STUDY ON IMPROVEMENTS TO THE STRUCTURAL PERFORMANCES AND COST OPTIMIZATION OF EXISTING TELECOMMUNICATION POLES

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Master in Engineering

Structural Engineering Designs

Department of Civil Engineering

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Sri Lanka

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This thesis is submitted to the Department of Civil Engineering of the University of Moratuwa, Sri Lanka in partial fulfilment of the requirement of Degree of Master of Engineering in Structural Engineering Designs

Department of Civil Engineering

University of Moratuwa,
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DECLARATION

"The work included in this report was done by me, and only by me, and the work has not been submitted for any other academic qualification at any institution".

Name: W.A.K Basil Kumara

Date: 26-02-2016

"I certify that the declaration above by the candidate is true to the best of my knowledge and that this dissertation is acceptable for evaluation for the Degree of Master of Engineering in structural engineering designs".

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Senior Lecturer

University of Moratuwa

Date: 26-02-2016

ABSTRACT

Presently SLT invests huge capital on production of RCC poles to draw various types of telecommunication cables and hence explores the possibilities of minimizing the investment on production of RCC poles by introducing economized pole design or modifying the existing RCC poles with less production cost.

There are numerous problems associated with conventional telecommunication posts currently being used in practice such as prone to corrosion in coastal belt areas. Much difficult to maintain uniform quality throughout the pole length due to manual practices of concrete mixing, bar bending, formwork etc., this will further aggravate due to lack of skilled personals for operation. In such a situation life span of the final products are doubtful. Manufacturing processes are often been carried out by sub-contracting labour groups so that they can produce maximum number of poles in very limited period to cater for the demand. At the same time they are trying their best to make maximum profit, result is sub standard products.

Sri Lanka Telecom currently invest huge sum of money for the production of telecommunication posts in an yearly basis as demands are high, therefore Sri Lanka Telecom is compelled to invest additional amount to investigate and overcome the above problems in sought of producing good quality products with optimum cost.

External forces encountered by the telecom poles are bending, axial forces, torsion forces or a combination of those three forces. These primary influences may be accompanied by shearing forces and sometimes by torsion. Effects due to changes in temperature, shrinkage, creep of the concrete, and the possibility of damage resulting from overloading, local damage, abrasion, vibration, chemical attack and similar causes may have also to be considered. An efficiently designed poles are one in which the weight, loads and forces are transmitted to the foundations by the economical means consistent with the intended use of the pole and the nature of the ground situation.

The objective of this research work is to study on current designs and their pros and cons, applicable standards, manufacturing process and simulation of structural performance of poles under recommended loading criteria by modern analytical tools.

At the initial stage of study variety of sections are analysed under applicable loading criteria to select most suitable one. Sections considered for analysis are square solid section with pre-stressed reinforcement, square hollow section with normal and pre-stressed reinforcement, circular section with normal and pre-stressed reinforcement, circular hollow section with normal and pre-stressed reinforcement. SAP2000 finite element programme was used to analyse the poles under different loading conditions as specified. Circular hollow section with pre-stressed reinforcements gave the best option and optimum results for the requirements with respect to structural performances, weight and cost.

As a result of this research study, most economical solution has been recommended to overcome the above difficulties. Usual casting practices are revised to spun casting technique with pre-stressed reinforcements and high grade concrete. This will result in finding the superior structural performances, high quality, comparatively low cost and less weight product. Few samples of the new designs are cast and been tested to witness their structural behaviours under the laboratory conditions.

Finally the current and proposed designs are compared to demonstrate the weight reduction. Structural details and specifications for new designs are prepared under different height category of poles to suit manufacturing facilities.

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I would like to dedicate this hard work to my parents, my spouse and son for their patient support and encouragement extended to me for the success of this research. There are many who helped me to succeed in education from my childhood to date. I regret for my inability to thank them individually but I offer my heartiest thanks to all of them.

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1.1. Introduction

Background

A pole generally acts as a cantilevered structure, and should be designed and analyzed as a tapered member with combined axial and bending loads. Because shear forces are small compared to bending moments, pre-stressed concrete poles are very resilient. Axial loads are small too, and are generally ignored except when the structure is guyed. Stresses induced by handling, transportation and erection should always be considered in design. Common lifting points are the third points or in some cases the centre of gravity. Weight of cross arms and other attachments should not be overlooked in calculating the centre of gravity. Unlike other pre-stressed concrete structures, a pre-stressed concrete pole has to withstand equal bending moments in opposite directions. Poles should be designed to withstand the maximum loading condition, including design overload factors, without exceeding the ultimate strength of the pole, and under normal working conditions without exceeding the cracking moment of the pole.

1.2. Scope of the project

1.2.1. Study on current designs, code requirements and manufacturing process.

1.2.2. Simulation of structural performance of various pole types under applicable loading criteria by modern analytical tools as well as first hand calculations.

1.2.3. Finding ways and means to optimizing the cost and improvement of structural performance by making changes to physical shape, design and casting practices, concrete grade and reinforcement steel etc.

1.2.4. Sample testing and verifications.

1.3. Design aspect

Most of the telecom poles are subjected to bending or to direct force (either tensile or compressive) and torsional forces or to a combination of bending, torsional and direct force. These primary influences may be accompanied by shearing forces and sometimes by torsion. Effects due to changes in temperature and to shrinkage and creep of the concrete, and the possibility of damage resulting from overloading, local damage, abrasion, vibration, frost, chemical attack and similar causes may also have to be considered. Design of poles includes the calculation of, or other means of assessing and providing resistance against, the moments, forces and other effects on the members. Efficiently designed poles are one in which the weight, loads and forces are transmitted to the foundations by the cheapest means consistent with the intended use of the pole and the nature of the ground situation. Efficient design means more than providing suitable sizes for the concrete members and the provision of the calculated amount of reinforcement in an economical manner.

1.4. Economic design and cost optimization

The cost of a reinforced or pre-stressed concrete structure is obviously affected by the prices of concrete, steel, formwork and labour. Upon realization between these prices, the economical proportions of the quantities of concrete, reinforcement and framework will be depend on the final product cost. There are possibly other factors to be taken into account in any particular case such as, whether less concrete of a rich mix is cheaper than a greater volume of a leaner concrete; whether the cost of higher priced bars of long lengths will offset the cost of the extra weight used in lapping shorter and cheaper bars; or whether uniformity in the sizes of members saves in formwork what it may cost in extra concrete, etc. There is also a wider aspect of economy, such as whether the anticipated life and use of a proposed structure warrant the use of a higher or lower factor of safety than is usual, whether the extra cost of an expensive type of construction is warranted by the improvement in facilities, or whether the initial cost of a construction of high quality with little or no maintenance cost is more economical than less costly construction combined with the expense of maintenance. The working methodologies, experiences, the location of the fabrication yards and the nature of the available materials, and even the method of measuring the quantities, together with numerous other points, all have their effect on economy of the structures. An essential aspect of economical design is an appreciation of the possibilities of materials other than concrete. The proper combination of such materials may lead to substantial economies. Included in such economic comparisons should be such factors as fire resistance, deterioration, depreciation, insurance, appearance and speed of construction, and structural considerations such as the weight on the foundations, convenience of construction and the scarcity or otherwise of materials. Recent advancement and availability of cheap computing technology have provided powerful tools for analysis and design of concrete structures and computer model for optimization problems. The advantages of computer simulation methods have been incorporated in the optimization of concrete design and mixture proportions etc.

This research is mainly focus on finding a new solution to overcome the above difficulties. Usual casting practices will be diverted to spun casting technique with pre-stressed reinforcements and high grade concrete. Both manual calculations and finite element model analysis are made to finalizing the physical shape, concrete grade requirements and reinforcement details. As a result we were able to find superior structural performances, high quality, comparatively low cost and less weight product to satisfy with service requirements. Few physical samples of the new designs were cast and been tested to witness the structural behaviours such as crack patterns, deflections, ultimate load at fracture under laboratory conditions.

Finally current and proposed designs are compared to witness the weight reduction. Structural details and specifications for new designs are prepared under different height category of poles for manufacturing facilities.

1.5. Outline of the thesis

The contents of the thesis are briefly outlined below,

Chapter 2: Presents a review of literature related to previous research on concrete poles and the spun-cast process with pre-stresses. Applicable codes and standards, geometrical properties, testing procedure, material properties and design criterion are also covered under this section.

Chapter 3: Provides a detailed description of the methodologies adopted, proposed specifications, structural calculation, reinforcements and tendon details and finite element model analysis.

Chapter 4: Provides the manufacturing process, main advantages of using pre-stressed spun cast technology, comparison between existing and proposed poles

Chapter 5: Presents the laboratory testing, results and observations.

Chapter 6: Presents the conclusion and future works.

CHAPTER 2

2.1. Introduction

This chapter provides an overview of various aspects of conventional casting process and spun-cast concrete poles, as used in utility and street lighting applications. It also summarizes applicable codes and standards as well as testing procedures.

2.2. Literature Review

2.2.1. Study the current design and production practice adopted by the Sri Lanka Telecom, future requirements and applicable codes of practices.

2.2.2. Cost analysis in the current production process.

2.2.3. Cost Parameters will define the parameters in the RC design process that affect the production cost per unit. This will inclusive of dimensions of a unit, area of reinforcement, steel ratios and different grades of concrete and reinforcements etc.

2.2.4. Study the various techniques and standards used in other countries for the production of economical concrete poles.

2.3. Applicable codes and standards

Design shall be in compliance with the BS 607: Part-2: 1970 [3], SLS 363: 1975 [2] and BS8110 [5] requirements of reinforced concrete and pre-stressed concrete. Various laboratory test data will be utilized for cost optimizations. This will include the structural design, quantity computations, cost estimation and cost comparison analysis.

2.4. Dimensions:

2.4.1. Shape:

The poles shall generally be square in section and they may be of uniform cross section throughout their length or tapering along their lengths on all four faces. The cross sectional dimensions shall be adequate to conform to strength requirements mentioned below unless otherwise specified by the purchaser and provided the strength requirements are full filled, dimensions given below shall be used.

2.4.2. Standard Lengths:

The poles shall be following lengths

1. 5.6m

2. 6.7m
3. 7.5m
4. 8.0m and
5. 9.0m

2.4.3. Standard Sections:

1. 140mm x 140mm
2. 180mm x 180mm
3. 185mm x 185mm
4. 190mm x 190mm
5. 205mm x 205mm

2.4.4. Tolerances:

Tolerance on length = ± 15mm

Tolerance on cross sectional dimensions = ±3mm

Tolerance on straightness = 0.5 percent

2.5. Ultimate Transverse Loads

Sri Lanka Telecom has specified the minimum ultimate transverse load in their specifications for OSP material [1]. Loads values are shown in the Table 1 below,

Table 1: Minimum recommended ultimate transverse loads

Length of Pole (m)	Minimum ultimate transverse load at 0.5m from top (kN)
5.6	2.6
6.7	3.6
7.5	3.6
8.0	3.6
9.0	3.6

The working load shall be taken as 40% of the ultimate load.

Figure 1 shows the typical detail of square type pre-cast concrete pole used in conventional practice.

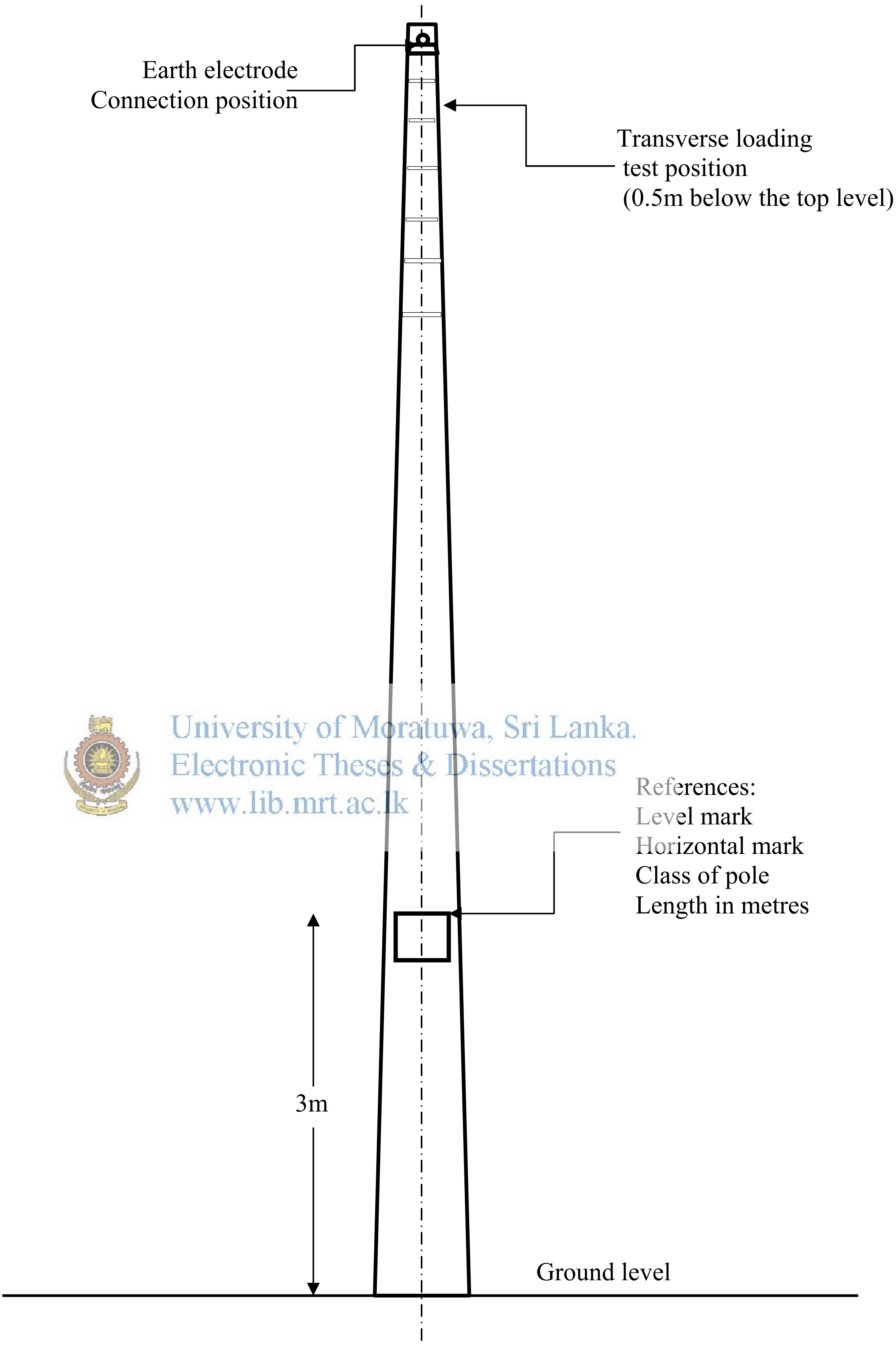


Figure 1: Typical details for concrete pole

2.6. Manufacturer's cost analysis for current pre-casting concrete poles

Rate analysis submitted to the Sri Lankan Telecom by one of the sub-contracting company on October 2012 are summarized under following section,

2.6.1. 5.6m High Pole: Rs.4,142.59, cost for unit length is Rs.739.75

6.7m High Pole: Rs.7,082.82, cost for unit length is Rs.1,057.15

7.5m High Pole: Rs.8,226.23, cost for unit length is Rs.1,096.85

8.0m High Pole: Rs.8,688.59, cost for unit length is Rs.1,086.10

9.0m High Pole: Rs.10,062.53, cost for unit length is Rs.1,118.05

Rate breakdown for the above cost analysis are given in Appendix-G

2.7. Standards and specifications issued by the Sri Lanka Telecom

This section is reference to the standards and specifications issued by the Sri Lanka Telecom for the production of concrete poles given in "Specification for OSP Material" [1].

2.7.1. General

This denotes the reinforced concrete pole. Basically, the concrete pole shall conform to BS607 of part-2:1970 [3] or subsequent editions relating to reinforced concrete poles.

Further, conditions laid down below have to be satisfied.

2.7.2. Material

Fine Aggregate

a. This shall consist of river sand

b. The fine aggregate shall be uniformly graded and shall meet the grading requirement as shown below in Table 2.

Table 2: Grading requirements for fine aggregates [1]

Sieve Designation(mm)	% By weight passing square mesh sieve
10	100
5	95-100
1.2	45-80
0.3	10-30
0.15	2-10

The fine aggregate shall be stored in such a manner as to prevent mixing with other aggregate or foreign material/bodies.

2.7.3. Coarse aggregate

This shall consist of gravel and or crushed stones having hard, strong and durable pieces free from adherent coating such as mud or any other slug/material.

The coarse aggregate shall be graded between a maximum size of 25mm and minimum size corresponding to 5mm sieve size. It must be free from dirt, flowing stones dust, earth or any similar material.

2.7.4. Water

All water for mixing and curing of concrete shall be from the public supply. Water from any other source may be used only with the total approval of the Sri Lanka Telecom. Water contaminated with dirt, oil, or any foreign material shall not be used.

