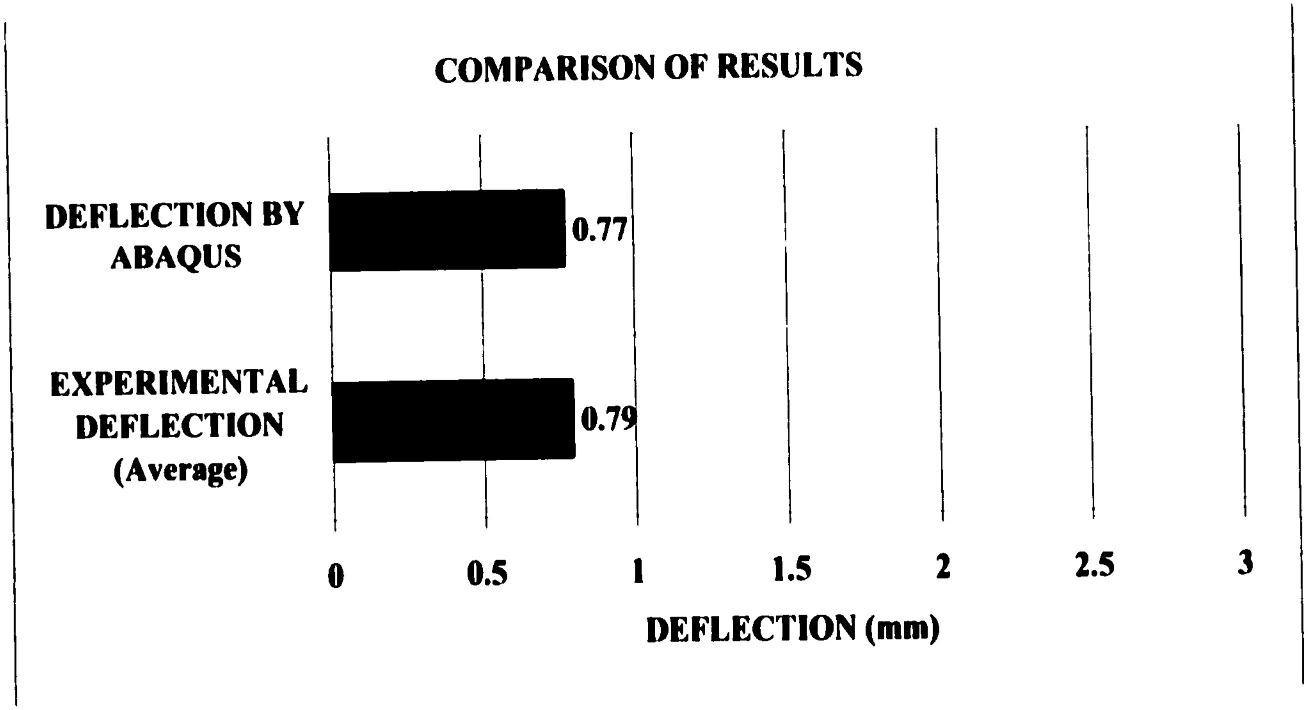


FIGURE 4.5 GRAPHICAL COMPARISONS FOR DEFLECTIONS - SPECIMEN S-3 (A)

Tower ID	Chord Dia.	Diagonal Dia.	LOAD APPLIED	EXPERIMENTAL DEFLECTION (Average)	DEFLECTION BY ABAQUS
	mm	mm	kN	mm	mm
S-5	69.9	15.9	6.0	0.23	0.22

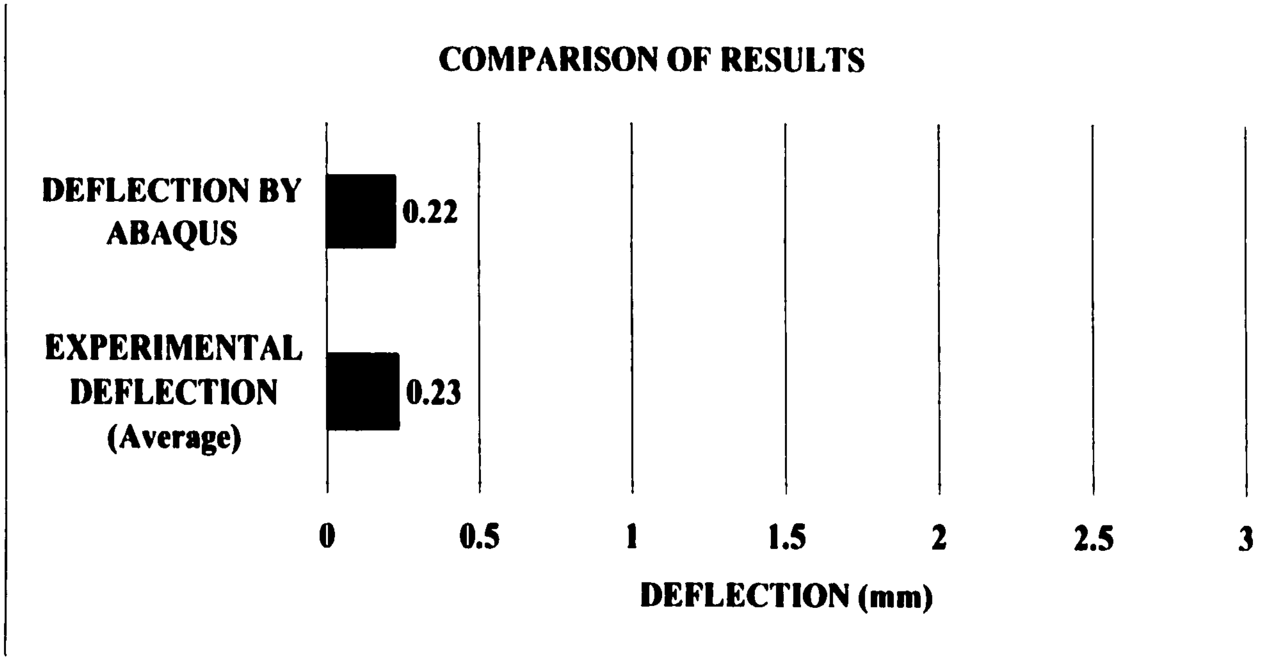


FIGURE 4.6 GRAPHICAL COMPARISONS FOR DEFLECTIONS - SPECIMEN S-5 (A)

CHAPTER FIVE

ANALYTICAL INVESTIGATION

5.1 GENERAL

The effective length factors of the chord members of guyed towers were also calculated analytically using the equation of equilibrium. The relative stiffness ratio, the G-factor, was calculated using the structural analysis computer software STAAD/Pro.

5.2 EQUILIBRIUM EQUATION

In this case the chord member of the tower, which was attached with diagonal bracings, is considered a case similar to an axially loaded column in a framed structure. Where continuous construction is used, the beam-to-column connections are rigid. In this situation, the column-end rotation, that take place during the bending motion of the column will induce corresponding rotations in the beam-ends. Bending moments will be developed at the connections, and these will restrain the motion of the column and thereby increase its strength.

To understand the analysis procedure, the buckling strength of a column in a typical frame is considered as shown in the Figure 5.1. Buckling would occur on the application of an axial force. During the buckling motion, the ends of the columns rotate through angles θ_U (Figure 5.2).

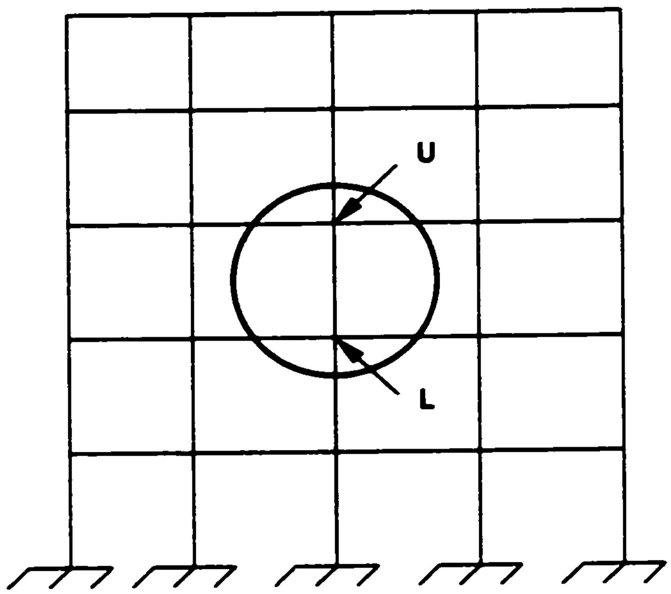


Figure 5.1 Framed Structure

In order to consider the buckling strength of the column UL, the portion of the frame has been isolated in Figure 5.2. The dotted line shows the position of the frame before load is being applied to it. At buckling the additional deformation of the column and adjacent members are also shown in the Figure. Assuming that during the buckling motion, the ends of columns rotate through an angle θ_U or θ_L; the beam ends are forced through this same amount of rotation. A resisting moment, M_{UG}, will be created at the beam-to-column connection given by the following equation:

$$M_{UG} = \frac{2EI_g}{L_g}\theta_u \qquad (5.1)$$

where L_g is the beam length and I_g is the Moment of Inertia of the beam about its axis of bending.

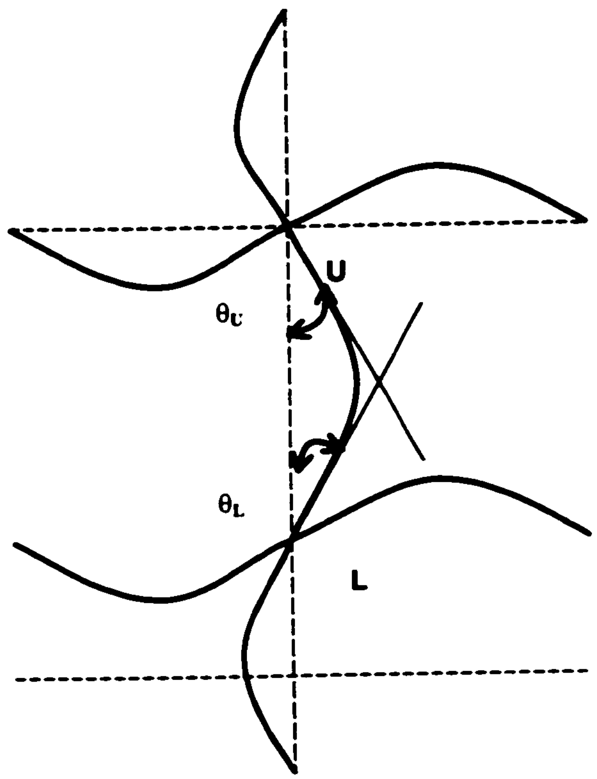


Figure 5.2 Buckling of column in a Framed structure

The other beams at the joint will develop similar moments and resist the rotation of the column ends during buckling. The beams at a particular joint will provide resistance to columns both above and below that joint. Assuming the resistance to buckling is in proportion to the stiffness, I_c/L_c, of the column considered, the net resisting moment, M_U, acting on the column will be given by:

$$M_U = \frac{(I_c / L_c)}{\sum (I_c / L_c)} \times 2E\theta_u \sum (I_g / L_g) = \frac{2EI_c}{GL_c}\theta_u \tag{5.2}$$

Where G is defined as

$$G = \frac{\sum (I_c / L_c)}{\sum (I_g / L_g)} \tag{5.3}$$

The symbol Σ indicates a summation for all members rigidly connected to the joint and lying in the plane in which buckling is being considered. I_c is the column moment of inertia and L_c is the length of the column (Kulak, Adams and Gilmor, 1985).

5.3 DETERMINATION OF ROTATIONAL STIFFNESS USING STAAD/Pro

The differential equation of equilibrium is no longer that for a pin ended column, but must account for the presence of the end moments and shears. The solution to the differential equation can be expressed by the following equation given by the structural stability research council (SSRC, 1976).

$$\frac{G_U G_L}{4}\left(\frac{\Pi}{K}\right)^2 + \left(\frac{G_U + G_L}{2}\right)\left(1 - \frac{(\Pi / K)}{\tan(\Pi / K)}\right) + 2\frac{\tan(\Pi / 2K)}{(\Pi / K)} = 1 \qquad (5.4)$$

where G_U and G_L are defined in equation 5.4 for joints U and L, respectively. K is the effective length factor of the column under consideration and is defined as the ratio of the effective length to the actual length. Equation 5.4 has been plotted as a Nomograph, as shown previously in Figure 2.2. The chord member of the tower, having similar conditions at both ends reduces Equation 5.4 into Equation 5.5:

$$\frac{G^2}{4}\left(\frac{\Pi}{K}\right)^2 + (G)\left(1 - \frac{(\Pi / K)}{\tan(\Pi / K)}\right) + 2\frac{\tan(\Pi / 2K)}{(\Pi / K)} = 1 \qquad (5.5)$$

The value of relative stiffness ration, G, was determined analytically by using the structural analysis software STAAD/Pro. The model of the whole tower was generated using the modelling facility given in STAAD/Pro, which is an interactive menu-driven graphics oriented procedure for creating a model. The model had 39 nodes. (Refer to Figure 5.3). The value of E was taken as 200 000 MPa and Poisson's ratio as 0.3.