2.7.5. Reinforcement Bars

Reinforcement bars shall be round deformed steel bars. They shall be free from dirt, oil paint, rust, grease etc., and shall confirm the following requirements

- Minimum tensile stress - 500N/mm^2
- Minimum yield point - 300N/mm^2
- Minimum elongation - 14%
- Design stress - 160N/mm^2

2.7.6. Composition of concrete

The concrete, when made with ordinary Portland cement, shall attain a minimum compressive strength of 18N/mm^2

2.7.7. Process

Mixing

The whole of the concrete shall be mixed together first in a dry state and there after proper proportion of clean water to ensure maximum density. The quantity of water shall be the minimum, which after thorough mixing will produce a stiff plastic mass of even colour.

Setting

The concrete shall be used soon after mixing. Not more than 30 minutes shall be allowed between the first wetting time of concrete mixture and the subsequent placing in the mould. Tamping, pressure or other effective methods shall be used to consolidate the material within the mould. After such consolidating, it shall not be disturbed during the period of setting, which would be a minimum of 24 hours.

Maturing

The poles shall be cast in the curing tank. After the setting period is over, the moulds will be dismantled and the tank filled with water to completely submerge the poles and for a continuous period of 7days.They shall be then stacked and kept wet for a further 14days.

Earthing

Electrical continuity shall be provided from the socket (to take in the cap spindle) on the pole top to the bottom of the pole. For this purpose, the cap socket shall be connected to the reinforcing framework by wire and one of the reinforcing bars, arrange to protrude at a point 9 inches from the bottom of the pole.

2.7.8. Sample selection

One in every 10 poles or part there of delivered shall be selected for testing

2.7.9. Testing methods

The testing method is based on SLS 363 of 1975 [2] with the modification as described in this section. The load shall be applied at a point 50cm from the top of the pole.

The Table 3 shows the ultimate and breaking loads values.

Table 3: Ultimate and breaking loads for poles [1]

Total height of pole (m)	Height of pole below ground level in metres (Ground level marker)	Minimum Ultimate transverse load in kN applied 0.5m from the top of pole	Breaking load kN
5.6	1.1	2.60	3.64
6.7	1.34	3.60	5.04
7.5	1.50	3.60	5.04
8.0	1.60	3.60	5.04
9.0	1.80	3.60	5.04

2.7.10. Testing procedure

This testing procedure is meant for 100 poles batch and any 100 poles portion of thereof to be tested as given below.

Select 10 poles out of 100 poles batch containing pole in every 10 poles concerned.

The sample 10 poles to be grouped into two (5 poles in each group) and tested for type test and proof test. That is one in every 20 poles should be subjected to type test and other pole selected in the same 20 pole category to be tested for proof test.

2.7.11. Type test

Unless otherwise specified with the enquiry or order, a written statement that the number of poles specified in Table 4: Numbers of test requirements for poles. Identical in all essential features of design with those purchased, have passed the type test shall be deemed to be sufficient evidence that the poles comply with the requirement of specifications for OSP Material [1] and SLS 363 of 1975 [2].The statement shall give the results of all tests and state the age of the poles when tested. The casting schedule of poles shall be available at site before testing.

The poles selected for the test shall be tested in accordance with appendix A of the SLS 363 of 1975 [2].The permanent set after removal of a test load of 60% of the minimum ultimate load specified in Table 4 shall not exceed 10% of the deflection at the test load. The hair cracks produced in testing shall clearly close up on removal of the test load specified. The test load at failure shall exceed the minimum-breaking load specified in Table 3

2.7.12. Proof test

The poles shall be tested in accordance with appendix A of SLS 363 of 1975 [2] except that the minimum load applied shall equal to 40% of the ultimate load specified in Table 3.The deflection at measurement, and the permanent set after removal of test load, shall not exceed by more than 15% the average of corresponding values for the poles subjected to the type test.

2.7.13. Samples and inspection

a. In any batch, all poles of the same dimensions shall be grouped together to constitute a lot.

b. If the number of poles in a lot exceeds 500, the lot shall be divided to suitable number of sub-lots, such that the number of poles in any sub-lot shall not exceed 500.

c. The sample size shall be made up of poles selected at random from lot or sub lot.

2.7.14. Number of test

The number of poles to be tested for dimensional requirements (overall length, cross section and uprightness) and strength shall be in accordance with the Table 4 shown below.

Table 4: Numbers of test requirements for poles [1]

Size of lot or sub lot up to 100	Dimensional requirements		Number of poles for strength test	
	Sample size	Permissible defectives	Type test	Proof test
01-100	10	01	05	05

2.7.15. Criterion for conformity

The number of poles, which do not satisfy requirements of overall length, cross-section and uprightness, shall not exceed the corresponding number given in the above Table 4.

2.7.16. Rejection

All poles for strength test satisfy the requirements of the test. If first pole for type test passes (i.e. reaching the breaking load as per the Table 3) all the other four poles for type test loaded only up to minimum ultimate load as per Table 3 and accept as satisfactory for type test. If the first pole for type test fails, all the other four poles for type test to be loaded for breaking load.

The following criteria will be then applied for the accepting/rejecting of poles.

a. Only first pole fails – Reject the twenty (20) poles containing the tested pole

b. If another one pole fails – Reject that twenty (20) poles containing the tested pole too

c. If another one pole fails – Reject the entire hundred (100) poles batch

Figure 2 shown in below shows the general flow chart for type test procedure as described in paragraph 2.7.11.

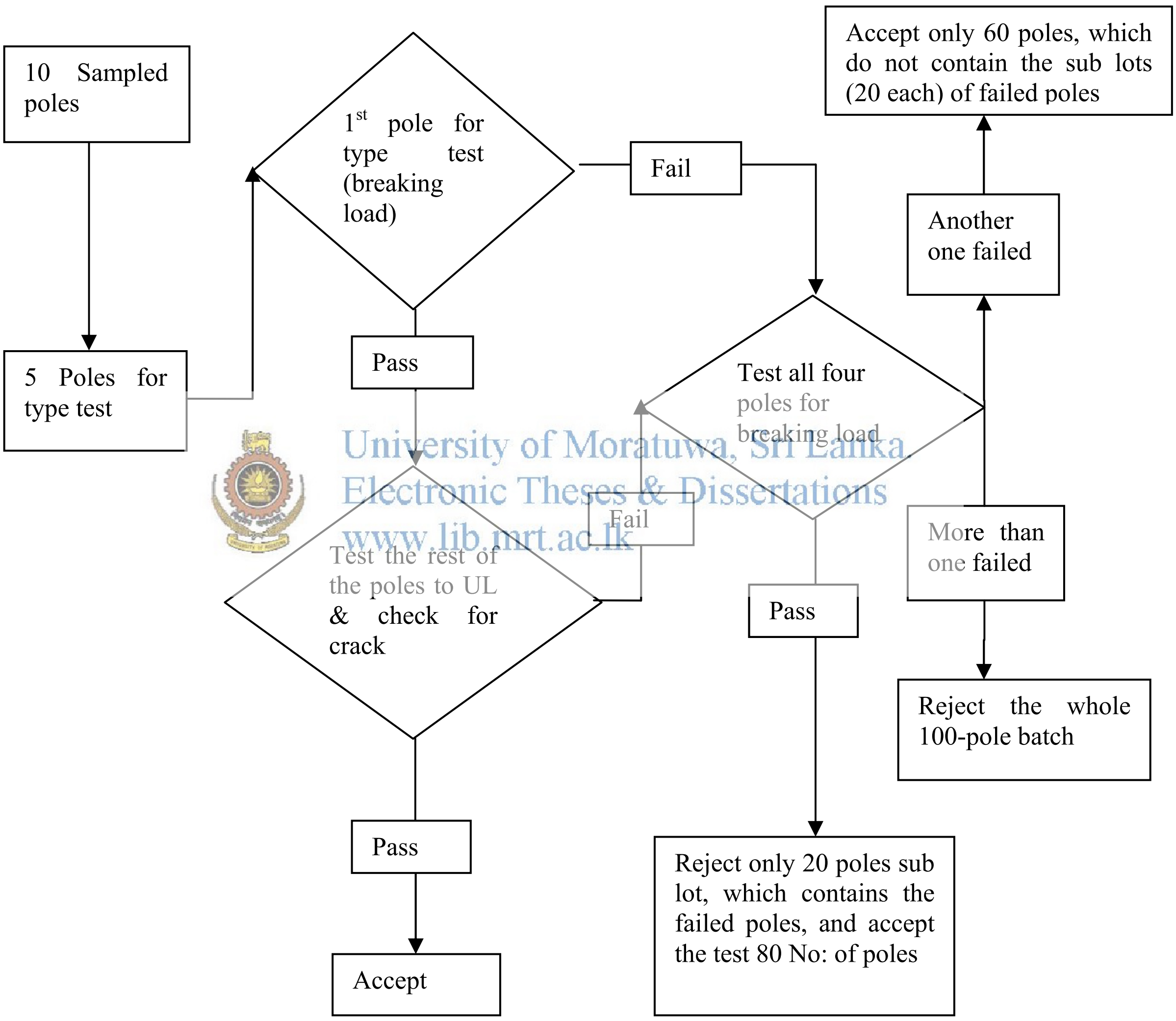


Figure 2: General flow chart for type test procedure

2.7.17. Finish

a. The poles shall have a smooth finish

b. The poles shall be free from any damage, e.g. broken corners, hair cracks etc,

c. The holes for arm bolts shall not be fouled by concrete, cement etc,

d. The bolt fixed on the cap shall be removable.

2.7.18. Marking

Identification marks

For the purpose of identification, the following markings shall be provided at a height of 3 metres from the pole.

a. SLT Logo

b. Height of pole in metres

c. Manufacturers code

d. Manufacturers serial number

e. Year of manufacture

f. The sizes of the figures and letter shall be 25mm by 25mm the depth of the lettering shall be 3mm.

2.7.19. Pole depth identification marks

For the purpose of safety, the following marking shall be provided at a height given in the Table 5.

a. It is required to mark two marks having a width of 50mm in one side of pole.

b. The colour shall be red

c. The depth of the engraved mark shall be 3mm

Table 5: Identification marks [1]

Total height of pole (m)	Height of the first indicator from bottom end of the pole (m)	Height of the second indicator from bottom end of the pole(m)
5.6	1.12	2.12
6.7	1.34	2.34
7.5	1.50	2.50
8.0	1.60	2.60
9.0	1.80	2.80

2.8. Pre-stressed concrete poles

2.8.1. Specifications for pre-stressed concrete poles

This section is references to the standards and specifications given in BS 607: Part 2:1970 [3] for pre-stressed concrete poles.

a. Concrete

For ordinary and rapid hardening Portland cement concrete, including coloured Portland cement concrete, Portland-blast furnace cement concrete or sulphate resisting Portland cement concrete the proportion of cement to total aggregate shall be not less than 1:5 by weight and the minimum works cube strength at 28 days when made, cured and tested in accordance with the requirement of BS 1881 (Methods of testing concrete) shall be not less than $40MN/m^2$ [33].

For high alumina cement concrete, the proportion of cement to total aggregate shall be not less than 1:6 by weight and the minimum works cube strength at 2 days when made, cured, and tested in accordance with the requirements of BS 1881 [33] shall not be less than $40MN/m^2$.

b. Pre-stressing steel

The pre-stressing steel shall be plain hard-drawn steel wire complying with the requirements of BS 5896 [34] or other wire having properties not inferior to those laid down in BS 5896 [34], or multi-wire strand for which a stress strain diagram has been established. Wires or bars shall be free from grease or other material likely to impair the bond. The steel shall be free from pitting.

Note: A slight film of rust is not necessarily harmful and may improve the bond.

c. Pre-tensioning and release of wires

All wires which are stressed in one operation shall be of the same nominal length except that the use of multi-wire strands shall not be excluded. The necessary elongation shall be directly determined by measuring the stretching force and elongation of the pre-stressing wire prior to concreting. In all cases the yield of the gripping devices shall be taken into account and the accuracy of the equipment used for determining the pre-stressing force shall be checked at least once every 14 days.

Between tensioning and release (i.e. during the setting and hardening of the concrete) the tension shall be fully maintained by some positive means.

The stretched wire shall not be released until the strength of the concrete in the column has attained a value of $27MN/m^2$.

Release of the pre-stressing wire shall be arranged in such a manner as to prevent any damage to the bond of these wires in the concrete. Severe eccentricity of stress in the concrete section shall be avoided.

d. Moulds

When moulds are required to withstand the pre-stressing force, they shall be sufficiently rigid to carry these forces without distortion.

e. Un-tensioned steel reinforcement

Steel reinforcing bars shall be free from loose rust, scale, oil or grease and shall comply with the requirements of BS 4449 [35] or BS 4466 [36] or BS 4482 [37] as appropriate.

2.8.2. Type test

The poles shall be tested in accordance with section 2.8.6

No visible hair cracks shall occur at a test load of 60% of the minimum ultimate load specified in Table 3.

The permanent set after the removal of test load of 60% of the minimum ultimate load specified in Table 3 shall not exceed 7.5% of the deflection at the test load.

The test load at failure shall exceed the minimum ultimate load specified in Table 3.

2.8.3. Proof tests

The poles shall be tested in accordance with section 2.8.6 except that the maximum load applied shall be equal to 40% of the ultimate load specified in Table 1.

The deflection of each measurement, and the permanent set after removal of test load, shall not exceed by more than 15% the average of the corresponding values for the three poles subjected to the type test.

No visible hair cracks shall occur during the test.

2.8.4. Permissible stresses

For the preliminary stretching of hard-drawn wires complying with the requirements of BS 2691 the tensile stress shall not exceed 70% of the tensile strength as defined in BS 18.

The final working stresses in the concrete shall not exceed:

1. Compressive stress $u_w/3$
2. Tensile stress $u_w/30$
3. Principal tensile stress at the section of maximum shear $u_w/60$

Where u_w is the strength of the concrete in the product at 28 days.

For the purpose of calculation, the modulus of elasticity of the hard drawn pre-stressing wire may be taken as $195x10^3 MN/m^2$ and the modular ratio shall be calculated from Table 6.

Table 6: Modular ratio for different grades of concretes [3]

Cube strength of concrete at transfer MN/m^2	Modulus of elasticity of concrete 10^3 MN/m^2
27	27
35	32
40	34
55	41

The whole section shall be regarded as homogeneous and the distribution of the bending stress across any section shall be assumed to be linear. When calculating the initial stresses due to pre-compression of the concrete, the concrete section only shall be taken into account.

The losses of pre-stress in the steel shall be calculated on the basis of the following assumptions:

a. That the relaxation of stress due to creep of steel for hard drawn steel wire in the "as drawn" condition is $100MN/m^2$ but this loss may be reduced to $70MN/m^2$ when the wire is straightened and subsequently treated by the wire manufacturer to reduce the creep, or when the wire is overstressed by 10% of the initial stress for a period of 2 minutes during the tensioning operation.

b. That the loss of pre-stress due to elastic deformation of the concrete is the product of the modular ratio obtained from Table 6 and the stress in the adjacent concrete.

c. That the ultimate shrinkage of the concrete is 300x10-6 per unit length.

d. That where the strength of the concrete at transfer is greater than $40MN/m^2$ the creep of the concrete $48x10^{-6}$ per MN/m^2.For lower values of u_t, creep shall be assumed to be $48x10^{-6}x40/u_t$ per MN/m^2.Where u_t is the product strength at transfer

2.8.5. Pre-stressed concrete poles exposed to impact

a. Construction

Pre-stressed concrete poles which are to be sited where they are exposed to the risk of impact from vehicles shall comply with the requirements of section 2.8 and of this section.

b. Mild steel reinforcement

Longitudinal mild steel reinforcement shall be provided in the lower portion of the poles extending from 0.75m below ground level to the height specified by the purchaser, or in any case, not less than 1.8m above ground level. Effective means shall be provided for maintaining this reinforcement in position during the manufacture of the pole and all buttons and other devices used for this purpose shall be of rust proof material. Such reinforcement shall be spaced by means of transverse reinforcement to form a rigid cage.

The diameter of the transverse reinforcement shall not be less than 5mm and the spacing not more than 16 times the diameter of the un-tensioned longitudinal reinforcement.

c. Cross sectional area of mild steel reinforcement

Not less than four bars shall be used and the gross cross-sectional area of the reinforcement shall be not less than:

$314mm^2$ for poles of up to 11.0m length (i.e. 4x10mm diameter bars)

$452\ mm^2$ for poles of up to 15.8m length (i.e. 4x12mm diameter bars)

2.8.6. Structural test for poles [3]

A pole may be tested in either the horizontal or vertical position. Hold the pole rigidly at the butt end in accordance with the supported length specified in Table 7 (i.e. equal to the nominal depth of planting).

Table 7: Supporting length of poles

Length of pole (m)	Supported length (m)
8.0 – 9.2	1.5
9.8 – 12.2	1.8
13.4 – 15.8	2.1

If tested in the horizontal position, provision may be made by suitable supports to minimize the bending moment induced by the weight of the pole.

Apply the test load at a point 0.5m from the top of the pole and raise it in increments of 10% of the ultimate load. Take measurements of deflection after each increment and other measurements as given in detailed below as appropriate.

2.8.6.1. Type test

At 40% and 60% of the ultimate load reduce the load to zero and measure the permanent set. Then increase the load in steps of 10% of the ultimate load until failure occurs. Maintain each load above 60% of the ultimate load for at least 2 minutes

2.8.6.2. Proof test

At 40% of the ultimate load reduce the load to zero and measure the permanent set.

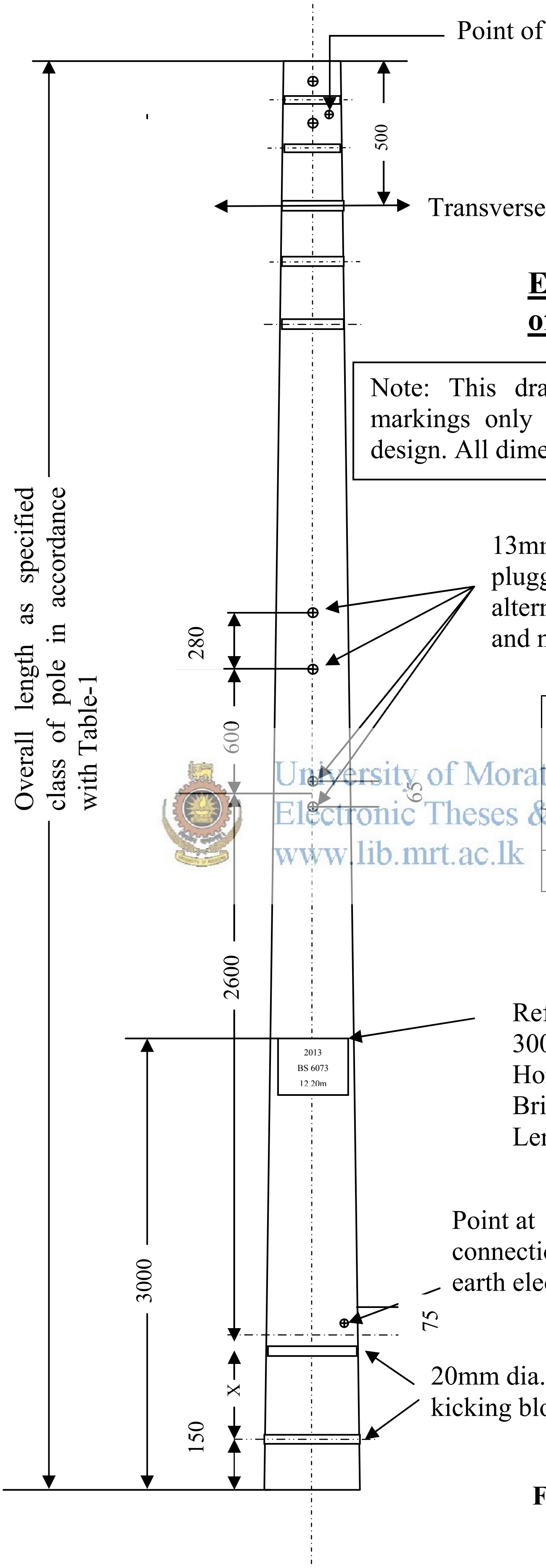
2.8.7. Recommendations for the provision of holes [3]

A typical arrangement of holes is shown in Figure 3, but other arrangements may be specified by the user.

The holes shown in the Figure 3 cater for the generally accepted design of high-voltage transmission intermediate poles and also for low-voltage distribution poles.

Elevation of typical pole in direction of line indicating holes and markings

Note: This drawing gives information on holes and markings only and should not be used as a basis for design. All dimensions in "mm" unless otherwise stated.



Overall length of pole	Dimension " X "
Up to 9.2m	0.9m
9.8m to 12.2m	1.2m
13.4m to 15.8m	1.5m

Figure 3: Details for marking on pole

2.9. Pre-stressed concrete poles

2.9.1. This section provides guidelines and a thought process recommended to be undertaken for the proper design and use of pre-stressed concrete poles by Pre-cast/Pre-stressed concrete institute, ASCE-PCI committee report [4].

Pre-stressed concrete poles were among the first application of pre-stressing that the French pre-stressing pioneer "Eugene Freysinet" developed in the 1930s. Today, pre-stressed concrete poles are used in most parts of the world as transmission and distribution structures, substation structures, lighting supports, highway sign and traffic signal structures, and communication structures.

In some countries, such as in India, concrete poles are used almost exclusively. In North America, their use is confined to specific regions such as the southeastern United States. Generally, where timber is plentiful, wood poles are used more often. However, the increased cost of wood and the environmental issues associated with the preservation of trees have resulted in an increased use of concrete poles. Therefore, the potential for a much greater use of precast pre-stressed concrete poles in the United States and throughout the world is promising.

There are two types of pre-stressed concrete poles. Spun cast and statically cast pre-stressed concrete poles. Within those types, several cross sectional shapes may be available. Spun cast poles are usually round, but may also be hexagonal, octagonal, or special architectural shapes. The most common shape for statically cast poles is square, although they may also be cast in octagonal, flanged I, or other special shapes.

Because it is inherent in the process, spun poles will always have a hollow core. The size of the hollow core is dependent on the wall thickness. Statically cast poles, however, may be solid or have a hollow core formed by the use of retractable mandrels or fiber voids.

2.9.2. Physical Characteristics

The basic pole structural configuration and location of all attachments should be made clear to the structural designer. However, the designer should be allowed as much latitude as possible to determine the design details of the structure.

2.9.3. Deflection

Limiting the deflection of a structure is sometimes necessary to ensure that clearance are maintained from the structure and its attachments to other objects, such as the edge of the right-of –way, building or bridges.

The appearance of the structure can also be affected by deflections. Sustained loads on structure may cause the pole to bow and be aesthetically unpleasant. To improve the appearance, a structure can be raked or the stiffness of the pole increased. Guys can also be installed to limit deflections. Stringent deflection limitations may increase the cost of the structure.

2.9.4. Decorative Applications

Many decorative colours, aggregates and textures may be specified for various architectural applications at additional cost. The colours are cast integrally throughout the pole during the manufacturing process. The pole surface may be polished to form a smooth terrazzo-like appearance or sand blasted to expose the aggregate and give a textured finish. Depending on the extend of the surface blasting the section properties may have to be modified.

2.9.5. Transportation and erection

The design of the structure should consider loads caused by loading, unloading, hauling, assembly, erection and stringing. The limitations of the handling equipment and the job site access should also be considered.

Concrete poles can be designed to be lifted or erected with one-point pick or may require multiple point-picks. Designing for a one-point pick without cracking may not be economical unless warranted by special conditions. The manufacturer should clearly indicate the proper procedure for handling, transporting and erecting the product.

2.9.6. Attached Items

The user, in this case Sri Lanka Telecom should inform the designer what accessories are to be mounted on the poles as well as the weight of those accessories so that the pole may be properly designed. Locations of bolt holes and inserts should also be provided. Holes or inserts within a hardware pattern, such as brackets, arms or X-braces, should be known to the manufacturers

2.9.7. Guying

Most of the Telecom Poles are designed as un-guyed poles. However it should be defined as many known conditions as possible, such as right-of-way limitations, size, grade, and allowable load of guys, guy angle limits, and quantity of guys, placement tolerances and terrain considerations.

2.9.8. Climbing and maintenance

One important concern for the user is the ability to climb the pole and access those areas of the pole where hardware is attached. The two most common climbing systems are step bolts and ladders. Step bolts for climbing are normally staggered at approximately 380mm intervals. Additional step bolts may be placed around the pole to provide a working level. Step bolts are installed in threaded inserts cast into the face of the pole.

Ladders are placed in clips bolted to the face of the pole using threaded inserts. In areas where maintenance is required, clips can be installed in multiple faces of the pole. A variety of ladder styles are available for use at maintenance locations. Manufacturing constraints may limit the location of inserts. The manufacturers should coordinate final placement of inserts with the user.

2.9.9. Grounding

The user should specify the grounding method. Grounding of concrete poles can be external or internal. For an external ground, threaded inserts can be embedded in the pole for clamping the ground to the pole's surface. Internal grounds can be embedded in the concrete or pulled through the centre void of the pole with pig tails or grounding pads as required. Many users in areas of high levels of lightning occurrence or high ground resistance bond all hardware to the grounding system.

2.9.10. Load Testing

The Sri Lanka Telecom should specify whether a full-scale structural test is required. A test may be performed to verify the design concept, meet legal obligations, and determine the level of reliability or to better understand structural, foundation or system behaviour under certain loading conditions. The height and type of structure and all loading cases to be tested should be clearly identified

Two types of testing are used to determine the flexural behaviour and flexural capacity of poles under static loading conditions, pole testing and structure testing. Pole testing is used to verify the design and quality of production of poles. Poles are generally tested in a horizontal position. The tests will check the cracking moment, ultimate moment and deflection of the poles. Structural testing is the simulation of the structure as it is to be used. The design loads are applied incrementally to check structural behaviour.

2.9.11. Foundations

The pole design can be affected by foundation rotation, which caused secondary moments due to additional deflection. Therefore, the type of foundation to be used is an important design consideration.

2.9.12. Pre-stressed concrete

Pre-stressing of concrete is defined as the application of compressive stress to concrete members. Those zones of the member ultimately required to carry tensile stresses under working load conditions are given an initial compressive stress before the application of working loads so that the tensile stresses developed by these working loads are balanced by induced compressive strength. Pre stress can be applied in two ways as Pre-tensioning or Post-tensioning.

2.9.13. Pre-tensioning

Pre-tensioning is the application of tensile force to high tensile steel tendons before casting the elements. When concrete has developed sufficient compressive strength, a compressive force is imparted to it by releasing the tendons so that the concrete member is in a permanent state of pre-stress.

2.9.14. Post-tensioning

Post tensioning is the application of a compressive force to the concrete elements at some time after casting. When the concrete has gained sufficient strength, pre-stresses are induced by tensioning the steel tendons passed through ducts cast into the concrete member and locking the stressed tendons with mechanical anchors. The tendons are normally grouted in place.

2.9.15. Advantages of Pre-stressing

The use of pre-stressed concrete offers distinct advantages over ordinary reinforced concrete. These advantages can be briefly listed as given below,

a. Pre-stressing minimizes the effect of cracks in concrete elements by holding the concrete in compression.

b. Pre-stressing allows reduced member sizes in comparison with ordinary reinforced concrete members.

c. Pre-stressed concrete is resilient and will recover from the effects of greater degree of overloads.

d. If the member is subjected to overloads, crack which may have developed will close up on removal of excessive loads.

e. Pre-stressing enables both entire structural elements and structure to be formed from a number of precast units. E.g. Segmental and Modular constructions.

f. Lighter elements permit the use of longer spanning members with a high strength to weight characteristics.

g. The ability to control deflections in pre-stressed members

h. Pre-stressing permits a more efficient usage of steel and enable the economic use of high tensile steel and strength concrete.

i. Pre-stressed concrete can provide significant cost advantages over structural steel sections or ordinary reinforced concrete.

2.10. Materials

2.10.1. Concrete

Design compressive strength

The minimum design 28 days concrete cylinder compressive strength f_c is 40MPa.

2.10.2. Stress-Strain curve

A typical stress-strain curve for concrete in compression is shown in Figure 4 below The elastic modulus of concrete can be defined as the secant modulus at $0.5f_c$.

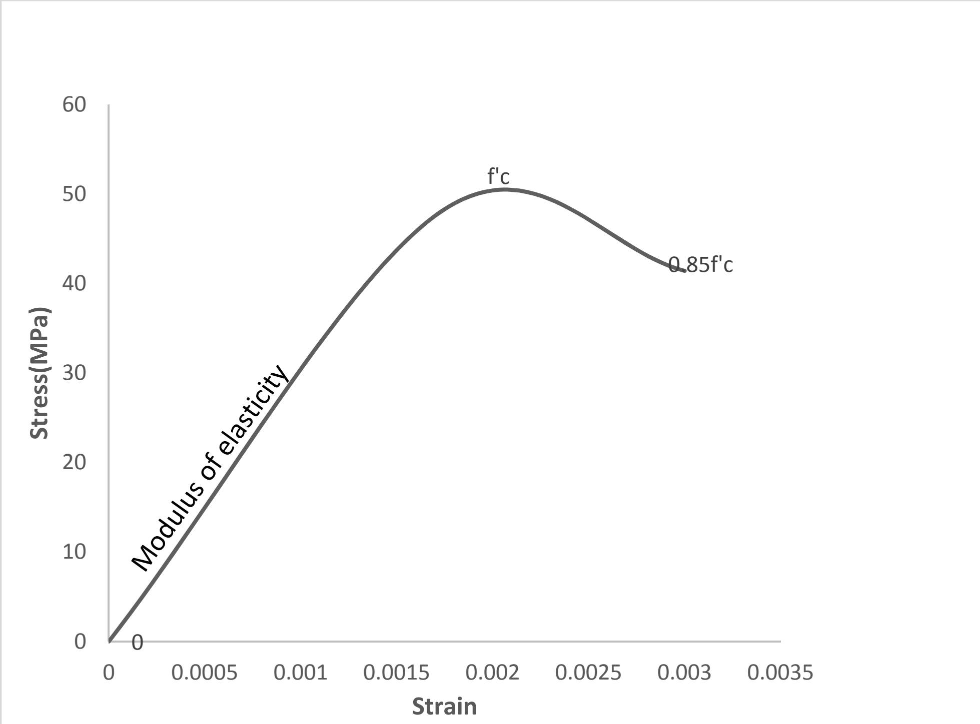


Figure 4: Stress, strain relationship for concrete in compression [4]

2.10.3. Pre-stressing steel

Pre-stressed concrete poles are typically reinforced with either uncoated, stress relieved steel wires (Appendix –C: ASTM A421) or uncoated low relaxation and stress relieved seven-wire strand (Appendix-C: ASTM A416). The steel is placed inside the form and stressed to the required tension. A typical arrangement of pole reinforcement is shown in below Figure 5.

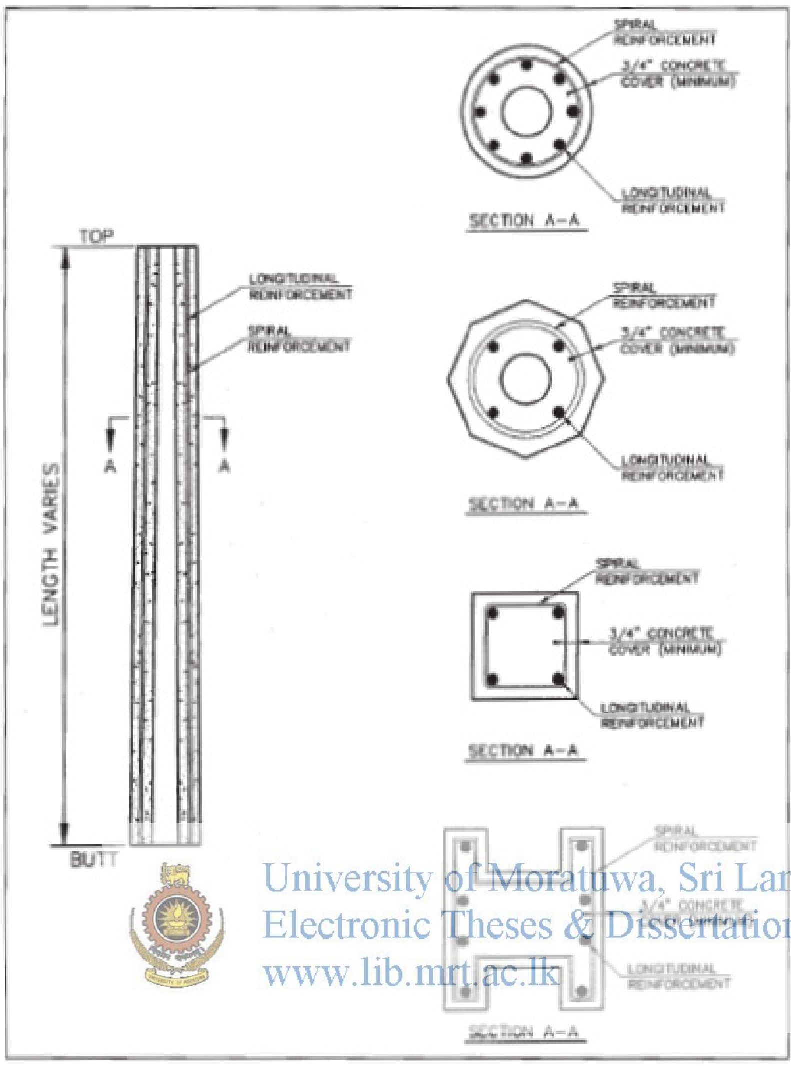


Figure 5: Typical details for reinforcement locations of precast poles [4]

Strands are also available with coatings such as epoxy (ASTM A882 [27], [28]) and galvanizing to provide protection in extremely corrosive environments. However the galvanizing process may result in the pre-stressing steel having lower breaking strength and a slightly lower modulus of elasticity. Increased development length due to epoxy coating should be considered.