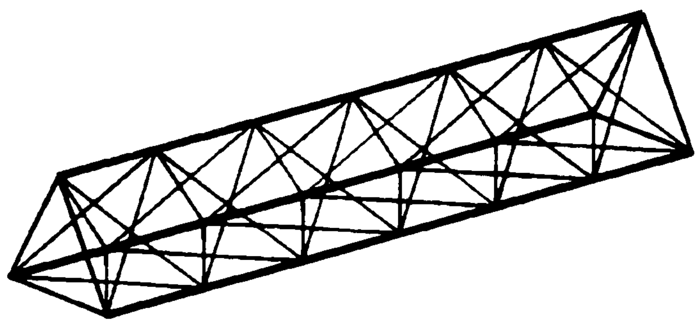


Figure 5.3 Tower model generated using STAAD/Pro

The dimensions of the chord members (panel length and diameter) were changed for different tower specimens as they varied for both the fabricating companies.

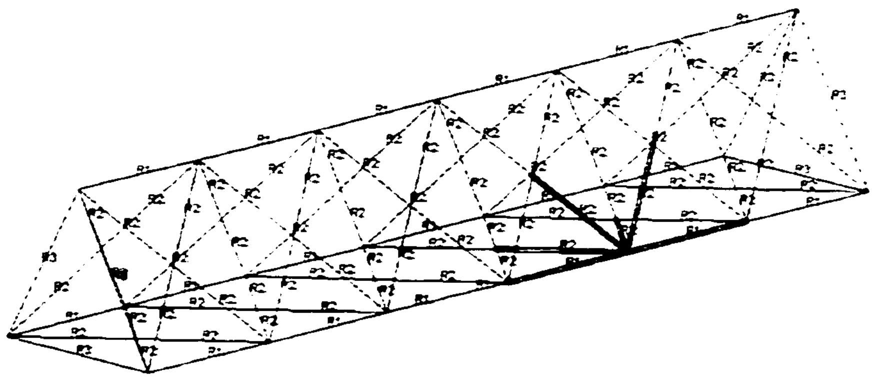


Figure 5.4 Section of tower taken out for analysis

One joint region of the tower, which included two chord members with four diagonal bracings attached to it, as shown highlighted in Figure 5.4, was taken out for analysis. This was to find out the rotational stiffness of the chord member for use in the equilibrium Equation 5.5.

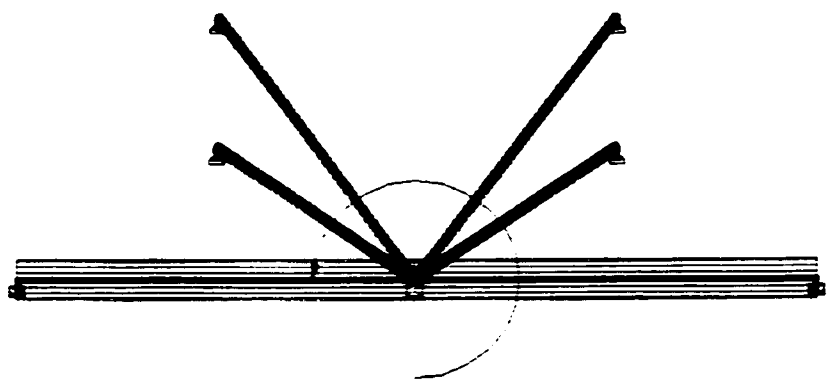


Figure 5.5 Moment applied on tower joint

The far ends of the chord members and the diagonal bracings were defined as fixed and pinned respectively. Moment was than applied, as shown in Figure 5.5. The application of the moment caused the joint to rotate (Figure 5.6) through an angle Θ_1.

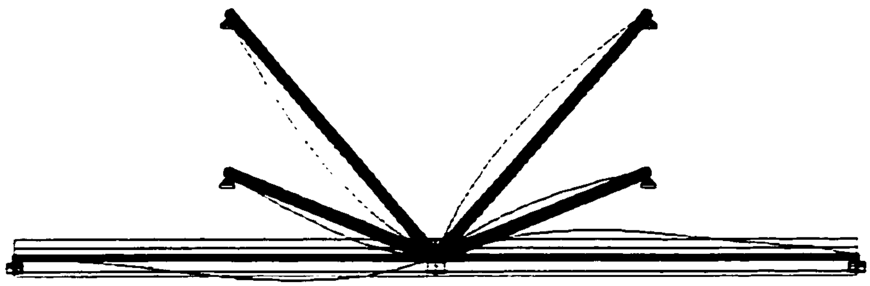


Figure 5.6 Rotation of tower joint

The same amount of moment was again applied on the same tower joint but without the diagonal bracings attached to it. The application of the moment caused the chord to rotate through an angle Θ_2. From equation 5.3, the value of G can now be given as:

$$G = \frac{M/\theta_2}{(M/\theta_1 - M/\theta_2)} \tag{5.6}$$

Equation 5.6 can also be written as:

$$G = \frac{\theta_1}{(\theta_2 - \theta_1)} \tag{5.7}$$

Refer to Table 5.1 for the value of G calculated this way for different tower specimens.

Table 5.1 VALUE OF G CALCULATED USING STAAD/Pro

S.No	Specimen ID	Chord Dia	Panel Length	Diagonal Dia	STAAD/Pro (Rotation)		G (Eq. 5.7)
					06 MEMBERS	02 MEMBERS	
		mm	mm	mm	radians	radians	
1	S-1 (A)	38.1	762	12.7	4.553	4.621	66.6
2	S-1 (B)	38.1	762	12.7	4.553	4.621	66.6
3	S-2 (A)	38.1	762	12.7	4.553	4.621	66.6
4	S-2 (B)	38.1	762	12.7	4.553	4.621	66.6
5	S-2 (C)	38.1	762	12.7	4.553	4.621	66.6
6	S-3 (A)	50.8	762	14.3	1.449	1.466	86.0
7	S-3 (B)	50.8	762	14.3	1.449	1.466	86.0
8	S-4 (B)	50.8	762	14.3	1.449	1.466	86.0
9	S-4 (C)	50.8	762	14.3	1.449	1.466	86.0
10	S-5 (A)	69.85	762	15.9	0.410	0.413	147.4
11	S-5 (B)	69.85	762	15.9	0.410	0.413	147.4
12	S-5 (C)	69.85	762	15.9	0.410	0.413	147.4
13	S-6 (A)	69.85	762	15.9	0.410	0.413	147.4
14	S-6 (B)	69.85	762	15.9	0.410	0.413	147.4
15	S-6 (C)	69.85	762	15.9	0.410	0.413	147.4
16	P1 (A)	38.1	711.2	15.9	4.133	4.600	8.8
17	P1 (B)	38.1	711.2	15.9	4.133	4.600	8.8
18	P2 (B)	38.1	711.2	15.9	4.133	4.600	8.8
19	P3 (A)	50.8	711.2	19.05	1.331	1.460	10.3
20	P3 (B)	50.8	711.2	19.05	1.331	1.460	10.3
21	P4 (A)	50.8	711.2	19.05	1.331	1.460	10.3
22	P6 (A)	57.15	711.2	22.225	0.829	0.913	9.9
23	P6 (B)	57.15	711.2	22.225	0.829	0.913	9.9
24	P8 (A)	38.1	711.2	22.225	3.690	4.600	4.1
25	P8 (B)	38.1	711.2	22.225	3.690	4.600	4.1

5.4 CALCULATION OF EFFECTIVE LENGTH FACTOR

By substituting the value of G in Equation 5.5 the effective length factor of the chord member was calculated. Equation 5.5 was solved such that the sum of all the values on the left-hand side equals to unity. Solver option of the Microsoft Excel package was used for this purpose.

Microsoft Excel Solver uses the Generalized Reduced Gradient. With Solver, an optimal value for a formula in one cell, called the target cell on a worksheet, was calculated. Solver worked with a group of cells that were related, either directly or indirectly, to the formula in the target cell. Solver adjusted the values in the changing cells which were specified, called the adjustable cells, to produce the result which were specified from the target cell formula. Constraints were applied to restrict the values solver can use in the model. Refer to Table 5.2 for the values of K calculated through this method.

5.5 DISCUSSION OF RESULTS

The effective length factors found through this procedure matched with the results obtained through the analysis of the load deflection data and varied from 0.93 to 0.99. The rotational stiffness values of chord and diagonal members were calculated from the structural analysis computer software STAAD/Pro. The relative stiffness factor, G, varied from 147 to 4. The effective length factors so determined were found to be in accordance with the Nomograph provided in CAN/CSA S16.1, which is based on the G factor.

TABLE 5.2 EFFECTIVE LEGTH FACTORS (STAAD/Pro)

S.No	Specimen ID	Dia. of chord	Dia. of Diagonal	Panel Length	Diagonal Length	M.I of chord member	M.I Diagonal Bracing	G	K
		mm	mm	mm	mm	mm4	mm4		
1	S-1 (A)	38.1	12.7	762	1117.6	103 000	1280	66.6	0.98
2	S-1 (B)	38.1	12.7			103 000	1280	66.6	0.98
3	S-2 (A)	38.1	12.7			103 000	1280	66.6	0.98
4	S-2 (B)	38.1	12.7			103 000	1280	66.6	0.98
5	S-2 (C)	38.1	12.7			103 000	1280	66.6	0.98
6	S-3 (A)	50.8	14.3			327 000	2050	86.0	0.99
7	S-3 (B)	50.8	14.3			327 000	2050	86.0	0.99
8	S-4 (B)	50.8	14.3			327 000	2050	86.0	0.99
9	S-4 (C)	50.8	14.3			327 000	2050	86.0	0.99
10	S-5 (A)	69.9	15.9			1170 000	3140	147.4	0.99
11	S-5 (B)	69.9	15.9			1170 000	3140	147.4	0.99
12	S-5 (C)	69.9	15.9			1170 000	3140	147.4	0.99
13	S-6 (A)	69.9	15.9			1170 000	3140	147.4	0.99
14	S-6 (B)	69.9	15.9			1170 000	3140	147.4	0.99
15	S-6 (C)	69.9	15.9			1170 000	3140	147.4	0.99
16	P1 (A)	38.1	15.9	711.2	1158.4	130 000	3140	8.9	0.96
17	P1 (B)	38.1	15.9			130 000	3140	8.9	0.96
18	P2 (B)	38.1	15.9			130 000	3140	8.9	0.96
19	P3 (A)	50.8	19.1			327 000	6530	10.3	0.97
20	P3 (B)	50.8	19.1			327 000	6530	10.3	0.97
21	P4 (A)	50.8	19.1			327 000	6530	10.3	0.97
22	P6 (A)	57.2	22.2			525 000	11900	9.9	0.96
23	P6 (B)	57.2	22.2			525 000	11900	9.9	0.96
24	P8 (A)	38.1	22.2			103 000	11900	4.1	0.93
25	P8 (B)	38.1	22.2			103 000	11900	4.1	0.93

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATION

6.1 GENERAL

Chord members of twenty-five specimens, actual tower sections provided by two companies, were tested by applying a load at the center. All the tower sections had a triangular cross section and were made up of solid rounds. The joints were welded and none of the tower sections was galvanized. The data gathered through experiments was used to find out the effective length factors. Effective length factors were also calculated by using the equilibrium equation.

6.2 CONCLUSIONS

Based on the analytical and numerical analysis, the following conclusions can be drawn:

1. The effective length factor of the tower sections calculated through the analysis of the experimental data, was found out to be less than unity. The value ranges from 0.95 to 0.99. Thus it was determined that the rotational restraint provided by diagonal bracing to the chord member is not very significant, specially for the case where a relatively thicker chord member is fabricated with a relatively thinner diagonal bracing.

2. The values of the effective length factors obtained by making use of the equilibrium equation given by structural stability research council (SSRC, 1976) were found to be close to the values obtained through the analysis of the load deflection data. Thus, the equilibrium equation or more conveniently the Nomograph (Figure 2.2), can be used to determine the effective length factor of chord members of guyed towers.

3. Good agreement was observed between the results obtained through commercial software package ABAQUS and the experimental data.