2.10.4. Characteristics of pre-stressing steel

Mechanical properties of commonly used pre-stressing steel are given in APPENDIX – C: Characteristic of pre-stressing steel. A typical loads-elongation curve for a strand is also shown.

2.10.5. Allowable stresses

The permissible stresses in pre-stressing steel according to ACI 318-95 [22] are given in "APPENDIX – C: Characteristic of pre-stressing steel", where f_{pu} is the ultimate strength of the steel and f_{py} is the specified yield strength (ACI-1995 [22]). For wire and strands, the yield strength is defined as the stress at which a total extension of 1 percent is attained.

2.10.6. Spiral Reinforcements

Spiral reinforcement enclosing the strands helps to resist radial stresses caused by the wedging effect of the strands at release. It can also control or minimize cracks due to torsion, shear, shrinkage or temperature induced stresses.

The wedging effect from the release of the pre-tensioning forces cause tensile stresses at every cut-off strand's locations throughout the pole. Thus along the length of transfer (about 50 times the strand diameter), strands produce radial pressure against the surrounding concrete, which could develop longitudinal cracks unless properly contained by adequate spiral reinforcements.

The spiral reinforcements generally conform to ASTM A82 [27] and its sizes should be in the range of No. 5 to 11 gauge wire, depending on the pole used and size. The minimum area of spirals should be computed as 0.1 percent of the concrete wall area in a unit length increment. More spiral reinforcement is required at the tip and butt segments of the pole to resist the radial stresses that occur at transfer of pre-stresses.

The minimum clear spacing of spiral is four-third of the maximum size of coarse aggregate and should not be less than 25mm.The maximum centre to centre spacing should not exceed 100mm unless it is shown through tests that the performance of the pole is not impaired.

Poles subject to high shear forces, such as those with short embedment length, may require additional calculations of spiral requirements in the embedded section. Embedment length is also related to the minimum distance required to develop the ultimate strength of the pre-stressing wires.

2.10.7. Mild Steel Reinforcements

Mild steel reinforcing bars or dormant stands may be used in addition to the pre-stressed reinforcements to increase the ultimate moment capacity of the pole. The bar reinforcement is usually placed at the critical sections only and does not extend throughout the entire length of the pole. Because mild reinforcing steel will yield at strain much less than pre-stressing reinforcement, the designer should be aware that if the structure undergoes deflections large enough to yield the mild steel, it will no longer recover fully after release of loads.

2.11. Design Loads

2.11.1. General

This section discusses the types of loadings that might be used for the design of the Transmission and distribution structures and Communication structures are designed to withstand loading conditions that have been specified by the user and or government agencies responsible for ensuring the safe, reliable and economic operation of the system.

The loading conditions typically considered to determine the required strength of the transmission and distribution structures are the ASCE [25] guidelines for Electrical Transmission Lines Structural Loading (ASCE 1991 [25]), the National Electrical Safety Code (NESC) loads, state and local safety code loads, local meteorological loads such as combination of wind, and temperature conditions, longitudinal loads such as line terminations and broken conductor loads and construction and maintenance loads.

For certain load cases, structure deflection may govern the design. Load factors are applied to the various loading cases as required by code or as determined to be appropriate by the utility or the designer. The “overload capacity factors” of the NESC are one example of code load factors. Load factors for climatic, security and construction loads are suggested in the ASCE loading Guide. Other than load factors for code loads, there are no required standards for the various load cases and the load factors should be determined using engineering judgments or utility guidelines.

The NESC provides a set of minimum loads (Heavy, medium, light and extreme wind), with specified overload capacity factors, for the various grades and types of constructions. Most states have adopted the NESC, however, some states or local government have written and adopted their own safety codes to satisfy regional safety requirements.

Meteorological load factors are associated with local climatic conditions that may occur during the life of the line. Theses loads are generally set by the utility or selected by the designer. Typical loads consist of wind, ice and temperatures taken singly or in combinations. Generally, a high (extreme) wind load and combinations of wind and ice loads are both used for the design. The ASCE [25] guidelines for electrical Transmission lines structural loading may be referred to for the development of meteorological loads as well as other typical loads,

Longitudinal loads on a structure fall into three major categories:

1. Permanent loads due to line termination or change in ruling span
2. Temporary loads due to unbalanced ice and wind conditions and
3. Loads due to a broken or slack wires

Longitudinal loads resulting from a difference in wire tension from one side of the structure to the other side are relatively easy to determine for dead-end structures. Suspension structures are more difficult to analyse because of the displacement of the suspension insulators, which acts to balance wire tensions with the longitudinal loads experience by the structure. Longitudinal loads may be selected that approximates the loading conditions of the suspension structure. Unbalanced longitudinal loads may induce torsion in pole type structures and this torsion must be considered in the strength evaluation of the structure design. Broken wire loads may also be considered.

Construction and maintenance loads should be considered to ensure the safety assembly, erection, loading and operation of the system. Loads commonly considered as construction loads are wire stringing loads, snub-off loads and clipping-in loads.

Wire stringing loads are unbalanced wire tension when the running board or wire may become caught in the stringing block and get “hung-up,” Snub-off loads are the temporary dead-ending of the conductors and shield wire on one longitudinal side of the structure to the ground during stringing operations. Clipping-in loads are the loads for lifting the conductor from the block after the conductors were brought to the initial sag position.

Maintenance loads are worker and equipment loads associated with procedures such as changing insulator strings and hardware. The construction and maintenance loads usually occur with a nominal wind and a temperature likely to occur during this operation. Various combinations of loads are considered to predict structure deflections. These deflections are used to determine clearances, right-of-way width, raking of the pole and other special requirements.

It is recommended that loading conditions be expressed as load trees, using an orthogonal coordinate system as shown in "APPENDIX – D: Typical load tree for concrete pole". Conductors and shield wire loads should be shown at the conductors and shield attachment points. The weight of all attachments, such as hardware and insulators, should be included in these loads. Wind pressure to the structure itself should also be specified. All loads should be shown are factored loads. The load factors are usually equal to one for the load cases used to check cracking.

The Electronic Industries Associate Standard such as EIA/TIA/222-E [38] "Structural standards for Steel Antenna Towers and Antenna Supporting Structures" is recommended for the determination of loads, tolerances, foundations, anchors, guys and allowable twist and sway values. When using the working loads from these standards, a minimum load factor of 1.25 is recommended for concrete structures. This factor is the same as that used in the AASHTO [21]" Standard Specifications for Structural Supports for High-way Signs, Luminaries, and Traffic Signals.

2.11.2. Other loads

Structure located in areas subject to earthquakes should be analysed for the effect of seismic forces. By studying ASCE 7-95, the applicable building codes and other appropriate standards, the designer can determine the earthquake zone in which particular structures are located. For very important structures such as un-guyed (dead-end or heavy angle) structures or structures having stringent safety requirements, special analyses may be required.

Handling loads should also be considered. These loads are generated during transportation and erection of the structure. The lifting of the entire structure from the horizontal position is typically controlled by the handling conditions. This load is caused by the weight of the structure itself plus the weight of any items that may be attached to the structure.

To allow for shock loads that may occur while the structure is being lifted, an impact factor of 1.5 should be applied to the dead weight of the structure and attached accessories. Also the reduced strength f'_{ci} should be considered for stripping and in-plant handling. The manufacturers should indicate the locations of single or multiple-point picks, unless otherwise specified by the user.

2.12. Structural design

2.12.1. General

Pre-stressed concrete poles may be analysed using classical reinforced concrete theory. These poles exhibit both linear and nonlinear behaviours. Prior to exceeding the tensile strength of the concrete (below the cracking moment) the pole has a relatively constant modulus of elasticity and deflects in a linear manner. Above the cracking moment, the pole behaves mostly nonlinear because of the altered properties of the cracked section. During this state, greater deflection will occur than that of an un-cracked section for a similar increase in load. These deflections cause secondary moments in the structure due to the offset axial loading of the pole's centre of gravity couple with the weight of the conductors and insulators (So called P- Δ effect).

It is important that the effect of nonlinear be considered in the structural analysis not only because of the secondary moment induced but also because deflection can become significant when an extreme wind loading cause a structure in combination with the swing of the conductors, to approach the allowable clearance to the right of way edge (a condition known as "blow out").

2.12.2. Design Methods

The design of pre-stressed concrete poles is a relatively complex process that involves consideration of various loading conditions, time-dependent and nonlinear material behaviours, ultimate strength and serviceability.

Pre-stressed concrete poles should be designed primarily by the ultimate strength method. Service loading conditions such as first circumferential crack, reopening of cracks and deflection should be investigated with un-factored loads.

The cross sectional area of a pole is determined using an iterative design process. Starting with a specified pole height and a load tree, the designer assumes a trial pole cross section. Then for each section that is incrementally investigated, specified limit states must be satisfied. If not satisfied, the trial and error process is repeated until a solution is found.

The four distinct design conditions that may be considered in the design of pre-stressed concrete poles are:

1. Ultimate flexural strength
2. Cracking strength

3. Zero tension strength and

4. Deflection

2.12.3. Ultimate flexural strength

The ultimate flexural strength of a pole is the moment at which the pole will fail, usually by crushing of the concrete. The pole should be designed to have the ultimate strength at all sections of the pole exceed the required strength calculated from the appropriate factored loads applied to the structure, Factored loads are specified in codes (NESC-1993) guidelines (ASCE-1991 loading guide [25]) or other documents.

2.12.4. Cracking strength

The cracking strength of a pole is the moment at which the first circumferential crack will occur. Under this condition, the moment in the pole courses the tensile strength of the concrete to be exceeded on the tension face of the pole. The tensile strength is a function of the concrete modulus of rupture. These cracks will close up on release of loads. The pole should be designed to have the cracking strength exceed the moments calculated from the service loads. Typical service loads are NESC district loading without a load factor.

2.12.5. Zero Tension Strength

The zero tension strength is the moment at which a crack that was previously created by exceeding the cracking moment strength will open again. Under this condition, an applied moment will not cause any tensile stress in the concrete. This strength will always be less than the cracking moment strength. Structures that are subjected to permanent lateral loads, such as un-guy dead-end or angle structures, or structure controlled by deflection should be designed to have zero tension strength exceed the moments calculated from service loads or sustained loads. This would avoid having a crack remain open for the life of this structure type. Avoiding open cracks is important in extremely corrosive environments, such as placement in sea water or proximity of industrial containments, in order to protect the steel reinforcements.

2.12.6. Deflection

The maximum allowable deflection of a structure, as specified by the user, may control the design of the structure. The user should specify to the pole designer the loading conditions that are to be considered in determining the pole deflection. The pole stiffness (EI) should be sized so that the pole deflection calculated from the specified loading conditions does not exceed the maximum allowable deflections.

2.12.7. Pre-stress losses

The magnitude of the pre-stressing force in the pole is not constant but decrease with time. This decrease in the pre-stressing force is referred to as the pre-stress loss. Some pre-stress losses are instantaneous and some are time dependant. Instantaneous losses are due to elastic shortening, anchorage slipping and friction, in the case of post-tensioning. Time-dependent losses are mainly due to shrinkage and creep of concrete and steel relaxation. A detail analysis of losses is not necessary except for unusual situations where deflections could become critical. Lump sum estimates of losses are commonly used. Depending on the materials used, 15 to 25 percent for total losses are common design assumptions.

2.13. Principles and assumptions made

The ultimate moment capacity of a pole at any given cross section is a function of the strain in the pre-stressing steel and concrete. The factored design moment should not exceed the ultimate moment capacity. The following assumptions are made in computing the ultimate moment capacity of a pole:

- Plane sections remain plane
- The steel and concrete are adequately bonded
- The steel and concrete are considered in the elastic and plastic ranges
- The concrete compressive stress at failure is 0.85f'c
- The tensile strength of concrete is neglected in flexural computation
- The ultimate concrete strain is 0.003

While the first two assumptions become somewhat less valid after the section has cracked, the overall behaviour of the member can still be predicted adequately.

2.14. Determination of ultimate moment capacity

2.14.1. Equilibrium of a section

Based on the above assumptions and the provisions in the ACI 318 building code (1995) [22], the assumed rectangular compressive stress distribution in the concrete is used herein for simplification and is represented by the cylinder compressive strength f'_c the parameter β_1, and the quantity K_c, which locate the centroid of the stressed block as shown in Figure 6.

Equilibrium of the section requires equal forces in the pre-stressing steel and concrete. The equation of equilibrium is (without axial loads)

$C_c = T_s$

Where C_c is the concrete compression and T_s is the steel tension,

The compression in the concrete is then computed from:

$C_c = 0.85f'_c A_a$

Where A_a is the area of the concrete in compression as defined by a rectangular stress block of depth β_{1c}.the parameter β_1 is defined as 0.85 for a concrete strength of 27.5 MPa and less, and is reduced by 0.05 for each 7 MPa in excess of 27.5 MPa with a minimum value of 0.65. The computation of the compressive concrete area A_a for a round hollow pole and the location of the centroid of compression are derived in "APPENDIX – B: Area and centroid of ".

The steel tension is expressed as:

$T_s = \sum^{n}_{i=1} A_{psi}f_{sei}$

Where A_{psi} and f_{sei} are area and stress of the i^{th} strand respectively. Trial and error iteration of the location of the neutral axis c is used to solve for the depth of the stress block, such that equilibrium between tension and compression is satisfied.

2.14.2. Ultimate moment capacity equation

The ultimate moment capacity of a pole section is given as the sum of the moments of tensile and compression forces with respect to the neutral axis.

$\Phi M_n = \sum^{n}_{i=1} e_i A_{psi}f_{sei}+cC_c(1\text{-}K)$

Where e_1= d_1-c and Φ is the capacity reduction factor (0.90 for flexure).

Note that A_{psi}, f_{sei} and c are previously defined, K_c is the position of the centroid of the reduced compressive concrete area (pressure line), d_i is the distance of the i^{th} strand from the extreme compressive fibre, and e_i is the distance of the i^{th} strand to the neutral axis.

The quantity $e_iA_{psi}f_{sei}$ is positive when the i^{th} strand is located below the neutral axis (tension zone) and negative when it is located above axis (compressive zone). In the case of braced H-Frame and guyed structures, the formula for the ultimate moment capacity should incorporate the effect of the applied axial loads.

2.14.3. Cracking moment and zero tension moment

Cracking start when the tensile stress in the extreme fibre of the concrete will reach its modulus of rupture. The cracking moment can be computed by elastic theory to predict the behaviour of poles.

For a symmetrically reinforced pre-stressed concrete pole section, a uniform stress P/A_g acts on the gross sectional area A_g due to the effective pre-stress P. Because of the external moment M, the section area is subject to the extreme tensile stress My_t/I_g, where y_t is the distance from the centroid axis to the extreme tensile fibre and I_g is the gross moment of inertia of the section. The cracking moment may be calculated using the following relationship:

$M_{cr} = f_rI_g/y_t + PI_g/A_gy_t$

Where f_rI_g/y_t is the resisting moment due to the modulus of rupture of concrete (f_r) and PI_g/A_gy_t is the moment due to the direct compression of the pre-stress.

In ACI 318(1995), the modulus of rupture is given as $7.5\sqrt{f'_c}$ where f'_c is the concrete compressive strength (in psi).

Concrete stress area and assumed stress distribution in a pole section is given in below Figure 6.

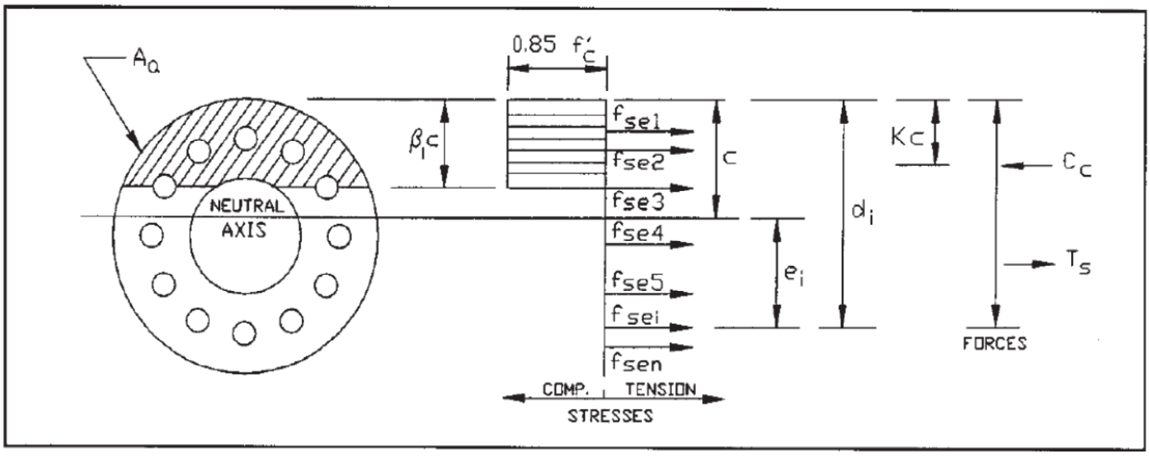


Figure 6: Stress distribution in a concrete pole section [4]

The zero tension moment M_0 may be calculated from the relationship:

$M_0 = PI_g/A_g y_t$

The stress distribution in a pole section at cracking and zero tension is shown in Figure 7 below.

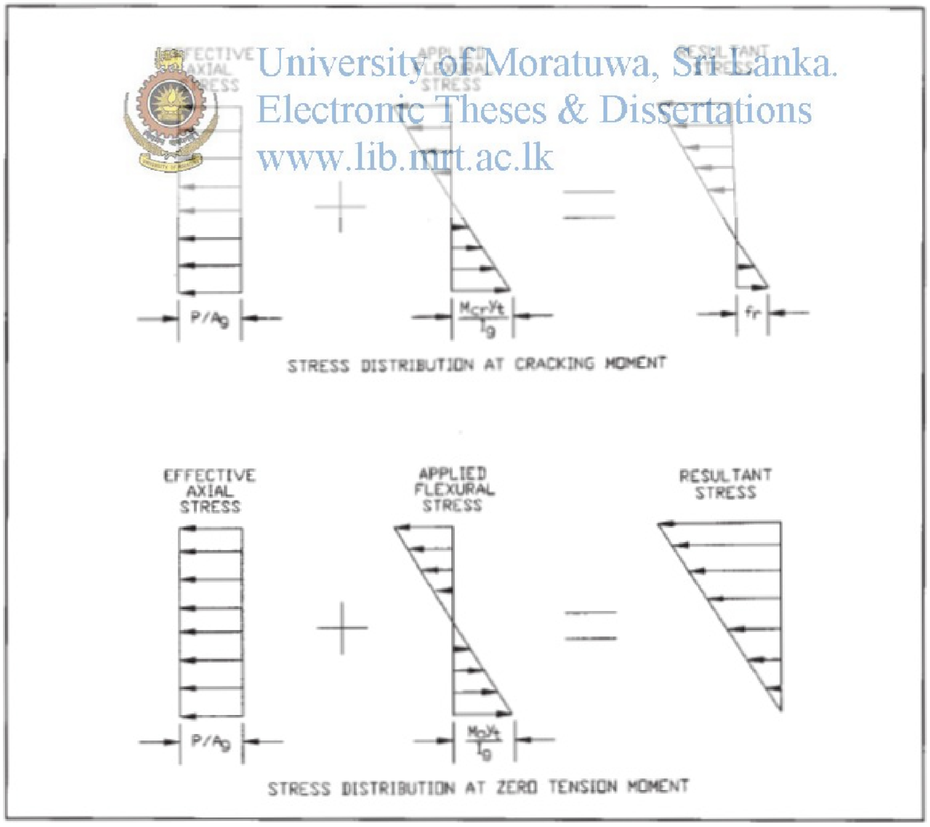


Figure 7: Stress distribution at zero tension and cracking moment [4]

2.14.4. Shear and Torsion

2.14.4.1. Shear

The design of concrete pole cross section subject to shear shall be based on:

$V_u \leq \Phi V_n$

Where V_u is the factored shear force at the section considered, Φ is taken as 0.85, and V_n is the nominal shear strength computed by:

$V_n = V_c + V_s$

Where V_c is the nominal shear strength provided by the concrete and V_s is the nominal shear strength provided by the shear reinforcement.

For square or rectangular pre-stressed concrete members with an effective pre-stress force not less than 40 percent of the tensile strength of the flexural reinforcement, V_c may be computed as:

$V_c = [0.6\sqrt{f'_c} + 700(V_u d/M_u)] b_w d$

But, V_c need not be less than $2\sqrt{f'_c} b_w d$ nor shall it be greater than $5\sqrt{f'_c} b_w d$

The quantity $V_u d/M_u$ shall not be greater than 1.0, where M_u is the factored moment occurring simultaneously with V_u at the section considered. The variable "d" shall be the distance from the extreme compression fibre to the centroid of the pre-stressing reinforcement and b_w shall be the width of the web.

For circular pre-stressed concrete members:

$V_c = \sqrt{(F^2_t + F_t f_{pc})}/(Q/2It)$

Where F_t=Tensile strength of concrete taken as $4\sqrt{f'_c}$

F_{pc}= Effective compressive stress in concrete due to pre-stress

Q = Moment of area above centroid

I = Moment of inertia of cross section

T = Wall thickness

For the shear force V_s contributed by the steel:

$V_s = A_v f_y d/s$

Where A_v is the area of the shear reinforcement within a distance s, f_y is the yield strength of the steel, and "d" is distance from the compression force to the centroid of the pre-stressing steel, or 0.8 times the outside diameter of the section, whichever is greater.

2.14.4.2. Torsion

The design of concrete pole cross sections subjected to torsion shall be based on:

$T_u \leq \Phi T_c$

Where T_u is the factored torsional force at the section considered is taken as 0.85, and T_c is the torsional resistance of the pre-stressed concrete member.

For square or rectangular cross sections [17]:

$T_c = 6\sqrt{f'_c}\sqrt{(1+10(f_{pc}/f'_c)}\sum\eta x^2 y$

Where η=0.35/(0.75+b/d)

And "x" is the shorter overall dimension of the rectangular part of the cross section, y is the longer overall dimension of the rectangular part of the cross section, and "b" is the width of the compression face of the member.

For circular cross sections:

$T_c = (J/r_0)\sqrt{(F^2_t+F_t f_{pc})}$

Where "J" is the polar moment of inertia and "r_0" is the outside radius of the section.

For members subject to simultaneous flexural shear and torsion, the following interaction equation may be used to represent the strength of the member:

$(V_u/0.85V_n)^2+(T_u/0.85T_c)^2 = 1.0$

2.14.5. Critical buckling loads

Although buckling of a concrete pole is unlikely under normal circumstances, in some cases of guyed structures it may be critical [18].

The best estimate of buckling loads for non-prismatic members can be obtained using numerical methods to solve the differential equations obtained from classical elastic stability theory or using a nonlinear finite element formulation, such numerical or finite element techniques are not practical without computers. These methods are described in advanced analysis text books. Today, finite element software programs are available at a modest cost.

For hand calculations, simplified techniques for determining buckling loads are available that gives conservative results for most cases. For poles with constant cross sections or uniform taper, the critical buckling loads can be approximated using the classical "Euler buckling" equation with appropriate effective buckling lengths. Using this approach, the critical buckling load may be determined by:

$P_{cr} = \pi^2 EI/(kL)^2$

The most difficult aspect of applying this equation is determining an appropriate value for EI. In place of a more precise calculation, EI for computing the buckling load may be taken as:

$EI=E_cI_g/2.5$

For poles of uniform cross section, I_g is the gross moment of inertia of the concrete. For uniformly tapered pole's I_g may be conservatively taken as the gross moment of inertia at a distance of one-third L from the smaller end of the un-braced length.

For cantilevered poles, the buckling length L to be evaluated should be from the centroid of the applied external loads to a point one-third of the setting depth below the ground line. The pole should be assumed fixed at the lower point and free at the upper end, giving a theoretical effective length factor k=2.0.

For poles guyed in both directions, the buckling length L to be evaluated should be the distance from the bottom guyed attachment to a point one-third the setting depth below the ground line. The pole should be considered fixed at the lower point and pinned at the upper end, giving a theoretical effective length factor k=0.7. In practice, however, a value of k=0.8 is preferable. Single poles guyed only in one direction should be treated as cantilevers for buckling purposes because they are free to deflect in the un-guyed plane. For H-frames, two modes of buckling should be checked. First, buckling in the plane of the cross brace should be checked using a length from the bottom cross brace attachment point to a point one-third of the setting depth below the ground line. The pole should be considered fixed at the lower point and pinned at the upper, giving a theoretical effective length factor k=0.7 (recommended value k=0.8).In addition, buckling in the plane perpendicular to the cross brace should be checked. This mode of buckling should be treated the same as the cantilever.

2.14.6. Deflections

Structure deflections should always be checked. For loads less than those causing the first crack, elastic deflections can be determined using the classical structural analysis methods. For loads in excess of cracking, an inelastic pole design method should be utilized. For the sake of appearance, excessive deflection under sustained loads should be avoided.

Additional structural deflection can occur due to concrete creep. This is the plastic deformation of the concrete due to application of loads over an extended time period.

This could result in increased deflections for poles used as strain poles, self-supporting dead-ends or guyed structure. For most pole applications, creep is not a major design consideration. However, it can be of significance for non-uniform stress distribution resulting from the combined effect of sustained load and pre-stress. Refer to ACI 318-95 [19] for design procedures involving creep.

2.14.7. Determination of elastic deflection

For loading conditions that do not exceed the cracking capacity of the pole, elastic methods may be used. This could include virtual work, the conjugate beam method, slope deflection, or a finite element computer analysis.

2.14.8. Determination of inelastic deflection

Up to the point of cracking, the deflection may be computed using elastic methods previously described. After cracking, the modulus of elasticity "E" become both stress and time dependent and the moment of inertia "I" becomes crack dependent. Because the product "EI" varies with stress, time and pole geometry, the process for computing inelastic deflection is too complicated for hand calculations and, therefore, lends itself to iterative computer computations.

The inelastic deflection can be approximated using reduced value of the elastic product "EI". These values may range from "E_cI_g" at a level of moment at cracking to "$E_cI_g/3$" as the member approaches ultimate strength.

2.15. Joints and connections

2.15.1. Connections

Connections between poles and attachments should be designed such that the allowable stresses of the connecting part and the concrete pole are not exceeded and excessive deformation or rotation is not induced. Hardware may be attached using through holes, bands or inserts, depending on the type and magnitude of loads.

Factors to be considered in connection design include the load transfer mechanism, load factors, ductility, durability, required tolerances, aesthetics and economics.

2.15.2. Bolted connections

Most hardware is bolted to concrete poles with galvanized through bolts. Good practice dictates that the bolts do not overload the concrete and that they be properly tightened. Bolts such as ANSI C135.1 or ASTM A307 are commonly used. Designing for use of lower strength bolts helps to ensure that the bolt loads do not exceed the allowable concrete bearing stresses. Because the low strength bolts are readily available, those which require replacement will be replaced with bolts of the correct strength. Sleeving of holes may be necessary as a mean of reducing concrete bearing stresses, particularly when higher strength bolts are used.

To spread the concentrated loads under the head of the bolt and under the nut, a square curved washer or other similar plate should be placed between the head or nut and the pole. For A307 bolts over 1 in.(25.4mm) in diameter or A325 bolts over ¾ in.(19.05mm) in diameter, use either two ¼ in.(6.4mm) thick washers or a single 3/8 in.(9.5mm) washer. Use of cast washers is not recommended. The turn of the nut method is applicable only to high strength bolts (A325 bolts).When A325 bolts are used, they should not be pre-tensioned to avoid overloading the hollow section.

For shear connections in which the bolt will bear against the side of the through hole, the maximum bolt bearing load will be determined by multiplying the diameter of the bolts times the effective wall thickness times the bearing strength of the concrete. In the absence of conforming tests, it is assumed that the bolt-to concrete interface carries all of the loads and none of it is carried through friction. The maximum effective wall thickness for calculating the bearing load is the least of 3 in/(76mm), four bolt diameters, or the actual wall thickness.

2.15.3. Climbing attachments

It is recommended that every individual part of the climbing system where a lineman could conceivably place his foot should be designed to withstand a static load of 750 lbs (3337.5N) without permanent deformation and a load of 500lbs (2225N) dropped 18 in.(457.2mm) without breaking, or the most recent occupational safety and health Administration(OSHA) recommendations for any other requirements.

2.15.4. Inserts

Insert should be made of materials that will not deteriorate in the environment in which they are placed. Care should be taken to ensure that the materials in the concrete, the insert and the bolts do not react unfavourably with each other.

The anchorage of the insert in the concrete should be such that they do not pull out under the design load. Preferably, they are designed and anchored in such a manner that the bolts will fail first.

Consult the appropriate ACI and PCI design guides for proper insert design loadings. It is necessary to ensure that bolts do not bottom out in the insert. This may require coordination between the user and or one or more suppliers. Inserts can be installed at various locations without reducing the strength of the pole; however in some cases the location and quantity of strands may be affected.

2.15.5. Splicing

Pre-stressed concrete poles can be spliced with several different types of connections to meet production, handling and transportation requirements, or to attain additional lengths. Four splices are considered here and details of these splices are shown in Figure 8 below:

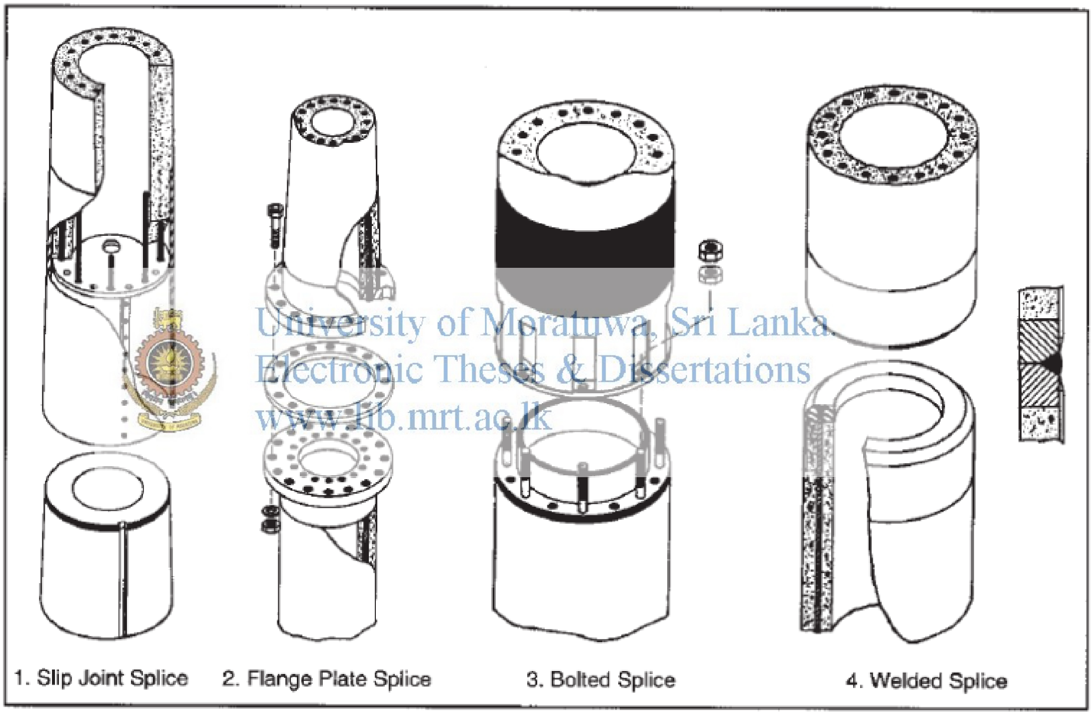


Figure 8: Details of some splicing [4]

2.15.6. Slip joint splice

This splice consists of a steel collar with the same taper as the pole. The upper pat is simply slid over the top of the lower part.

2.15.7. Flange plate splice

This splice consists of two flat steel plates that are held in place by the combination of strands and wedges. The two flanges are bolted together similar to a pipe connection.

2.15.8. Bolted splice

This splice consists of bolts embedded into the lower section of the pole, which is topped with a steel plate to which the pre-stressing strands are attached. The upper section has a steel plate to which the strands are attached and block-outs in the embedded side of the plate to act as voids for the bolts.

2.15.9. Welded splice

In this splice, steel plates are pre-stressed to the ends of the pole sections, which are then welded together in the field during erection.

2.15.10. Additional design considerations

2.15.10.1. Field drilling

When it is necessary to field drill holes in concrete poles, a rotary hammer drill or core drill should be used. Care should be taken not to cut the pre-stressing strands. If the strands must be cut, the pole should be checked for structural integrity.

2.15.10.2. Pre-stressing steel spacing

ACI 318(1995) [19] recommends a minimum clear distance between pre-stressing steel strands to be either four-thirds times the maximum aggregate size or three times the strand diameter, whichever is larger. In the event that this condition is not met at the pole tip, a closer spacing would be permitted provided the placement of concrete can be accomplished satisfactorily, adequate stress transfer can take place, and appropriate confining reinforcements is added.