6.3 RECOMMENDATION

Since the effective length factors were found to be very close to 1.0 so it is recommended that the effective length factor for solid round chord members of guyed towers should be taken as unity as recommended by Canadian Standards Association S37-94 "ANTENNAS, TOWERS, AND ANTENNA-SUPPORTING STRUCTURES".

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APPENDIX – A

FIGURES

Load Applied	Experimental deflection	Theorectical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.40	0.11
2.0	0.80	0.22
3.0	1.20	0.33
4.0	1.59	0.44
5.0	1.99	0.55
6.0	2.39	0.66

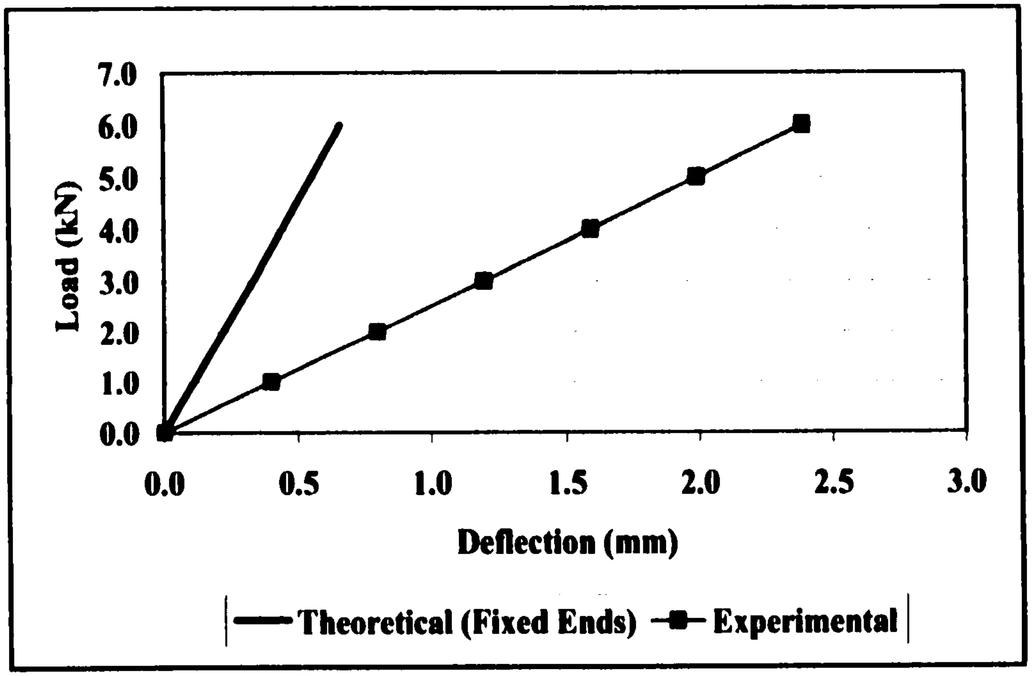

Figure A.1 Load vs. Deflection data for chord member - Specimen S-1 (A)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.38	0.11
2.0	0.77	0.22
3.0	1.15	0.33
4.0	1.54	0.44
5.0	1.92	0.55
6.0	2.30	0.66

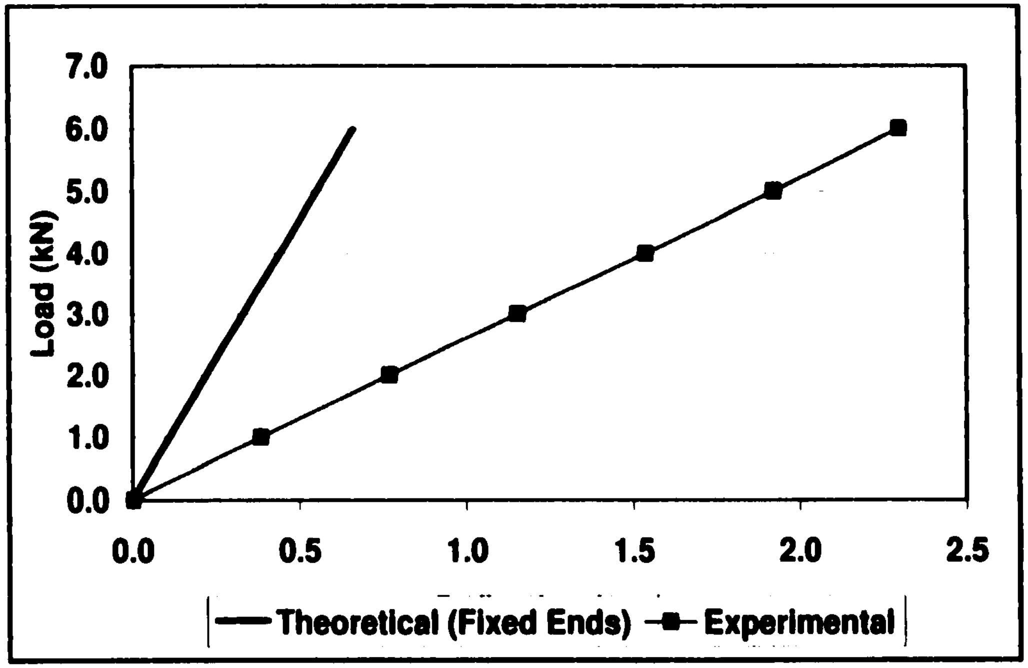

Figure A.2 **Load vs. Deflection data for chord member - Specimen S-1 (B)**

Load Applied	Experimental deflections	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0	0
1.0	0.44	0.11
2.0	0.88	0.22
3.0	1.32	0.33
4.0	1.76	0.44
5.0	2.20	0.55
6.0	2.60	0.66

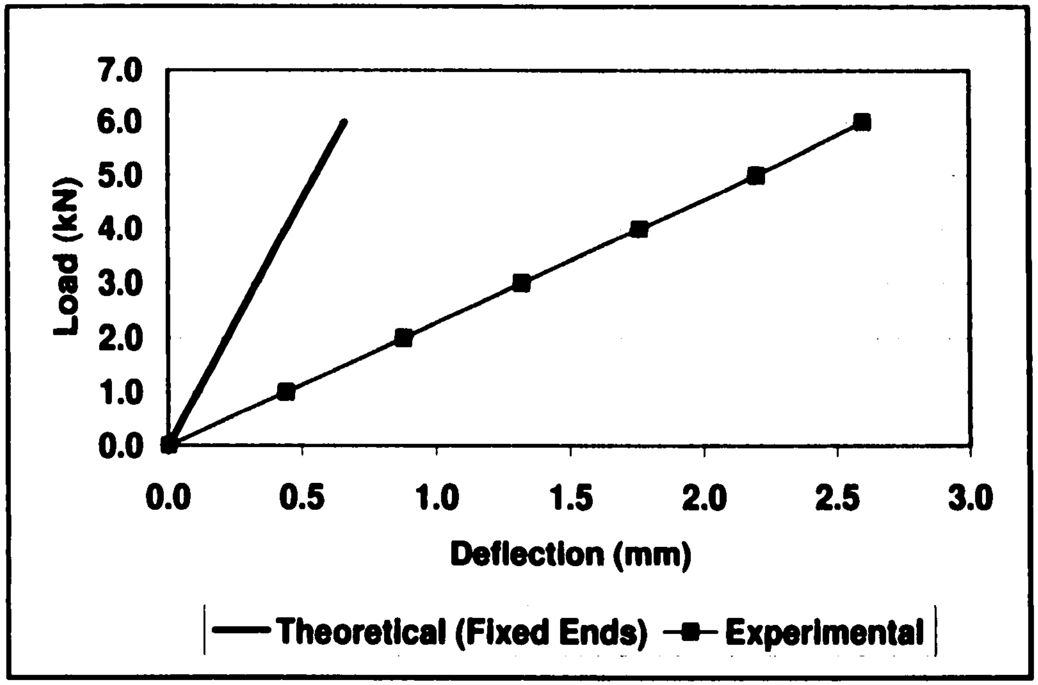

Figure A.3 Load vs. Deflection Data for Chord Member - Specimen S-2 (A)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.44	0.11
2.0	0.88	0.22
3.0	1.32	0.33
4.0	1.76	0.44
5.0	2.20	0.55
6.0	2.64	0.66

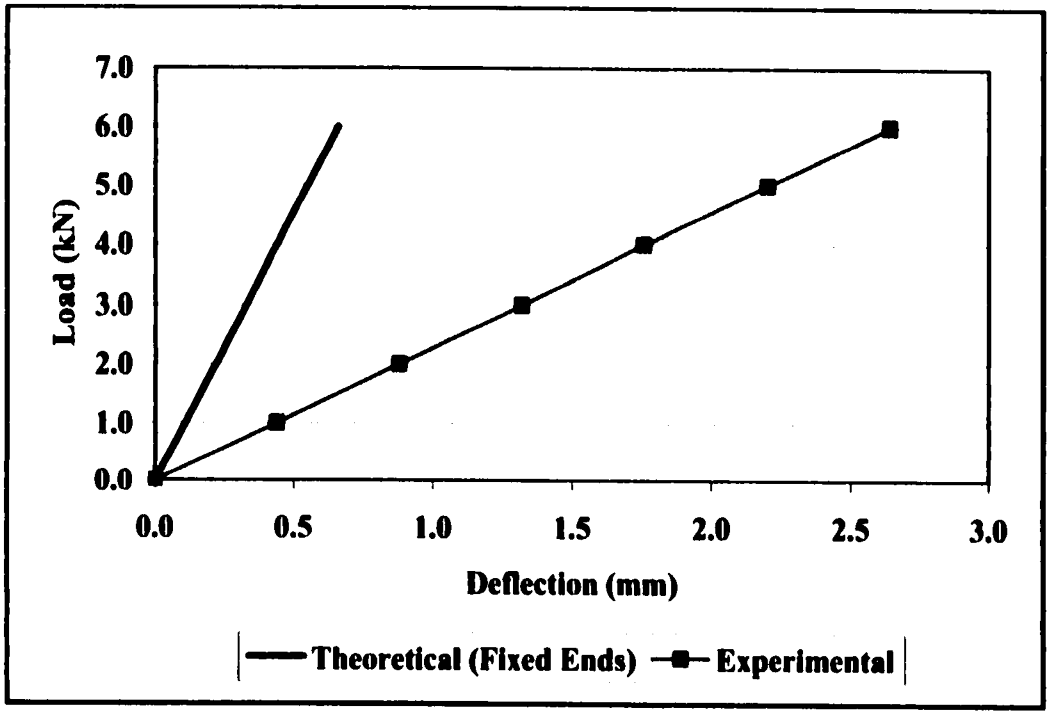

Figure A.4 Load vs. Deflection data for chord member - Specimen S-2 (B)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.44	0.09
2.0	0.88	0.18
3.0	1.32	0.27
4.0	1.75	0.36
5.0	2.19	0.45
6.0	2.63	0.54

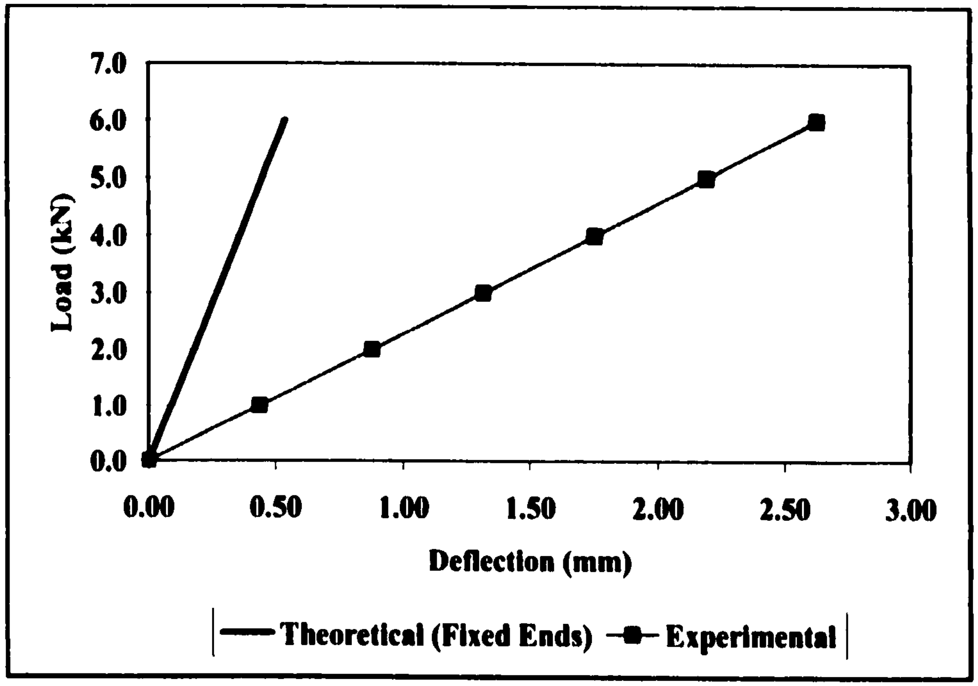

Figure A.5 Load vs. Deflection data for chord member - Specimen S-2 (C)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0	0
1.0	0.13	0.03
2.0	0.26	0.07
3.0	0.39	0.10
4.0	0.52	0.14
5.0	0.65	0.17
6.0	0.78	0.21

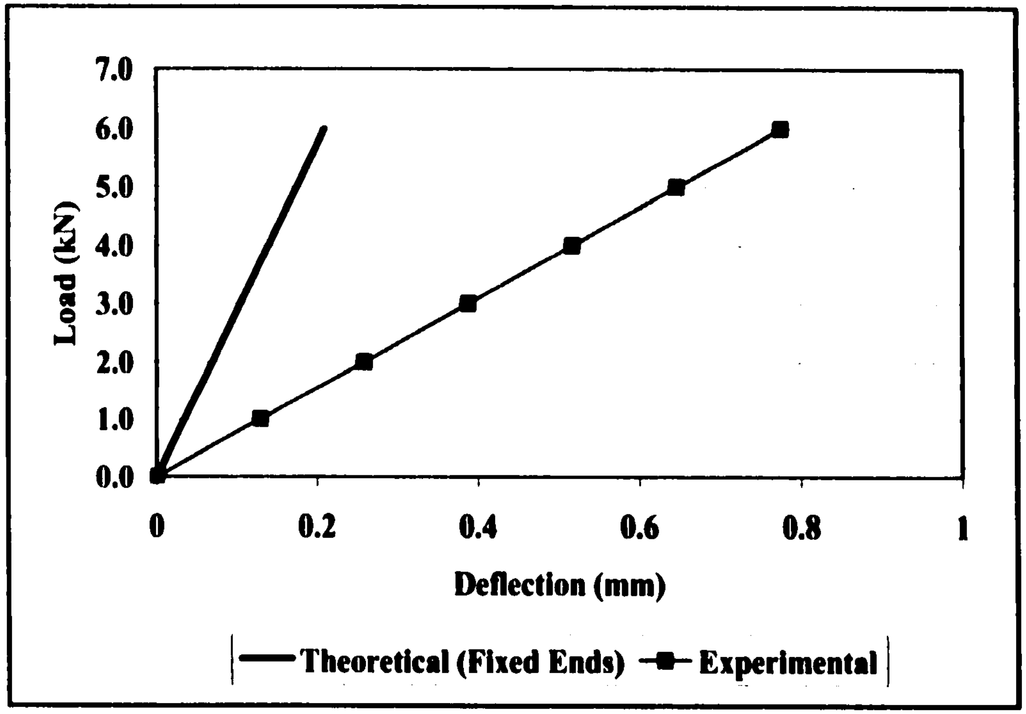

Figure A.6 Load vs. Deflection data for chord member - Specimen S-3 (A)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.14	0.03
2.0	0.27	0.07
3.0	0.41	0.10
4.0	0.54	0.14
5.0	0.68	0.17
6.0	0.81	0.21

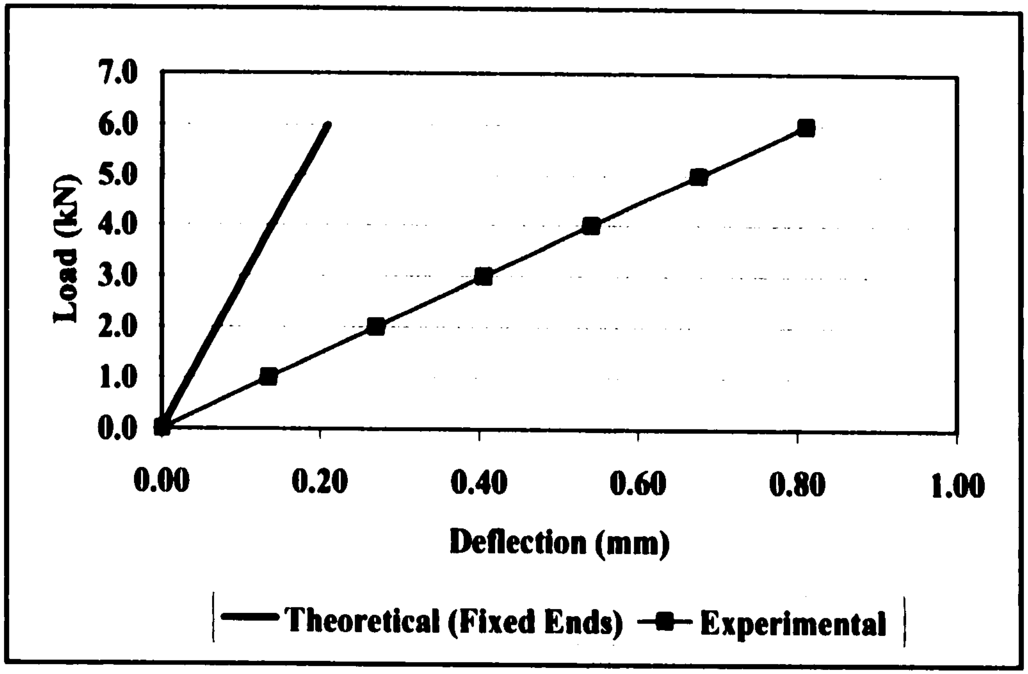

Figure A.7 Load Vs. Deflection data for chord member - Specimen S-3 (B)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0	0
1.0	0.13	0.03
2.0	0.27	0.07
3.0	0.40	0.10
4.0	0.53	0.14
5.0	0.67	0.17
6.0	0.80	0.21

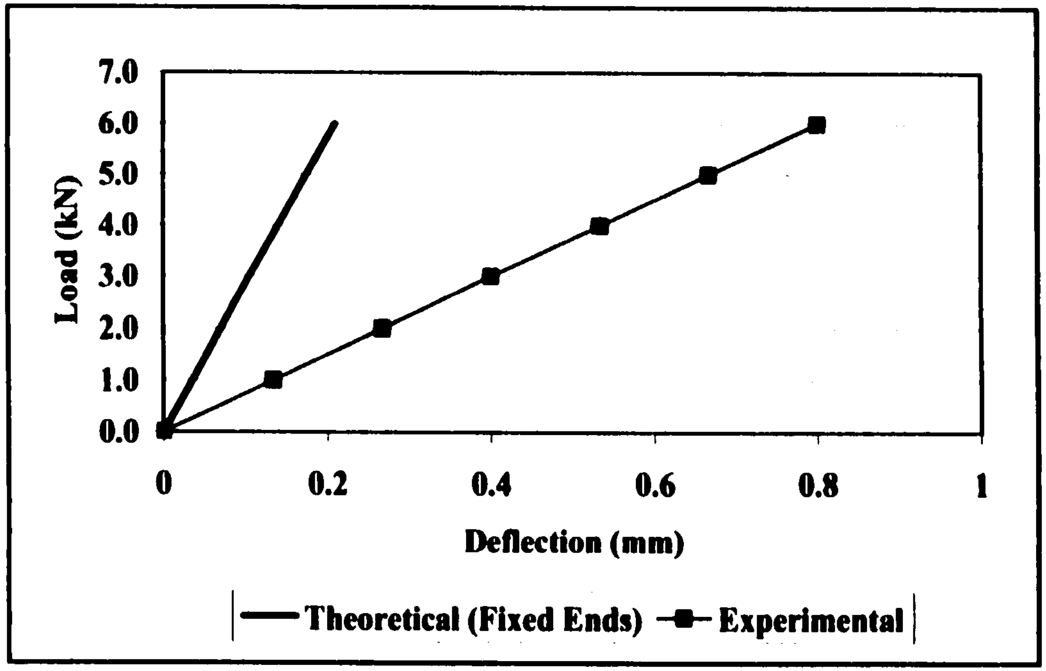

Figure A.8 Load vs. Deflection data for chord member - Specimen S-4 (B)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0	0
1.0	0.14	0.03
2.0	0.28	0.07
3.0	0.41	0.10
4.0	0.55	0.14
5.0	0.69	0.17
6.0	0.83	0.21

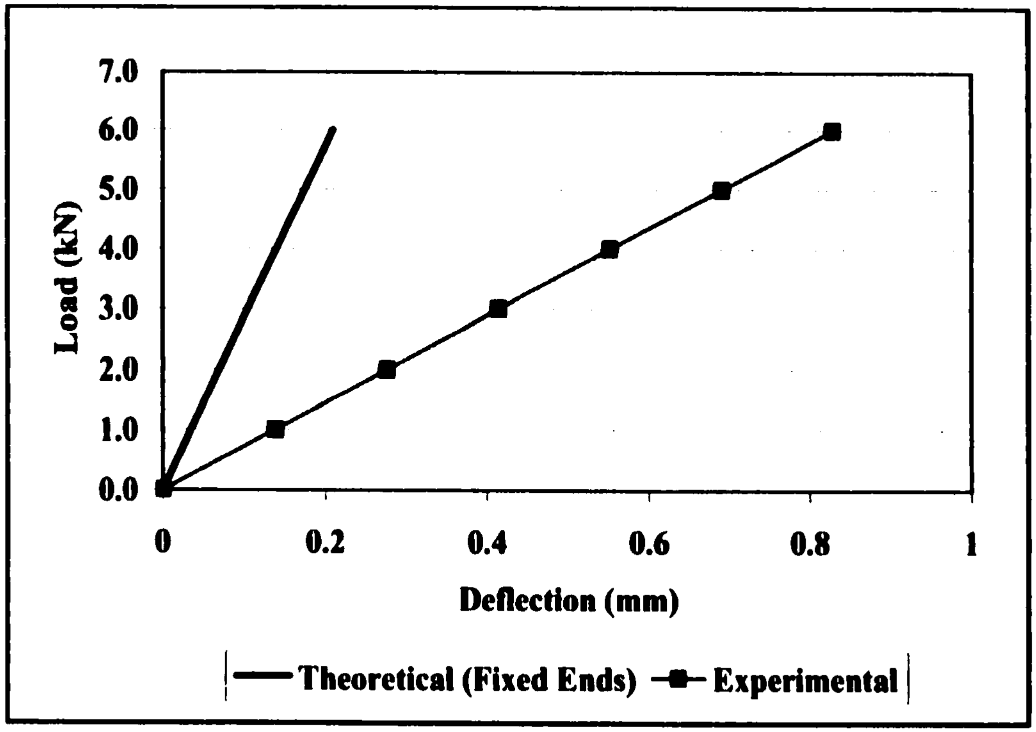

Figure A.9 Load vs. Deflection data for chord member - Specimen S-4 (C)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0	0
1.0	0.04	0.01
2.0	0.08	0.02
3.0	0.12	0.03
4.0	0.15	0.04
5.0	0.19	0.05
6.0	0.23	0.06

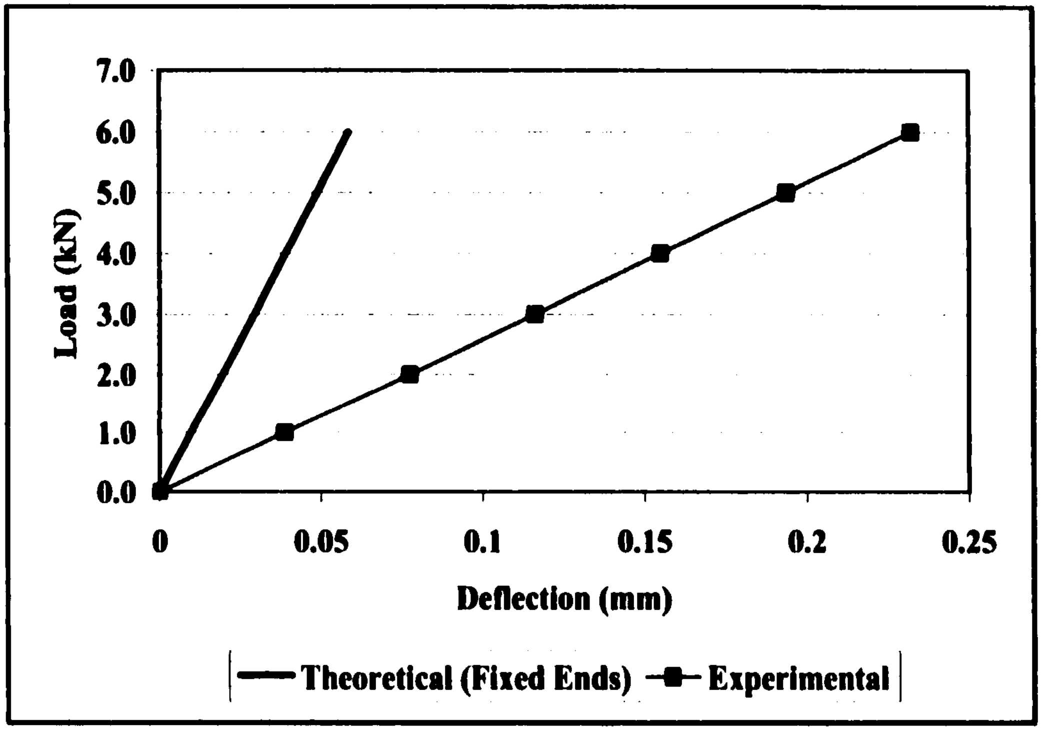

Figure A.10 Load vs. Deflection data for chord member - Specimen S-5 (A)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0	0
1.0	0.04	0.01
2.0	0.08	0.02
3.0	0.11	0.03
4.0	0.15	0.04
5.0	0.19	0.05
6.0	0.23	0.06