2.16. Overview of the spin casting method

The traditional material for manufacturing distribution poles is wood, but alternatives are being sought as wood not only may become scarce, but is also susceptible to damage, and its preservation process is getting scrutiny from environmental groups. Spun-cast concrete poles have been used commercially for decorative street lighting, distribution poles, rail electrification, supports for high voltage transmission poles, communication towers, and wind turbine support structures.

Spun-cast poles are lower in cost and quicker to manufacture than conventional pre-cast poles. They are also more readily available and are not attacked by insects or animals. In addition, spun cast poles are elastic, corrosion resistant, maintenance free, cost effective and long lasting. Although the initial investment may be necessary, a life cycle analysis shows that spun cast poles are more economical in the long run. The pre-stressed spun-cast concrete poles are stronger and stiffer than conventional post implies that fewer poles can be used at a larger spacing, thus reducing the overall cost of a project.

2.16.1. Historical background

The first spun-cast reinforced concrete pole was produced in 1907 by the firm Otto and Schlosser in Meiszen, Germany, where the firm created a spinning machine and patented it in 1908. The first pre-stressed spun-cast poles were produced in the mid 50's in Europe by Fouad, 1992 [29]. The biggest problem with regular reinforced concrete poles is the corrosion of the steel reinforcement caused by water getting through cracks. These cracks may be caused by poor quality concrete, insufficient concrete cover, or excessive tensile stresses under loading. Once corrosion has started, failure may be inevitable with time. Pre-stressed poles are superior to regular reinforced concrete poles in this regard as they are usually fabricated using better concrete, and pre-stressing places the concrete in compression, preventing cracks from occurring. Additionally, the ability of pre-stressed concrete poles to withstand cyclic loading, which is a major consideration for the design of poles under wind loading, makes them superior to reinforced concrete poles [30].

2.16.2. Materials and Design

The minimum 28 day concrete compressive strength for spun-cast poles is typically 33 MPa, and strengths of 40 MPa to 80 MPa, based on statically cast cylinders, are also common. The reinforcing cage consists of regular rebar, pre-stressing strands and wire spirals. Regular rebar is used in sections where the flexural resistance based on the pre-stressed steel alone is inadequate. The pre-stressing reinforcement typically consists of stress-relieved wires or seven wire strands. In extremely corrosive environments galvanized or epoxy coated steel may be used, however, galvanizing may reduce the ultimate strength and modulus of elasticity of the steel. Also, if epoxy coating is used, a suitable development length should be chosen. The spirals are typically 4 to 6 mm in diameter and may have strength of up to 500 MPa. A larger amount is required at the ends of pre-stressed poles to resist the pre-stress transfer stresses. The spirals are used as shear and torsion reinforcement, and also to resist thermal stresses [31]. Poles are mainly flexural members but axial forces may need to be considered in some cases such as guyed poles, which are usually designed as compression members due to the high compressive forces produced. Shear and torsion rarely control the design, although they may be high in some cases. The major loads that need to be accounted for in the design process are meteorological loads such as wind, ice and temperature change. Other loads to be considered are caused by termination of a power line, unbalanced wind or ice loads, or loads due to broken wires in the transmission line. These last set of loads can cause torsion which should be accounted for as well. Construction and maintenance loads should also be considered. Poles may be analysed using classical reinforced concrete theory, however, second order effects at large deflections should be considered as they may have a large impact. When designing pre-stressed poles the conditions to be checked are ultimate flexural strength, cracking strength, zero tension strength, and deflection [31].

2.16.3. Manufacture

The basic equipment needed in the spin-casting process are the spinning machine and the steel forms. The spinning machines are heavy duty and have sets of spinning wheels at approximately 3 m intervals. Some machines are equipped with automatic form loaders and unloaders. The steel forms are usually made from two halves, but forms made from a single piece are also available. The steel forms are built for durability, and are balanced dynamically to minimize vibration during spinning. These forms are available in a large array of shapes and sizes with the most common being the circular tapered form [30].

The spin-casting process usually involves placing the steel spiral along the length of the oiled lower half of the form. Pre-stressing strands are then pulled through the spiral and a small force is applied to remove any slack. A pre-calculated amount of concrete is placed, and the form is sealed using the upper half before the strands are stressed against the form itself. The other alternative is to use “closed form filling” where the form is closed and the strands are stressed before the concrete is placed [30]. The form is then placed on the spinning machine, where it is spun for several minutes. Two speeds are usually used in the spinning process, at the lower speed the concrete is spread out along the form and pushed to the perimeter, creating the hollow section. At the high speed, the large centrifugal force produced pushes the excess water out of the concrete mix and consolidates the concrete, creating a dense and strong material. The reduced porosity protects the internal steel, and as a result concrete covers as small as 16 mm have been used with satisfactory results. After spinning, the form is steam cured until sufficient strength is achieved to transfer the pre-stressing force. After which, the pole is removed from the form and air cured before transportation [30].

The spun-cast pole combines the benefits of spinning, pre-stressing and using high strength concrete. The hollow core produced by the spinning reduces the weight and provides space for passing wires. Square and hexagonal sections can also be spun. Poles up to 36 m in length can be produced in one piece but longer poles are made by joining separate segments. Although statically cast poles, which are solid or have a hole formed by retractable mandrels or tubes, can be produced, they lack the enhanced concrete properties resulting from consolidation and could be heavier than spun cast poles. Fouad [29] reported that it is possible to apply 90% of the theoretical ultimate load to a spun-cast pre-stressed pole and not produce any noticeable damage in the pole when the load is released.

2.16.4. Erection

The poles are usually transported by truck, train or barge. Usually all hardware is attached while they are on the ground and then they are lifted and in-stalled by crane. However, the moments produced at lifting due to the self-weight of the pole may be significant and should be considered. The pole may be directly embedded in the ground, placed on a spread footing, or on a pile, depending on the soil conditions and anticipated loads. Segmental poles are used when transportation is difficult, the pole is too large, or when the erection area is congested. These segments have the advantage of requiring smaller cranes [31].

2.16.5. Properties of Spun-Cast Concrete

Most of the literature on spun-cast poles deals with design and installation with little information available on the material aspects. Dilger in 1996 [32] showed that the spinning process can cause the concrete to segregate producing a mortar layer on the inner surface of the pole. In some cases this inner layer may be one third of the thickness of the concrete. This segregation of the fine and coarse aggregates during spinning causes differential shrinkage with the mortar layer shrinking more than the outer layer which causes vertical cracking. The resulting cracks are deep and usually reach the longitudinal reinforcement. External factors such as temperature and service loads may cause the cracks to propagate to the outer surface causing the reinforcement to corrode, the concrete to spall and possibly the pole to fail. To overcome this problem the amount of fines and the water-cement ratio have to be reduced, but these solutions produce very stiff mixtures that are difficult to deal with, necessitating the use of super plasticizers. The spinning speed and duration also have an effect on segregation and compaction which needs to be studied. In general, the volume of fines should not be more than 30% of the total volume of aggregate, and at least 4% air content should be used for the concrete to be able to withstand freezing effects [32].

2.16.6. Field Performance of Spun Cast Poles

Fouad [29] undertook a study to evaluate the performance of spun cast round pre-stressed poles during hurricane Andrew. This study involved a site investigation and full scale testing. The field inspection's purpose was to assess the condition of the poles and to document the degree of damage caused by the extreme winds of the storm. Thirty three poles were inspected in two locations in Florida. The pole lengths ranged from 12 m to 23 m, with tip diameters between 152 mm and 330 mm, and outside pole slopes of either 18 mm/m or 15 mm/m. The visual inspection concentrated on 2.1 m above the ground line, where maximum stresses were developed, and that location is also easily accessible by a standing person. The number of cracks and the displacement at the foundation clearly showed the severity of the storm. However, the majority of the cracks were completely closed, and all the poles remained straight with no noticeable permanent deformation. This was attributed to the pre-stressing effect. It was concluded that the wind pressure probably loaded the poles close to their theoretical ultimate load capacity. Two - 27 m long poles were tested in flexure. It was shown that a pole deflected during one of the tests. The wall thickness ranged from 64 mm to 92 mm, and the compressive strength of the concrete was 66 MPa. The flexural reinforcement consisted of 13 mm pre-stressing strands placed symmetrically around the cross section. The pre-existing crack locations and sizes caused by the hurricane were measured before testing. The poles were tested horizontally with the maximum moment location approximately at ground level, and the loading point 0.6 m below the tip. The poles were loaded in increments of 1.8 kN, with each load held for at least 5 minutes, while crack and deflection measurements were taken. At predetermined loading points the poles were unloaded to see if the cracks would completely close. The poles were completely unloaded at their theoretical ultimate load, rotated 180 degrees, and the entire procedure repeated, this time until failure. It was found that the poles were unaffected by the extreme winds of the hurricane, and that the two load-deflection curves were very similar, having the same trend as the theoretical curve, but with higher stiffness and ultimate load. The increase in stiffness in the elastic range was attributed to concrete tension stiffening, and a conservative assumption of the modulus of elasticity. The ultimate loads were 8 % and 32 % higher than the theoretical ultimate load. Although the poles underwent large deflections (2 to 3 m), no permanent damage was caused before failure due to the effect of partial pre-stressing. Several pre-existing cracks in the maximum moment zone were monitored to determine the load at which they re-opened. The loads were 13 % and 6 % higher than the predicted load for the poles. It was concluded that the behaviour of these poles was not affected by previous loading, and the cracks always reclosed because of the pre-stress force.

Formulas described in the preceding chapter are used to derive optimum pre-stressed concrete section for different height categories. Proposed specifications and structural calculations are shown in the next chapter of the report.

CHAPTER 3

3.1. Methodology

Theoretical studies were performed prior to testing to predict the behavior of the spun concrete poles reinforced with pre-stressing and normal reinforcement bars and were compared with the experimental results. Design equations available in the literature and design guidelines for concrete poles reinforced with pre-stressing reinforcements were evaluated and modified to estimate the flexural capacity and crack widths.

Currently available telecommunication posts are made as tapered square section as described in section 2.4 made with normal reinforcements and 1:2:4 (20mm) in-situ concreting poured to the formwork. But our finding evidence that pre-stressed, tapered, hollow concrete poles are more economical, less weight and the structural performances are more superior in comparisons with concrete post used in practice now. Table 8 shown below are the summarized description of proposed new pre-stressed concrete poles, designed lateral loads as per the service requirements, bending capacity, nominal weight against their standard length.

Table 8: Proposed specifications for pre-cast, pre-stressed concrete poles

Pole Marking	Length (L)	Top Diameter (d_o)	Bottom Diameter (D_o)	Height of Bearing Point (l_1)	Design Service Load Point (P)	Design Bending Moment (P x l_2)	Nominal Weight
	m	mm	mm	m	kN	kNm	kg
5.6-100-1.04	5.6	100	150	0.950	1.04	4.32	140
6.7-100-1.44	6.7	100	170	1.145	1.44	7.28	184
7.5-100-1.44	7.5	100	175	1.300	1.44	8.20	210
8.0-120-1.44	8.0	120	200	1.360	1.44	8.84	305
9.0-140-1.44	9.0	140	260	1.500	1.44	10.08	502

Specifications for Poles marking
5.6-100-1.04

Design Service Load (kN)

Top Diameter (mm)

Length (m)

Calculations shown below further verified the structural performances of poles under service and ultimate loading conditions, pre-stressed steel reinforcement requirements, maximum crack width etc,.

SAP 2000 [11] finite element analysis programme is used to check the deflection and dynamic behaviours of those poles under influence of specified loadings. Program output analysis results are shown in "APPENDIX – F: SAP2000 Finite elements analytical results."

Structural drawings of proposed pre-stressed concrete poles and concrete post used in current practice are shown in "APPENDIX – A: Structural drawings".

3.2.Structural calculations for the 9m high pole

9m high Pre-stressed Pole		
Analysis of critical section of the pole (at ground support position)		
r_1 (Radios of outer circle) =	130	mm
r_2 (Radios of inner circle) =	90	mm
Thickness of pole =	40.00	mm
Gross concrete area at base A_g	27,646.02	mm^2
Assume C= Neutral axis depth by trial and error	90.00	mm
β_1= Parameter to calculate rectangular concrete compressive stress block	0.69	
$\beta_1 C$ =	62.10	mm
f'_c = Concrete compressive strength	50.00	N/mm^2
$f_r=0.62\sqrt{f'_c}$ Modulus of rupture of concrete	4.38	N/mm^2
y_t = Distance from centroid axis to extreme tensile fibre	170.00	mm
I_g = Gross moment of inertia of the section	1.73E-04	m^4
Finding area of Annulus		
Φ_1 =	2.52	radians
Φ_2 =	2.22	radians
Area A_1 =	16,312.97	mm^2
Area A_2 =	5,768.06	mm^2
Therefore Annulus area $A_a = A_1 - A_2$	**10,544.91**	mm^2
Finding centroid of annulus A_a		
Assume distance to the centroid from the centre x=	76.45	mm
Φ_3 =	1.88	radians
Φ_4 =	1.11	radians

A'_a =	**7,011.66**	mm^2
Find the "x" so that $A_a \approx 2A'_a$		
Cross sectional area of Φ7mm strand A_{psi} =	38.48	mm^2
Number of strands	4	No's
Modulus of elasticity of pre-stressing steel E_s =	205.00	kN/mm^2
Modulus of elasticity of normal steel E_y =	200.00	kN/mm^2
Modulus of elasticity of concrete E_c =	31.75	kN/mm^2
f_{py} = Specified yield stress of pre-stressing steel	1620.00	N/mm^2
Total pre-stressing force per strand = F_{py}	62.34	kN
Minimum breaking load F_{pu} =	62.50	kN
Therefore permissible pre-stressing force per strand = lesser of $0.80f_{pu}$ and $0.94f_{py}$ (assume 10% loss due to relaxation)=	50.00	kN
Assessment of transmission length: $l_t = K_t\Phi/\sqrt{f_{ci}}$	593.97	mm
where K_t is a coefficient for tendons =	600.00	
Average diameter of the section	200.00	mm
Average cross sectional area of concrete	20106.19	mm^2
Exposed perimeter of the section	628.32	mm
Effective section thickness of concrete (under immersed conditions)	600.00	mm
Elastic deformation of concrete at the age of stress transfer	2.82	mm
Creep strain ε_{cc}=stress x ø/E_t	4.70E-04	
Therefore creep deformation of concrete	4.23	mm
where ø = creep coefficient	1.50	
E_t = Modulus of elasticity of concrete at the age of t = E_c		
Design as class 3 member with 0.1mm crack width at ultimate loading		
For grade 50 concrete for limiting the crack width to 0.1mm		
Design flexural stress for class 3 member f_r =	4.80	N/mm^2

Design compressive stress at extreme fibre should not exceed $0.5f_{ci}$

where f_{ci} is the concrete strength at transfer

$0.5\ f_{ci}$ =	25.00	N/mm^2
Concrete stresses due to effective pre-stresses = < $\lvert 0.5f_{ci} \rvert$	-7.23	N/mm^2
Concrete stresses due to bending: assume compression " - "		
Concrete compression at compression zone	-2.54	N/mm^2
Therefore maximum concrete compression = < $\lvert 0.5f_{ci} \rvert$	-9.78	N/mm^2
Maximum concrete tension for class 3 member f_r =	4.80	N/mm^2
Strain at extreme fibre at tension zone =	1.51E-04	

Calculation of steel stresses and moment about neutral axis :

Area of pre-stressing strand =	38.48	mm^2
Strands stresses due to effective pre-stresses =	1299.22	N/mm^2
Strands stresses-1 due to bending at tension zone=	7.29	N/mm^2
Moment about neutral axis	4.02	kNm
Strands stresses-2 due to bending at tension zone=	27.34	N/mm^2
Moment about neutral axis	12.25	kNm
Normal steel stresses due to bending =	20.95	N/mm^2
Area of normal steel =	56.55	mm^2
Moment about neutral axis	0.14	kNm
Total moment about neutral axis due to steel tensions	**16.41**	kNm
Concrete compression $C_c = 0.85f_c'A_a$	448.16	kN
Centroid distance	36.45	mm
Moment about neutral axis due to concrete compression	**16.34**	kNm

Trial and error to find the value of "C" so that above two figures are almost equal

Zero tension moment $M_0 = P\, I_g/(A_g\, y_t)$	**7.35**	kNm
Cracking moment $M_{cr} = P\, I_g/(A_g\, y_t)+f_r\, I_g/y_t$	**12.23**	kNm
Service moment of the pole =< M_{cr}	**12.24**	kNm
Therefore pole design is satisfactory for given service requirements		

Refer the “APPENDIX – E: Structural calculations for the 8m, 7.5m, 6.7m and 5.6m poles” for calculations of other poles.

3.3.Extract from the SAP2000 finite elements analysis results [11]

Table 9: SAP2000 analysis results.