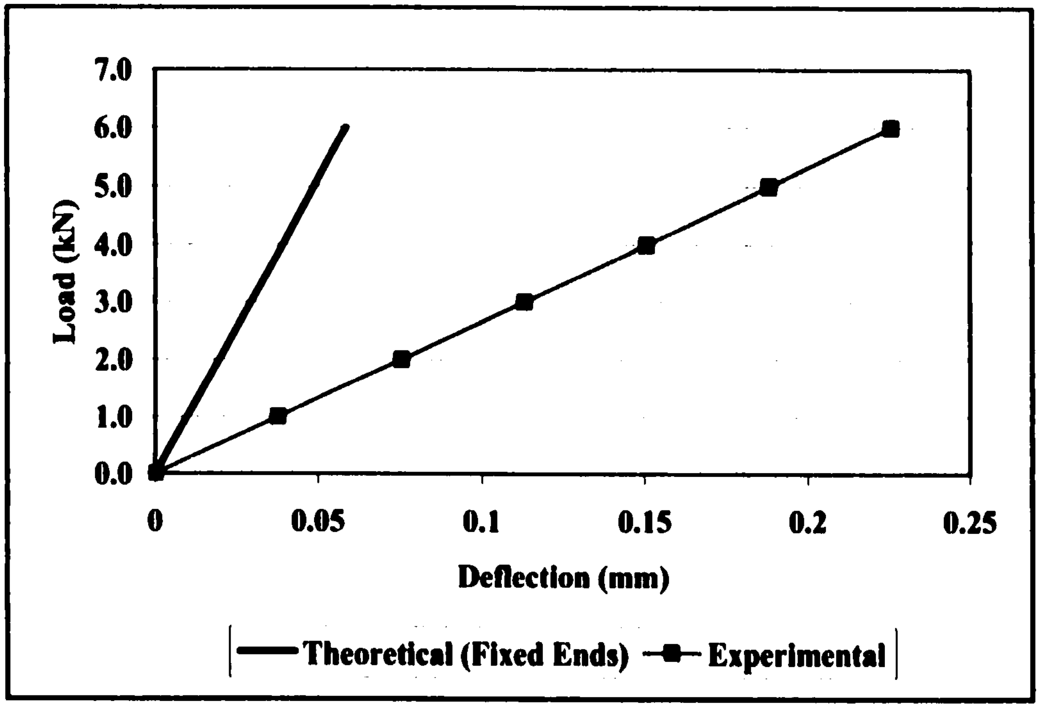

Figure A.11 Load vs. Deflection data for chord member - Specimen S-5 (B)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.04	0.01
2.0	0.07	0.02
3.0	0.11	0.03
4.0	0.15	0.04
5.0	0.19	0.05
6.0	0.22	0.06

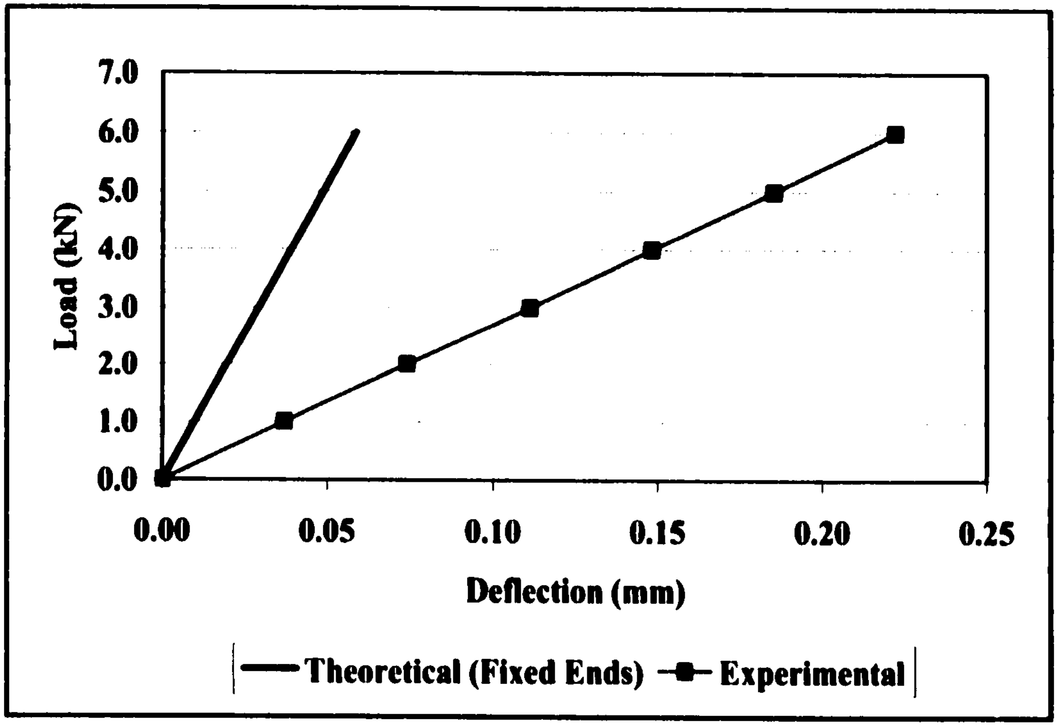

Figure A.12 Load vs. Deflection data for chord member - Specimen S-5 (C)

Load Applied	Experimental Deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.04	0.01
2.0	0.07	0.02
3.0	0.11	0.03
4.0	0.14	0.04
5.0	0.18	0.05
6.0	0.21	0.06

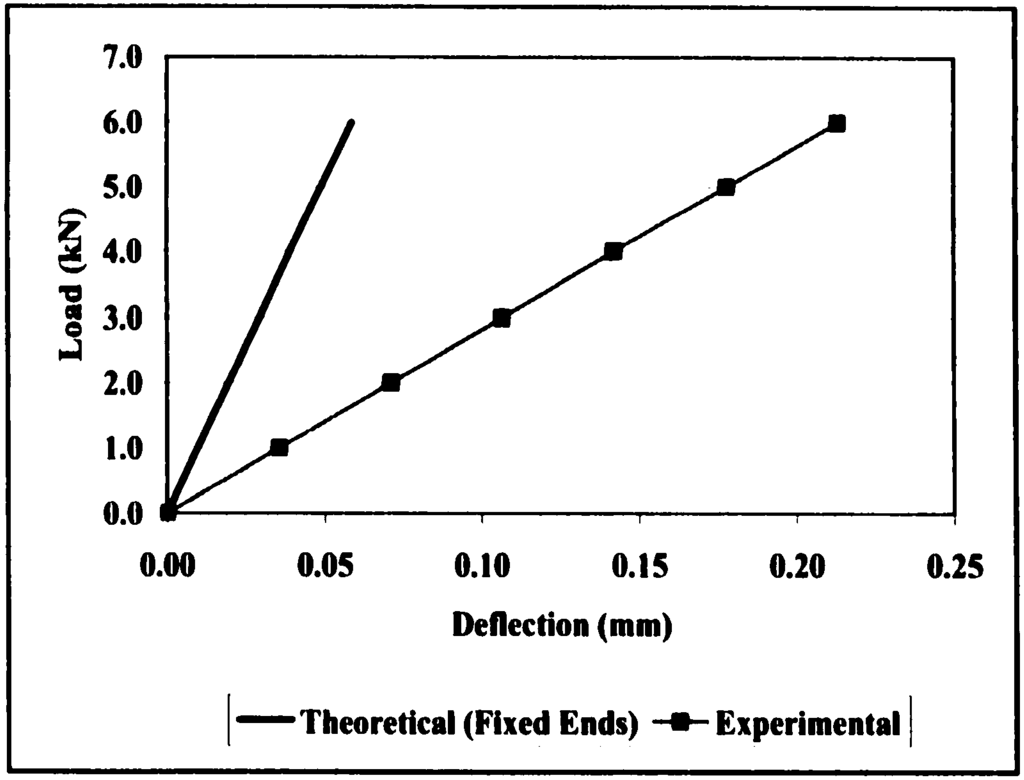

Figure A.13 Load vs. Deflection data for chord member - Specimen S-6 (A)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0	0
1.0	0.04	0.01
2.0	0.07	0.02
3.0	0.11	0.03
4.0	0.14	0.04
5.0	0.18	0.05
6.0	0.21	0.06

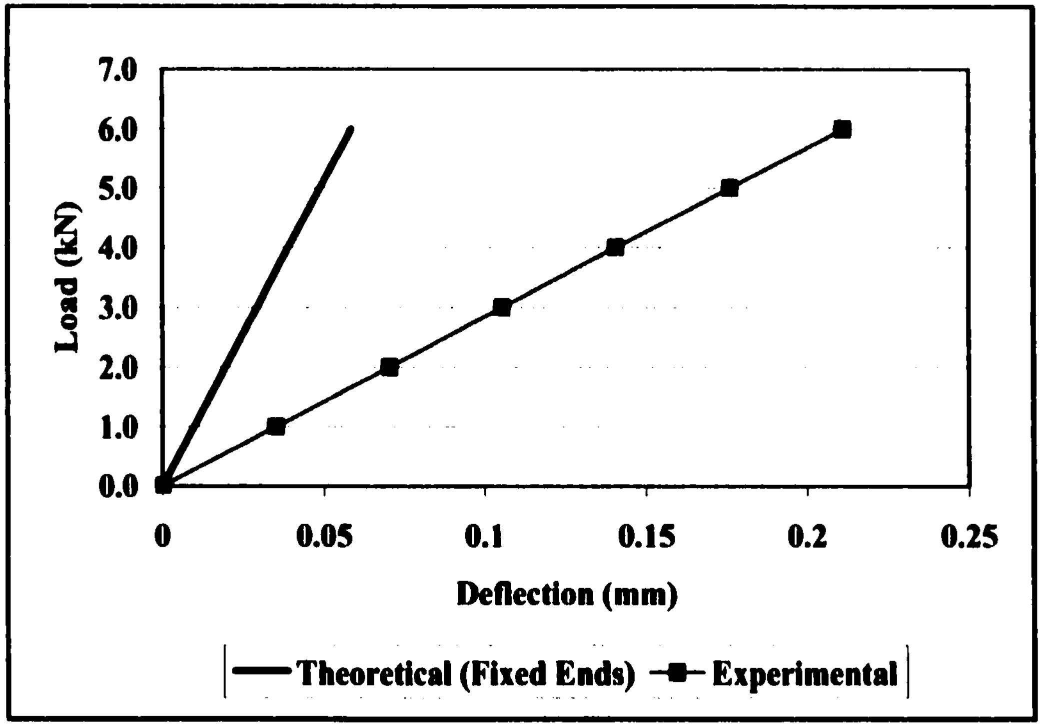

Figure A.14 Load vs. Deflection data for chord member - Specimen S-6 (B)

Load Applied	Experimental deflections	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.04	0.01
2.0	0.07	0.02
3.0	0.11	0.03
4.0	0.14	0.04
5.0	0.17	0.05
6.0	0.20	0.06

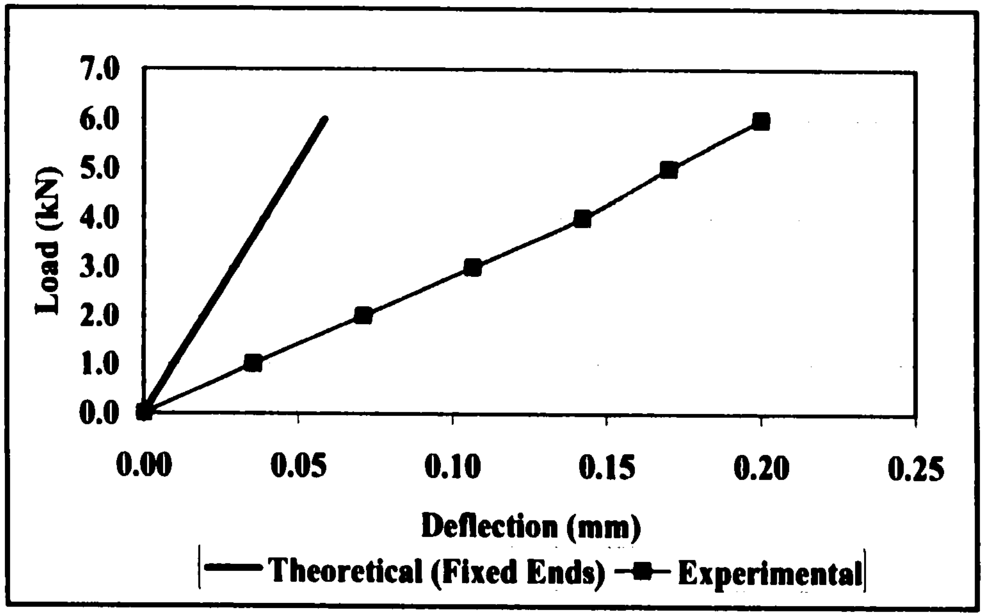

Figure A.15 Load vs. Deflection data for chord member - Specimen S-6 (C)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.29	0.09
2.0	0.57	0.18
3.0	0.86	0.27
4.0	1.15	0.36
5.0	1.44	0.45
6.0	1.72	0.54

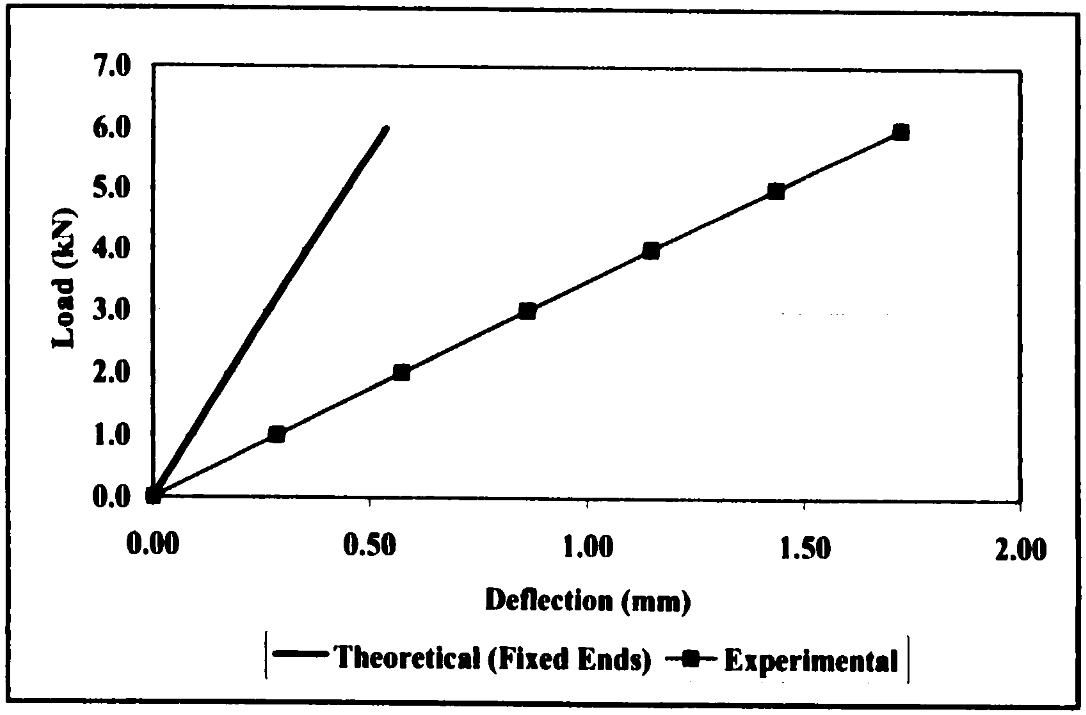

Figure A.16 Load vs. Deflection data for chord member - Specimen P1 (A)

Load Applied	Experimental Deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0	0.00	0.00
1	0.28	0.09
2	0.57	0.18
3	0.85	0.27
4	1.13	0.36
5	1.42	0.45
6	1.70	0.54

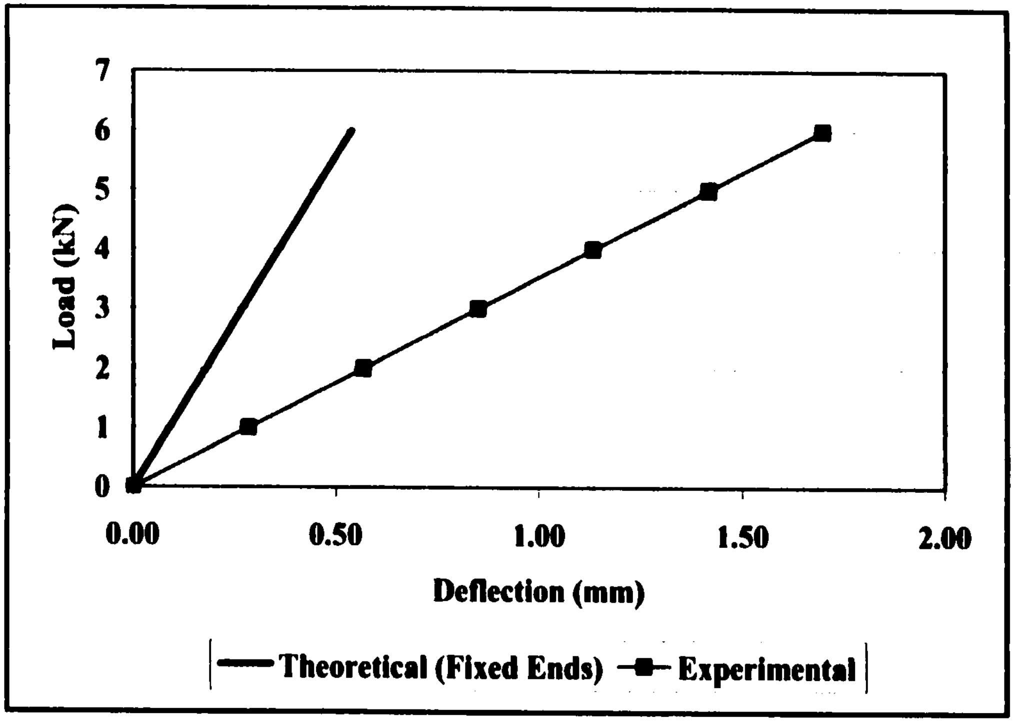

Figure A.17 Load vs. Deflection data for chord member - Specimen P1 (B)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.34	0.09
2.0	0.68	0.18
3.0	1.02	0.27
4.0	1.36	0.36
5.0	1.70	0.45
6.0	2.05	0.54

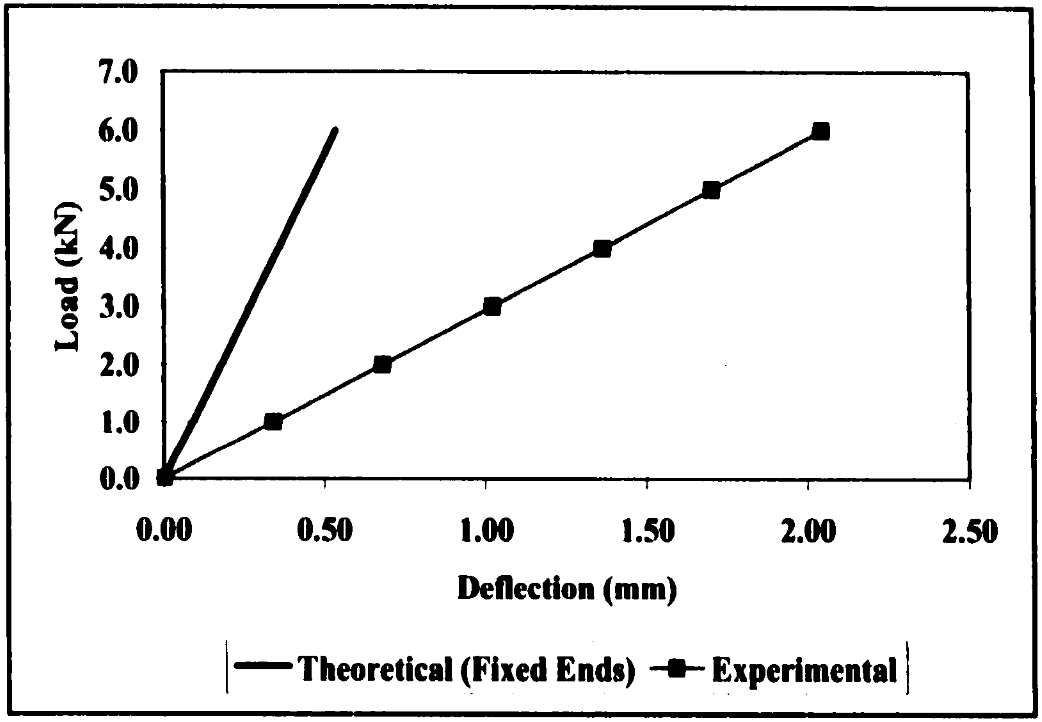

Figure A.18 Load vs. Deflection data for chord member - Specimen P2 (B)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.10	0.03
2.0	0.20	0.06
3.0	0.30	0.09
4.0	0.40	0.11
5.0	0.51	0.14
6.0	0.61	0.17

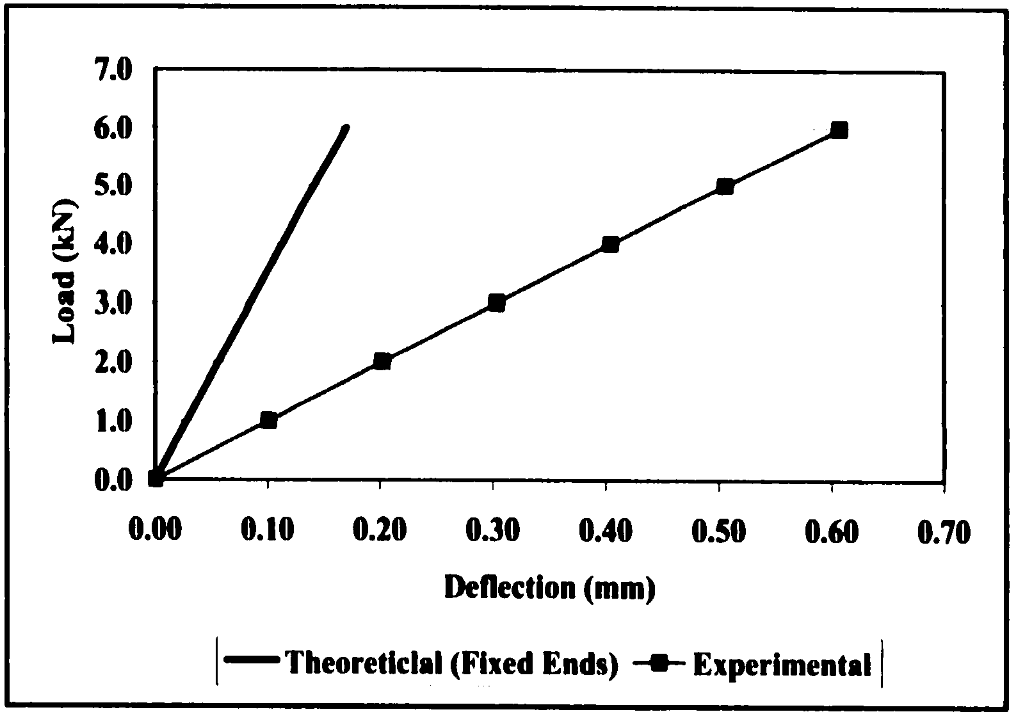

Figure A.19 Load vs. Deflection data for chord member - Specimen P3 (A)

Load Applied	Experimental Deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.10	0.03
2.0	0.20	0.06
3.0	0.30	0.09
4.0	0.39	0.11
5.0	0.49	0.14
6.0	0.59	0.17