Pole height(m)	Maximum deflection at top of the pole due to service load (mm)	Period at first mode of vibration (seconds)	Base reactions due to applied service loads X, Y and Z in kN
9	98	0.48	1.44, 0, 4.45
8	173	0.52	1.44, 0, 2.74
7.5	244	0.53	1.44, 0, 1.96
6.7	130	0.44	1.44, 0, 1.86
5.6	115	0.38	1.040, 1.39

Modelling geometry and analysis results for 9m high pole are shown in Appendix F

Section under considered are 9m high tapered hollow circular section as defined in Table 8. Pole is modelled as 3D free cantilevered vertical pole with fixed support at bottom. Service load is applied as horizontal force 600mm below the tip of the pole as recommended in the code. 4 pre-stressing tendons with 7mm diameter and 4 normal reinforcement steel bars with diameter 6mm are also incorporated to the model geometry. Analysis results such as vibrating modes, period of vibration, deflection patterns, base reactions etc., are observed.

Manufacturing process and main advantages of using spun cast pre-stressed concrete poles are described in the next section.

CHAPTER 4

4.1.Pre-stressed spun cast concrete poles

Pre-stressed concrete poles (PC Poles) have proven to be of higher quality and greater durability than normal reinforced concrete poles. PC poles are better alternative to existing reinforced concrete post, steel and wooden poles used widely in electrical and telecommunication sectors, which tend to deteriorate due to metal fatigues and weather conditions. This is especially beneficial in coastal areas where rust is a problematic concern.

4.2.The main advantages of proposed PC poles

Economical

PC poles have higher strength-to-weight ratio compared to ordinary concrete poles. They are more durable compared to the ordinary concrete, wooden and steel poles and hence, more economical as frequent replacement due to deterioration is not required.

Light Weight

PC poles are hollow, light and easy to transport and install

Superior Quality

The poles, which are manufactured under stringent quality controlling at factory to comply with the Industrial Standards, which require strict adherence to correct quality control, modern and up to date technology.

Safety and Maintenance

PC poles are maintenance free, as they do not require painting nor periodic inspection offers extra safety in lightning prone areas compared to steel poles.

Attractive Finishes

PC poles are aesthetic as they are modern, slim and tapered with smooth texture. Various colour schemes can be made available to blend in with the surrounding landscape and at customer's request.

Long-Lasting

The high compaction forces generated during the centrifugal process result in a super dense concrete, which is practically impermeable. This makes PC poles more durable compared to other concrete poles, wooden poles and steel poles. PC poles are not subject to decay, termite attack and damage by fire or underground decay.

4.3.Manufacturing process flow chart for the pre-stressed spun cast concrete poles

Figure 9 shown below illustrate the general flow chart for the large scale manufacturing process of spun cast pre-stressed concrete poles.

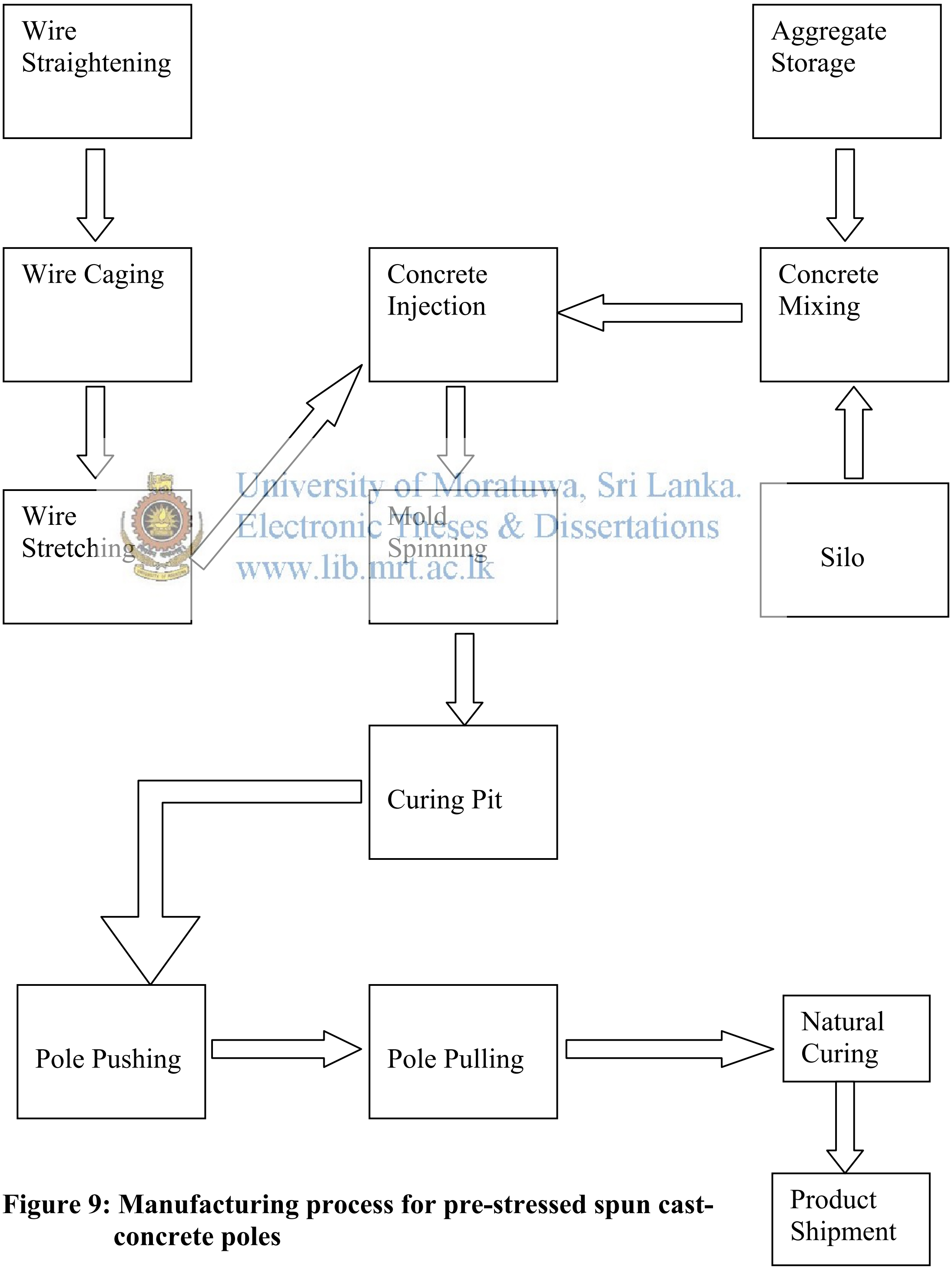


Figure 9: Manufacturing process for pre-stressed spun cast-concrete poles

4.4.Geometrical comparisons

Table 10 shows the weight reduction of proposed poles in comparison with current concrete posts used in practice.

Table 10: Poles geometrical comparisons (Current and proposed)

Pole category		Type		Weight (kg)			
Length (m)	Service Load (kN)	Current	Proposed	Current	Proposed	Difference	%
5.6	1.04	RC-Rectangle	PSC-Circular Hollow	177	140	37	21%
6.7	1.44	RC-Rectangle	PSC-Circular Hollow	364	184	180	49%
7.5	1.44	RC-Rectangle	PSC-Circular Hollow	408	210	198	48%
8	1.44	RC-Rectangle	PSC-Circular Hollow	450	305	145	32%
9	1.44	RC-Rectangle	PSC-Circular Hollow	576	502	74	13%

RC-Reinforced Concrete
PSC-Pre-stressed Spun Concrete
Therefore it is proposed to replace existing RC rectangular poles with PSC Poles

4.5.Loads specifications

Table 11 shows service and ultimate loading requirements of proposed poles against its heights.

Table 11: Loads specifications

Factor of Safety – 2.5

Pole Category						
Length (m)	Service Load (kN)	Ultimate Load (kN)	Embedment Length (m)	Lever Arm (m)	Required Service Moment (kNm)	Required Design for ULT Moment (kNm)
5.6	1.04	2.6	0.95	4.15	4.32	10.79
6.7	1.44	3.6	1.14	5.06	7.28	18.21
7.8	1.44	3.6	1.30	6.00	8.64	21.60
8	1.44	3.6	1.36	6.14	8.84	22.10
9	1.44	3.6	1.50	7.00	10.08	25.20

Structural details and specifications for proposed designs are shown in Appendix – A. Few Samples were cast as per the proposed designs and load testing were performed to observe the deflection and failure behaviours as described in the following chapter.

CHAPTER 5

5.1. Results and observations

The main objective of this experimental program was to evaluate the flexural behavior of spun concrete poles reinforced with per-stressing and normal reinforcements. The tip deflection was recorded by two means. The first was a scale attached to the test frame near the tip of the pole, and the second was a tape connected to the pole. After the first cracking of the specimen, the crack width was measured at each load increment using crack comparators. The strain gauges and the load cell readings were recorded via a data acquisition system.

Three pole samples are cast and tested by the third party, Senaka Zenn (Pvt) Ltd for verification purposes. Later results are sent to the Department of Civil Engineering, University of Moratuwa for their observation and verifications.

Load testing results are given under following paragraphs.

5.2.Load testing

Report on load testing of 7.5m length pre-stressed concrete spun cast circular type poles [14].

5.3.General

Three numbers of pre-stressed concrete spun cast circular type poles of 7.5m length were tested for their bending strength at Senaka Builder's (Pvt) Ltd., testing facility at Panagoda, Homagama, Sri Lanka on 10th October 2014. Some of the guide lines provided by Ceylon Electricity Board (CEB) specification, Appendix V-044-2-1997, Volume 11.2 have been followed during the testing of theses poles. Information on tested poles is given in Table 12.

Table 12: Specifications of tested poles

Pole Identification	Length of pole in (m)	Top diameter (mm)	Bottom diameter (mm)	Average wall thickness (mm)	Date of casting *	Grade of concrete *	Details of Pre-stressing and other steel**
Sample 000001	7.5	102	178	32	06/05/2014	60	See Figure 16 for details
Sample 000014	7.5	101	176	32	06/05/2014	60	
Sample 000016	7.5	102	177	30	06/05/2014	60	

* Information provided by the client

** Number of bars and diameter of steel members indicated in the drawing were verified by breaking the test sample after the bending test.

The supported length of each pole during the testing was 1.25m. This length is equivalent to the 1/6th of the total length of the pole. The test load was applied at a point 0.6m away from the top of the pole.

Poles were designed to withstand following loads as shown in Table 13.

Table 13: Design loads of tested poles

Load	Value (kgf)	Factor of safety against working load
Working Load (Service Load)	100	1.0
Proof Load (Load at which 0.25mm crack shall occur under bending)	200	2.0
Ultimate/Breaking Load (Maximum load that pole shall be able to withstand)	260	2.6

5.4. Methodology adopted for testing

Based on the guidelines provided in CEB specification Appendix V-044-2-1997, Clause 11.2, following methodology was adopted for the testing of each pole.

1. In first stage, pole was loaded up to the 40% of proof load (see Table 13). At each load increment of 10% of proof load, pole deflection was measured. Then load was released and residual deflection was measured.

2. In second stage, pole was loaded up to the 60% of proof load. At each load increment of 10% of proof load, pole deflection was measured. Then, load was released and residual deflection was measured.

3. In third stage, pole was loaded up to the breaking load. At each load increment of 10% of proof load, pole deflection was measured. In addition, load to produce 1^{st} hair line crack and first 0.25mm crack were measured. Finally breaking (ultimate) load was recorded.

Each loading step, applied load was measured using a tensometer which was fixed to the loading rope. Top deflection of the pole was measured using a pointer fixed to the pole which was directed to the steel ruler lying perpendicular to the pole. Tensometer was later calibrated using a proving ring of capacity 3 tons (Ring POT-15470) which bears the dial gauge (26136).

Data recorded sheets of the tested poles are shown in Table 14,Table 16 and Table 18. Respective cracking patterns at failure are shown in Figure 10, Figure 12 and Figure 14. Test results of the three Samples are presented in section 5.5

5.5.Test results

Table 14: Bending test data recording sheets for Sample 000001

Basic data	
Date of testing	10/10/2014
Location of testing	Senaka Builders (Pvt) Ltd, testing facility at Panagoda
Pole identification	000001
Type of Pole	Spun cast circular type pole
Length of pole	7.5m
Top and Bottom diameters	102mm, 178mm, W/T=32mm
Reinforcement details	See Figure 16
Grade of concrete used	60 (Indicated by the client)
Specified working load	100kgf
Specified proof load	200kgf
Specified breaking load	260kgf
Date of cast	06/05/2014 (Indicated by the client)

Figure 10: Crack pattern at ultimate load of Sample 000001

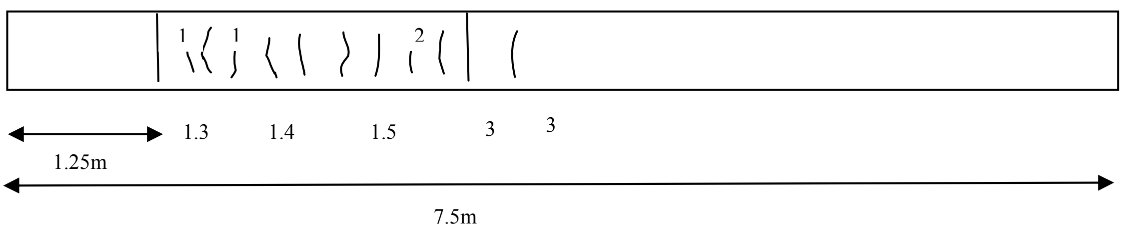


Table 15: Load-displacement relationship for Sample 000001

C. Load-displacement relationship				
Loading step	Load applied (kg)	% of ultimate load	Deflection (mm)	Remarks
1	0	0	0	
2	20	10	11	
3	40	20	29	
4	60	30	43	
5	80	40	59	
6	0	0	02	No cracks at this load
7	20	10	13	
8	40	20	29	
9	60	30	43	
10	80	40	59	
11	100	50	75	
12	120	60	114	No cracks at this load
13	0	0	02	
14	20	10	16	
15	40	20	32	
16	60	30	47	
17	80	40	68	
18	100	50	91	
19	120	60	115	
20	140	70	167	1st crack appeared
21	160	80	241	
22	180	90	332	
23	200	100	440	
24	210	105	491	0.25mm crack occurred
25	220	110	551	
26	240	120	683	
27	260	130	834	
28	280			**282kg failure load**

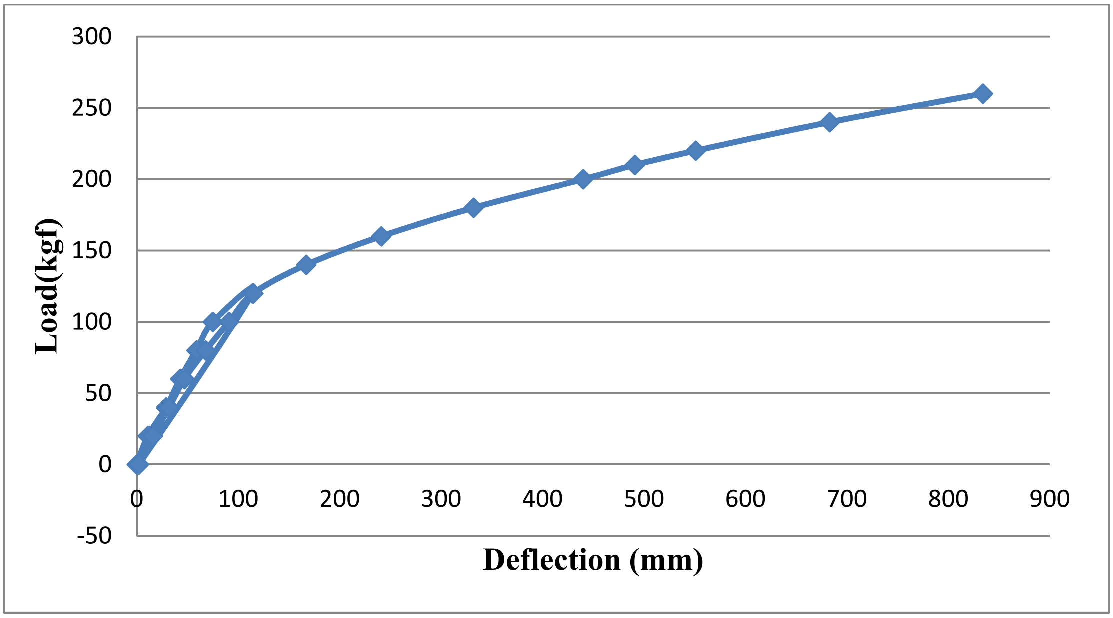


Figure 11: Load deflection relationship for Sample 000001

Table 16: Bending test data recording sheets for Sample 000014

A. Basic data	
Date of testing	10/10/2014
Location of testing	Senaka Builders (Pvt) Ltd, testing facility at Panagoda
Pole identification	000014
Type of Pole	Spun cast circular type pole
Length of pole	7.5m
Top and Bottom diameters	101mm, 176mm, W/T=32mm
Reinforcement details	See Figure 16
Grade of concrete used	60 (Indicated by the client)
Specified working load	100kgf
Specified proof load	200kgf
Specified breaking load	260kgf
Date of cast	06/05/2014 (Indicated by the client)