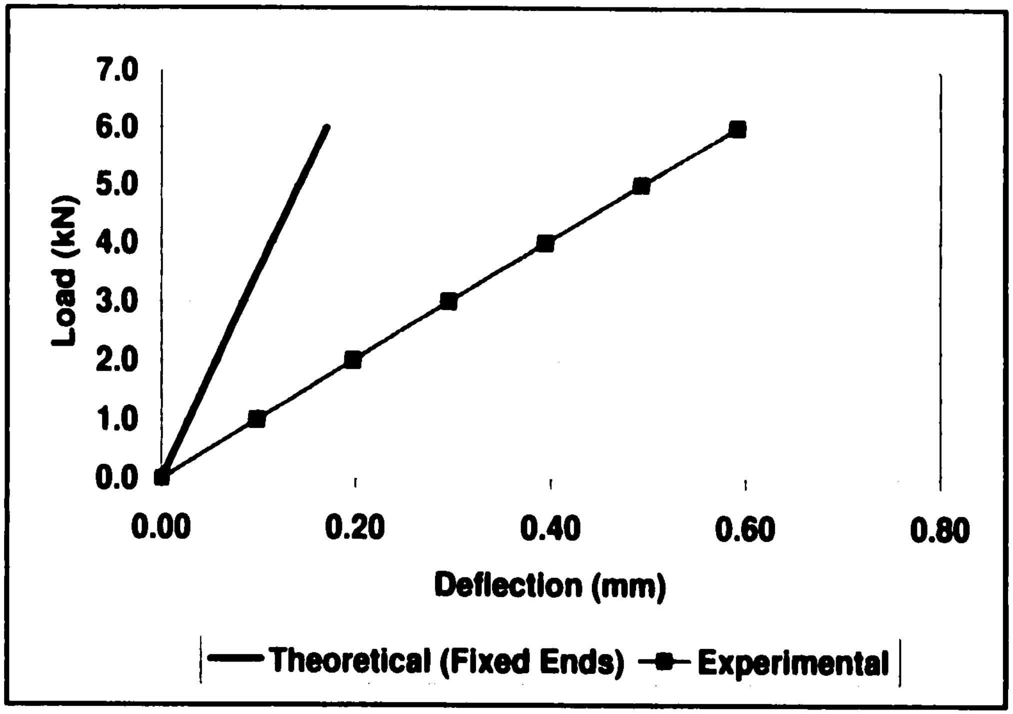

Figure A.20 Load vs. Deflection data for chord member - Specimen P3 (B)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.04	0.03
2.0	0.08	0.06
3.0	0.12	0.09
4.0	0.16	0.11
5.0	0.20	0.14
6.0	0.25	0.17

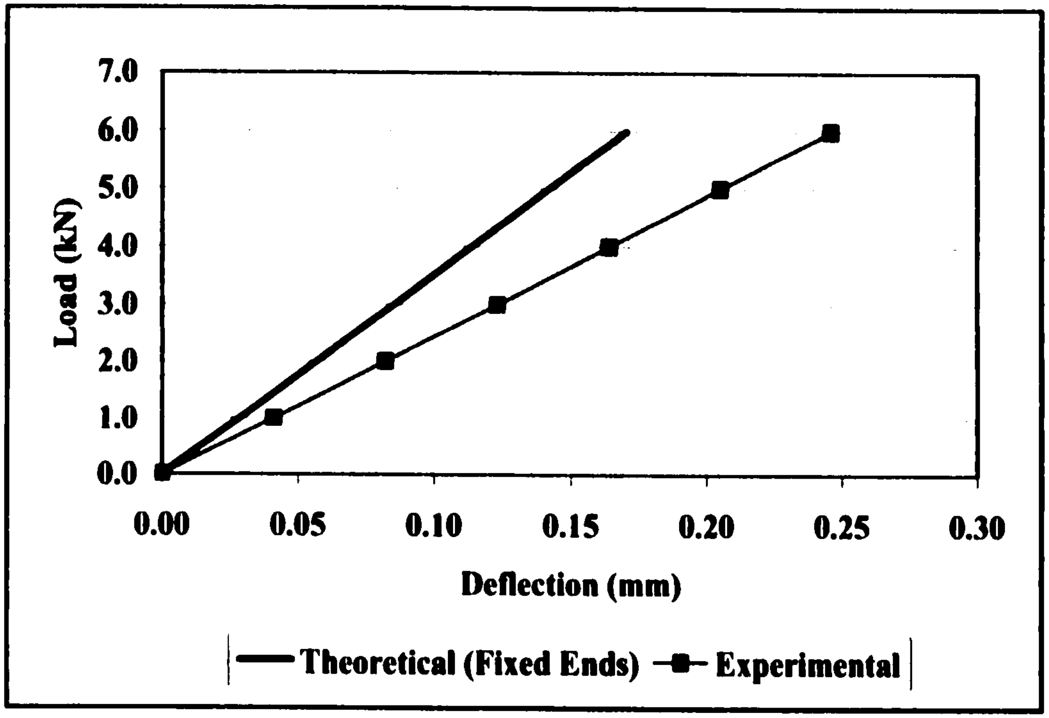

Figure A.21 Load vs. Deflection data for chord member - Specimen P4 (A)

Load Applied	Experimental Deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.06	0.02
2.0	0.13	0.04
3.0	0.19	0.05
4.0	0.26	0.07
5.0	0.33	0.09
6.0	0.40	0.11

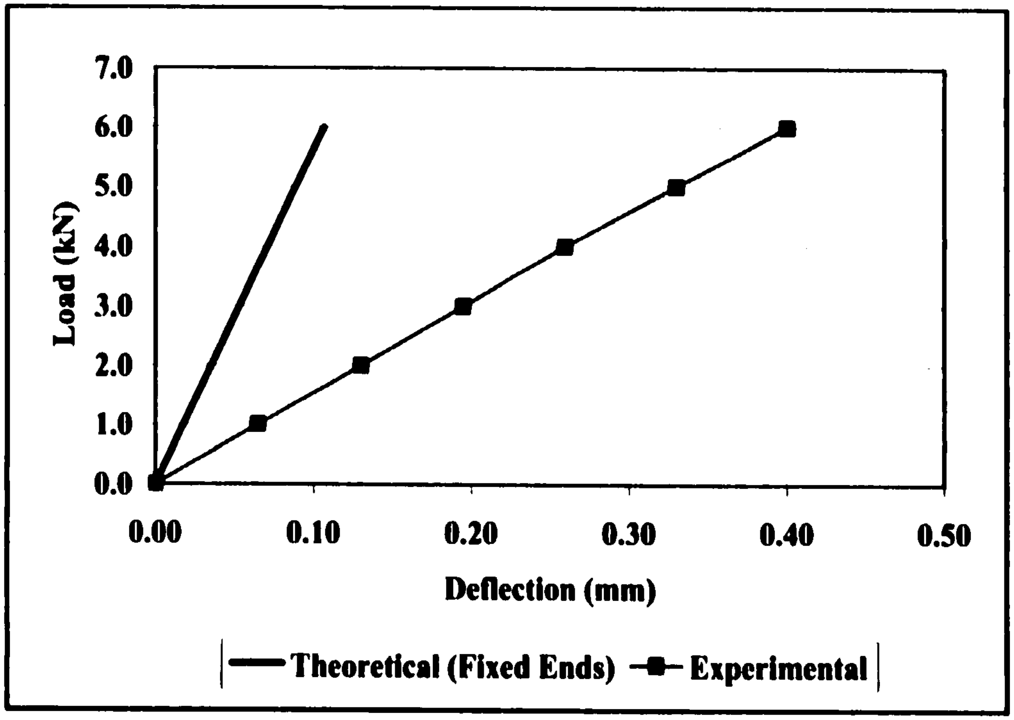

Figure A.22 Load vs. Deflection data for chord member - Specimen P6 (A)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.06	0.02
2.0	0.13	0.04
3.0	0.19	0.05
4.0	0.25	0.07
5.0	0.32	0.09
6.0	0.38	0.11

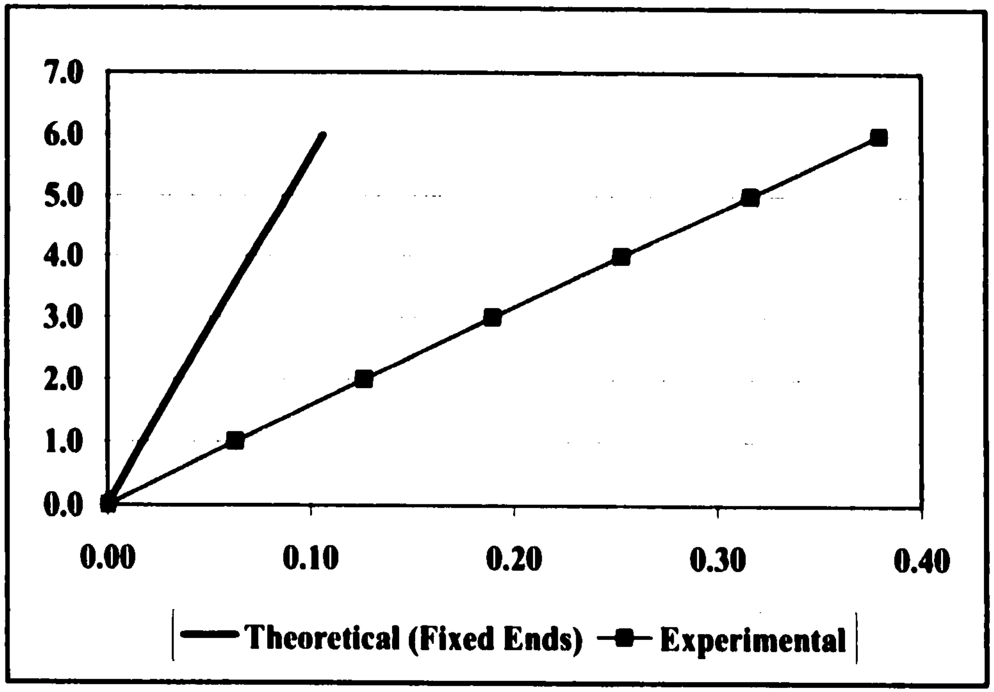

Figure A.23 Load Vs. Deflection data for chord member - Specimen P6 (B)

Load Applied	Experimental Deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0	0.00	0.00
1	0.23	0.09
2	0.45	0.18
3	0.68	0.27
4	0.90	0.36
5	1.13	0.45
6	1.35	0.54

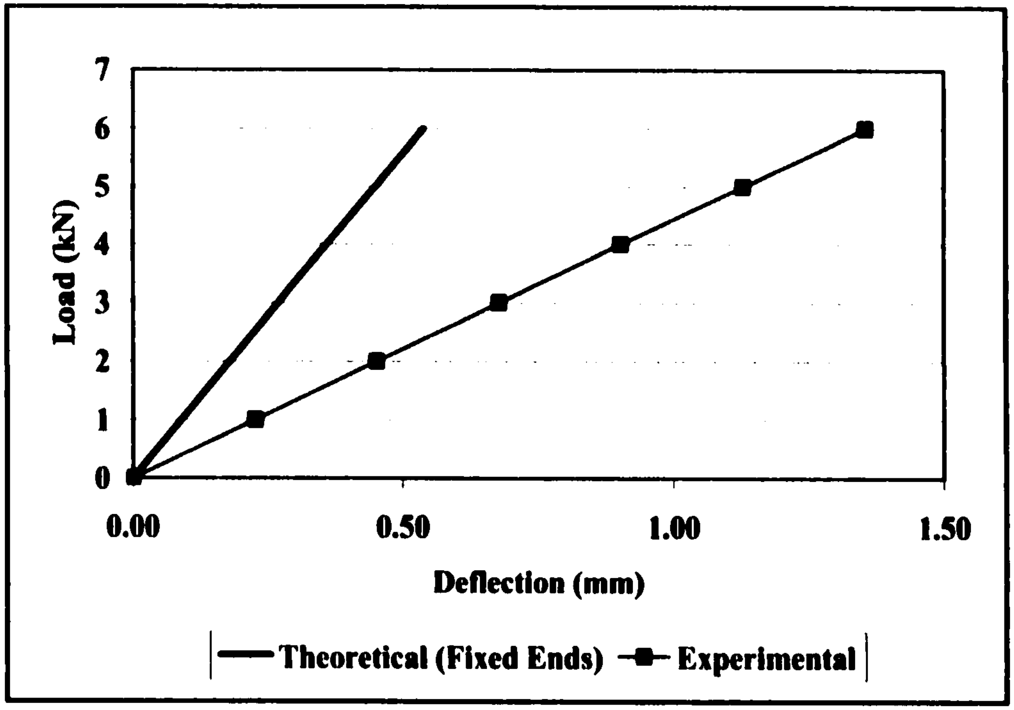

Figure A.24 Load vs. Deflection data for chord member - Specimen P8 (A)

Load Applied	Experimental deflection	Theoretical deflection (Fixed Ends)
kN	mm	mm
0.0	0.00	0.00
1.0	0.21	0.09
2.0	0.43	0.18
3.0	0.64	0.27
4.0	0.86	0.36
5.0	1.07	0.45
6.0	1.28	0.54

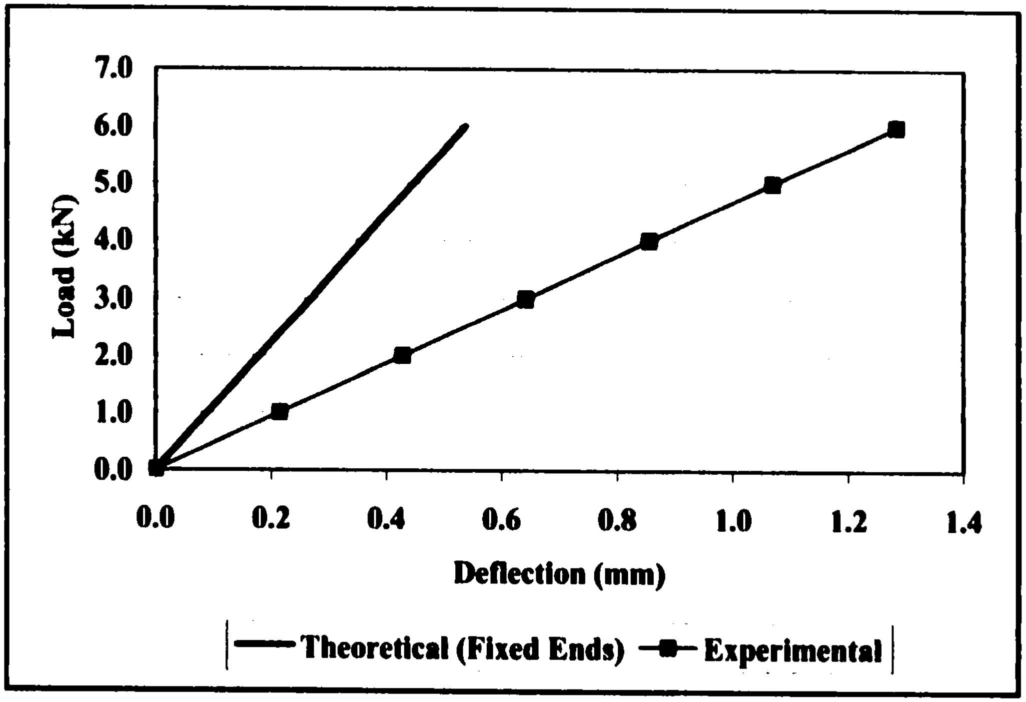

Figure A.25 Load vs. Deflection data for chord member - Specimen P8 (B)

APPENDIX – B

ABAQUS INPUT FILES

B.1 ABAQUS INPUT S-1 (A).txt

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ERI TOWER
SI Units (mm, N)
*RESTART, WRITE
**Model Definition
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2, 0., 0., 914.4
3, 0., 791.718, 457.2
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9, 1524., 0., 0.,
10, 2286., 0., 0.,
11, 2286., 0., 914.4,
12, 2286., 791.718, 457.2,
13, 3048., 0., 0.,
14, 3048., 0., 914.4,
15, 3048., 791.718, 457.2,
16, 3810., 0., 0.,
17, 3810., 791.718, 457.2,
18, 4572., 0., 0.,
19, 4572., 0., 914.4,
20, 4572., 791.718, 457.2,
21, 381., 395.732, 228.6,
22, 1143., 395.732, 228.6,
23, 1905., 395.732, 228.6,
24, 2667., 395.732, 228.6,
25, 4191., 395.732, 228.6,
26, 381., 395.732, 685.8,
27, 1143., 395.732, 685.8,
28, 1905., 395.732, 685.8,
29, 381., 0., 457.2,
30, 1143., 0., 457.2,
31, 1905., 0., 457.2,
32, 2667., 0., 457.2,
33, 3429., 0., 457.2,
34, 4191., 0., 457.2,
35, 2667., 395.732, 685.8,
36, 3429., 395.732, 685.8,
37, 4191., 395.732, 685.8,
38, 3810., 0., 914.4,
39, 3429., 395.732, 228.6,
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3, 8, 11
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8, 6, 9
9, 9, 12
10, 12, 15
11, 15, 17
12, 17, 20
13, 1, 4
14, 4, 7
18, 16, 18
103, 13, 40
104, 40, 10
108, 4, 9
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19.05

*ELEMENT, TYPE=B31, ELSET=M2
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35, 16, 34
36, 34, 19
37, 18, 34
38, 34, 38
39, 38, 33
40, 33, 13
41, 13, 32
42, 32, 11
43, 11, 31
44, 31, 7
45, 7, 30
46, 30, 5
47, 5, 29
48, 1, 29
49, 1, 29
50, 21, 6
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56, 22, 9
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59, 7, 27
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91, 26, 5
92, 26, 5
93, 22, 8
94, 8, 23
95, 28, 11
96, 11, 35
97, 35, 15
98, 15, 36
99, 36, 38
100, 38, 37
101, 37, 20
105, 2, 5
106, 5, 7
107, 7, 11
109, 30, 9
110, 31, 9
111, 21, 1
112, 6, 8
113, 8, 12
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6.35

*ELEMENT, TYPE=B31, ELSET=M3
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22, 20, 18
24, 19, 20
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9.525

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23,18,19,20
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9.525

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200000., 0.3
**
*STEP, PERTURBATION
6000N CONCENTRATED LOAD
*STATIC
*BOUNDARY
4, 2
5, 2
10, 2
13, 2
18, 1, 3
19, 1, 3
*CLOAD
40, 2, -6000
*ELPRINT
*NODE PRINT
U
*END STEP
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B.2 ABAQUS INPUT S-3 (A).txt

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ERI TOWER
SI Units (mm, N)
*RESTART, WRITE
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6, 762., 791.718, 457.2,
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18, 4572., 0., 0.,
19, 4572., 0., 914.4,
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21, 381., 395.732, 228.6,
22, 1143., 395.732, 228.6,
23, 1905., 395.732, 228.6,
24, 2667., 395.732, 228.6,
25, 4191., 395.732, 228.6,
26, 381., 395.732, 685.8,
27, 1143., 395.732, 685.8,
28, 1905., 395.732, 685.8,
29, 381., 0., 457.2,
30, 1143., 0., 457.2,
31, 1905., 0., 457.2,
32, 2667., 0., 457.2,
33, 3429., 0., 457.2,
34, 4191., 0., 457.2,
35, 2667., 395.732, 685.8,
36, 3429., 395.732, 685.8,
37, 4191., 395.732, 685.8,
38, 3810., 0., 914.4,
39, 3429., 395.732, 228.6,
40, 2667., 0., 0.,
*ELEMENT, TYPE=B31, ELSET=M1
1, 2, 3
2, 5, 8
3, 8, 11
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5, 14, 38
6, 38, 19
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9, 9, 12
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11, 15, 17
12, 17, 20
13, 1, 4
14, 4, 7
18, 16, 18
103, 13, 40
104, 40, 10
108, 4, 9
*BEAM SECTION, ELSET=M1, MATERIAL=STEEL, SECTION=CIRC
25.4

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36, 34, 19
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98, 15, 36
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100, 38, 37
101, 37, 20
105, 2, 5
106, 5, 7
107, 7, 11
109, 30, 9
110, 31, 9
111, 21, 1
112, 6, 8
113, 8, 12
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7.15

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21, 2, 3
22, 20, 18
24, 19, 20
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9.525

*ELEMENT, TYPE=B31, ELSET=M4
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23,18,19,20
*BEAM SECTION, ELSET=M4, MATERIAL=STEEL, SECTION=CIRC
9.525

*MATERIAL, NAME=STEEL
*ELASTIC
200000., 0.3
**
*STEP, PERTURBATION
6000N CONCENTRATED LOAD
*STATIC
*BOUNDARY
4, 2
5, 2
10, 2
13, 2
18, 1, 3
19, 1, 3
*CLOAD
40, 2, -6000
*ELPRINT
*NODE PRINT
U
*END STEP
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B.3 ABAQUS INPUT S-5 (A).txt

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ERI TOWER
SI Units (mm, N)
*RESTART, WRITE
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5, 762., 0., 914.4,
6, 762., 791.718, 457.2,
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8, 1524., 791.718, 457.2,
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22, 1143., 395.732, 228.6,
23, 1905., 395.732, 228.6,
24, 2667., 395.732, 228.6,
25, 4191., 395.732, 228.6,
26, 381., 395.732, 685.8,
27, 1143., 395.732, 685.8,
28, 1905., 395.732, 685.8,
29, 381., 0., 457.2,
30, 1143., 0., 457.2,
31, 1905., 0., 457.2,
32, 2667., 0., 457.2,
33, 3429., 0., 457.2,
34, 4191., 0., 457.2,
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38, 3810., 0., 914.4,
39, 3429., 395.732, 228.6,
40, 2667., 0., 0.,
*ELEMENT, TYPE=B31, ELSET=M1
1, 2, 3
2, 5, 8
3, 8, 11
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4, 11, 14
5, 14, 38
6, 38, 19
7, 3, 6
8, 6, 9
9, 9, 12
10, 12, 15
11, 15, 17
12, 17, 20
13, 1, 4
14, 4, 7
18, 16, 18
103, 13, 40
104, 40, 10
108, 4, 9
*BEAM SECTION, ELSET=M1, MATERIAL=STEEL, SECTION=CIRC
34.95

*ELEMENT, TYPE=B31, ELSET=M2
25, 2, 29
26, 29, 4
27, 4, 30
30, 31, 10
31, 10, 32
32, 32, 14
33, 14, 33
34, 33, 16
35, 16, 34
36, 34, 19
37, 18, 34
38, 34, 38
39, 38, 33
40, 33, 13
41, 13, 32
42, 32, 11
43, 11, 31
44, 31, 7
45, 7, 30
46, 30, 5
47, 5, 29
48, 1, 29
49, 1, 29
50, 21, 6
53, 4, 21
54, 21, 3
55, 4, 22
56, 22, 9
58, 28, 7
59, 7, 27
60, 23, 12
61, 12, 24
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62, 24, 13
63, 13, 39
64, 39, 17
65, 17, 25
66, 25, 18
69, 25, 16
70, 25, 20
71, 6, 22
72, 9, 23
73, 23, 10
74, 10, 24
75, 24, 15
76, 15, 39
77, 39, 16
78, 19, 37
79, 37, 17
80, 17, 36
81, 36, 14
82, 14, 35
83, 35, 12
84, 12, 28
85, 28, 8
86, 8, 27
87, 27, 6
88, 6, 26
89, 26, 2
90, 3, 26
91, 26, 5
92, 26, 5
93, 22, 8
94, 8, 23
95, 28, 11
96, 11, 35
97, 35, 15
98, 15, 36
99, 36, 38
100, 38, 37
101, 37, 20
105, 2, 5
106, 5, 7
107, 7, 11
109, 30, 9
110, 31, 9
111, 21, 1
112, 6, 8
113, 8, 12
*BEAM SECTION, ELSET=M2, MATERIAL=STEEL, SECTION=CIRC
7.95

*ELEMENT, TYPE=B31, ELSET=M3
19, 3, 1
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21, 2, 3
22, 20, 18
24, 19, 20
*BEAM SECTION, ELSET=M3, MATERIAL=STEEL, SECTION=CIRC
9.525

*ELEMENT, TYPE=B31, ELSET=M4
20,1,2,3
23,18,19,20
*BEAM SECTION, ELSET=M4, MATERIAL=STEEL, SECTION=CIRC
9.525

*MATERIAL, NAME=STEEL
*ELASTIC
200000., 0.3
**
*STEP, PERTURBATION
6000N CONCENTRATED LOAD
*STATIC
*BOUNDARY
4, 2
5, 2
10, 2
13, 2
18, 1, 3
19, 1, 3
*CLOAD
40, 2, -6000
*ELPRINT
*NODE PRINT
U
*END STEP
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VITA AUCTORIS

Adnan Karim Qureshi was born in 1971 in Hala, a city of Province of Sind, Pakistan. He graduated from Public School Hyderabad in 1988. From there he went to Mehran University of Engineering & Technology, Jamshoro, Pakistan, where he obtained his Bachelor's with Honors in Civil Engineering in 1995. After his Bachelor's he worked for about three years in the field of Civil Engineering in Pakistan and in Norway.

He is currently a candidate for the Master's degree in Structural Engineering at the University of Windsor and hopes to graduate in Fall 1999.