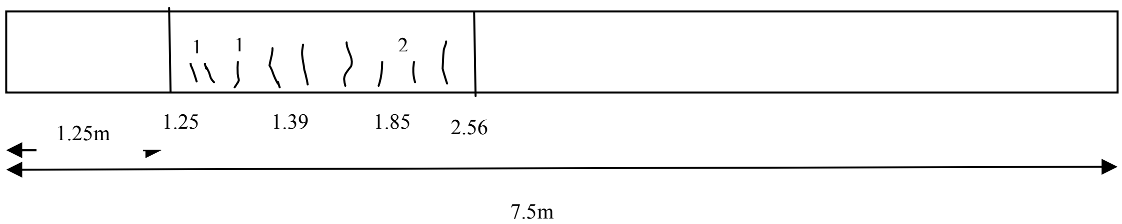


Figure 12: Crack Pattern at ultimate load of Sample 000014

Table 17: Load-displacement relationship for Sample 000014

C. Load-displacement relationship				
Loading step	Load applied (kg)	% of ultimate load	Deflection (mm)	Remarks
1	0	0	0	
2	20	10	17	
3	40	20	32	
4	60	30	52	
5	80	40	71	
6	0	0	03	No cracks at this load
7	20	10	21	
8	40	20	37	
9	60	30	52	
10	80	40	71	
11	100	50	92	
12	120	60	121	
13	0	0	8	
14	20	10	23	No cracks at this load
15	40	20	39	
16	60	30	56	
17	80	40	76	
18	100	50	98	
19	120	60	121	
20	140	70	173	Hairline cracks appeared
21	160	80	242	
22	180	90	328	
23	200	100	421	
24	210	105	476	0.25mm crack occurred
25	220	110	532	
26	240	120	671	
27	260	130	823	
28	268			**268kg failure load**

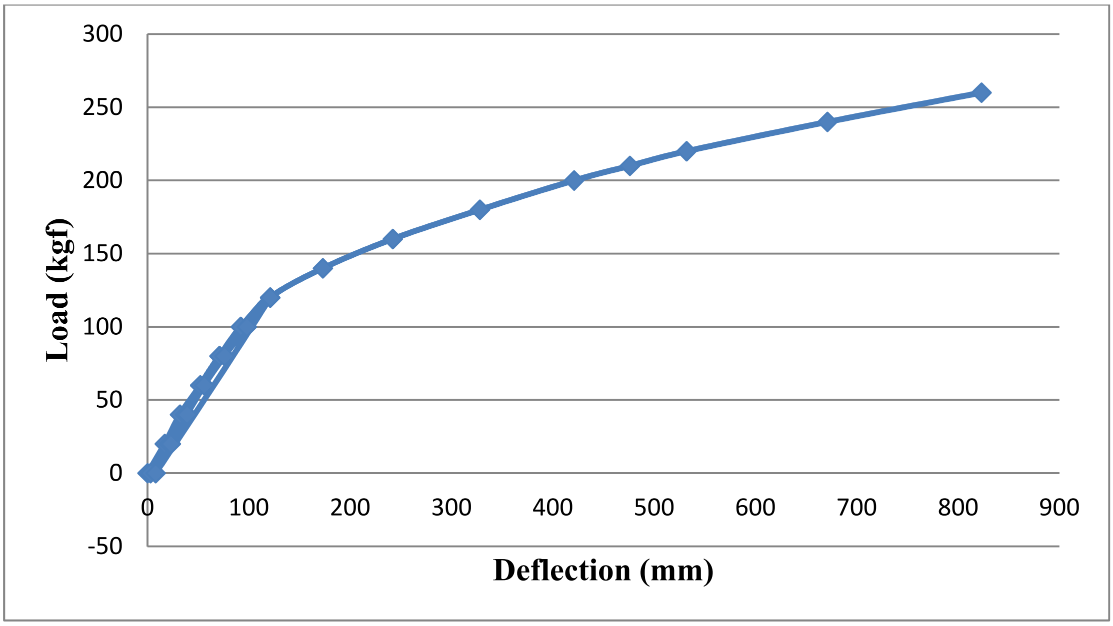


Figure 13: Load deflection relationship for Sample 000014

Table 18: Bending test data recording sheets for Sample 000016

A. Basic data	
Date of testing	10/10/2014
Location of testing	Senaka Builders (Pvt) Ltd, testing facility at Panagoda
Pole identification	000016
Type of Pole	Spun cast circular type pole
Length of pole	7.5m
Top and Bottom diameters	102mm, 177mm, W/T=30mm
Reinforcement details	See Figure 16
Grade of concrete used	60 (Indicated by the client)
Specified working load	100kgf
Specified proof load	200kgf
Specified breaking load	260kgf
Date of cast	06/05/2014 (Indicated by the client)

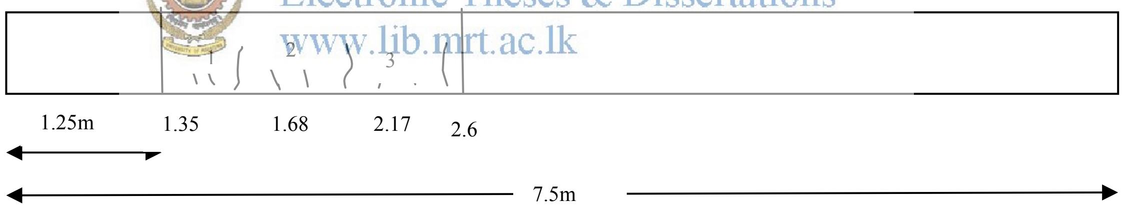


Figure 14: Crack patterns at ultimate load of Sample 000016

Table 19: Load-displacement relationship for Sample 000016

C. Load-displacement relationship				
Loading step	Load applied (kg)	% of ultimate load	Deflection (mm)	Remarks
1	0	0	0	
2	20	10	16	
3	40	20	32	
4	60	30	54	
5	80	40	85	
6	0	0	04	No cracks at this load
7	20	10	22	
8	40	20	39	
9	60	30	59	
10	80	40	86	
11	100	50	132	1st hairline cracks (1.5m)
12	120	60	192	
13	0	0	11	Crack closed at unloading
14	20	10	26	
15	40	20	43	
16	60	30	67	
17	80	40	102	
18	100	50	144	
19	120	60	192	
20	140	70	262	
21	160	80	351	
22	180	90	456	0.25mm crack occurred
23	200	100	566	
24	220	110	705	
25	240	120	843	
26	260	130		
27	268			**268kg failure load**

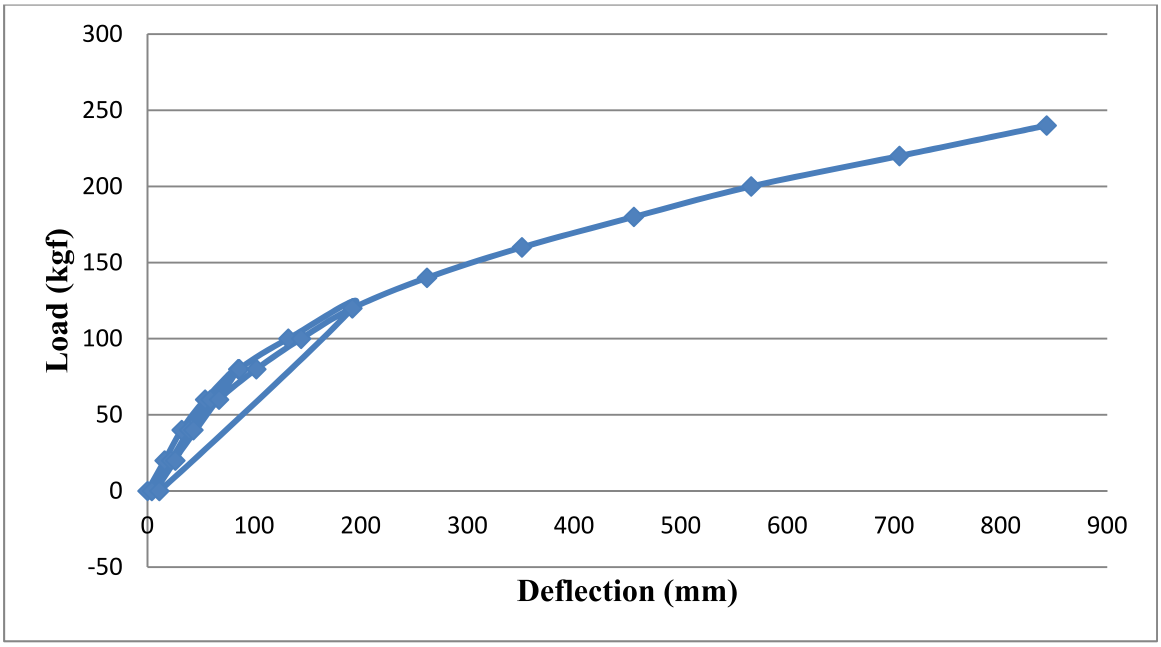


Figure 15: Load deflection relationship for Sample 000016

Table 20: Behaviour of poles under 40% of proof loads

Behaviour of poles under 40% of proof load – 80kgf – First stage of loading				
Sample identification	Deflection (mm)	Residual deflection after releasing 40% of proof load	Crack observation	Remarks
Sample 000001	59	02	No cracks observed at this load	Residual deflection is less than 10% of deflection at 80kgf
Sample 000014	71	03	No cracks observed at this load	Residual deflection is less than 10% of deflection at 80kgf
Sample 000016	85	04	No cracks observed at this load	Residual deflection is less than 10% of deflection at 80kgf

Table 21: Behaviour of poles under 60% of proof loads

Behaviour of poles under 60% of proof load – 120kgf – Second stage of loading				
Sample identification	Deflection (mm)	Residual deflection after releasing 60% of proof load	Crack observation	Remarks
Sample 000001	114	02	No cracks observed at this load	Residual deflection is less than 10% of deflection at 80kgf
Sample 000014	121	08	No cracks observed at this load	Residual deflection is less than 10% of deflection at 80kgf
Sample 000016	192	11	Hairline crack has occurred at a load of 100kgf.Cracks have completely closed once the load was released.	Residual deflection is less than 10% of deflection at 120kgf

Table 22: Final bending test results of three samples

Final bending test results					
Sample identification	Test load to produce first hairline crack (kgf)	Proof load (Load to produce 0.25mm crack)		Ultimate/Breaking load	
		Test load value (kgf)	Factor of safety against working load	Test load value (kgf)	Factor of safety against working load
Sample 000001	140	210	2.1	282	2.82
Sample 000014	140	210	2.1	268	2.68
Sample 000016	100	180	1.8	268	2.68
Average	126	200	2.0	272	**2.72**

Structural details of the tested poles and specifications are shown in the Figure 16 and Table 23 respectively as verified at the testing laboratory.

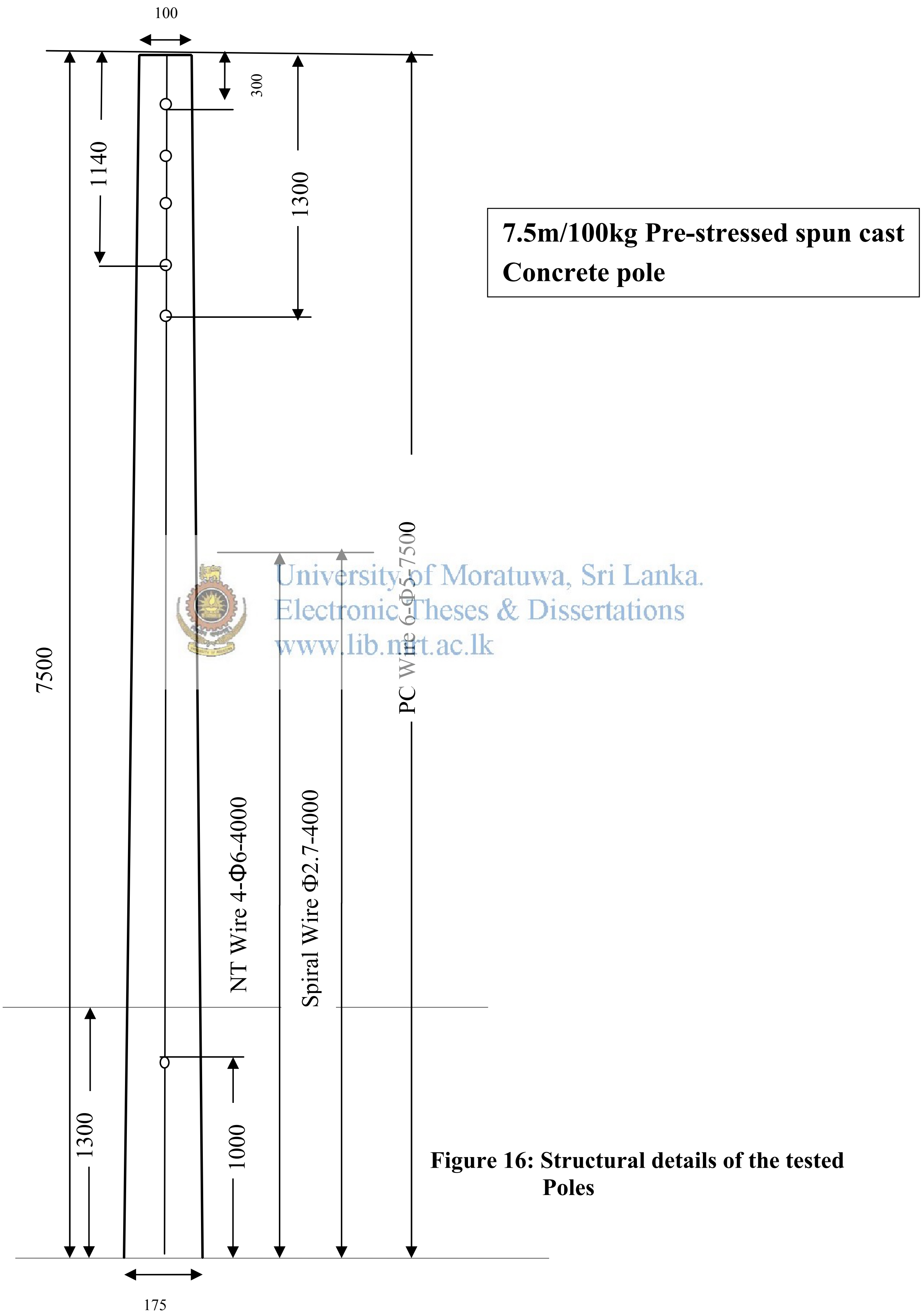


Figure 16: Structural details of the tested Poles

Table 23: Specifications of the tested poles

Note:	
Item	Requirement/Tolerance
Design	
Standards Transverse Load Factor of Safety Pole Taper	BS607, SLS363 112kg(1.1kN) > 2.0 x design load 1/100
Dimension	
Length Top diameter Bottom diameter Wall thickness(Min) Concrete cover Weight(Min)	7500 + 25mm - 10mm 100 + 4mm - 2mm 160mm + 4mm - 2mm 30mm >13mm 210kg
Material	
Concrete PC wire(diameter) NT wire(diameter) Spiral wire(diameter)	Min. Grade 50 (50MPa) at age of 28 days 5mm 6mm 2.7mm

CHAPTER 6

6.1. Conclusions

An efficiently designed poles are one in which the weight, loads and forces are transmitted to the foundations by the cheapest means consistent with the intended use of the pole and the nature of the ground situation.

By applying pre-stressing forces to concrete pole can considerably improve the flexural rigidity of the member. Those zones of the member ultimately required to carry tensile stresses under working load conditions, are given an initial compressive stress before the application of working loads so that crack width can be reduced.

Pre-stressed concrete poles are being widely used in many countries as utility poles. These poles have many advantages when compared to traditional poles currently used by the Sri Lanka Telecom. The manufacturing process is a significant process and involves many steps as seen in preceding report. This thesis is mainly focused on the poles currently used by the Sri Lankan Telecom for their wire stretching and distribution purposes. The main factors considered in manufacturing and designing of such poles are economy, required strength and durability. The various materials used are steel, cement, aggregates, admixtures like curing compounds and hardening compounds etc.

The objective of this research work is to study on current designs and their pros and cons, applicable code standards, manufacturing process and simulation of structural performance of poles under recommended loading criteria by modern analytical tools.

At the initial stage of study variety of sections were analyzed under applicable loading criteria to select most suitable one. Sections under considered for analysis were square solid section with pre-stressed reinforcements, square hollow section with normal and pre-stressed reinforcements, circular section with normal and pre-stressed reinforcement, circular hollow section with normal and pre-stressed reinforcements. SAP2000 [11] finite element programme was used to analyze the poles under different loading conditions as specified.

Preceding analytical and experimental results revealed that the circular hollow section with pre-stressed reinforcements shows the best option and optimum solution to cope with the requirements with respect to flexural strength, light weight and the economy.

Proposed designs are immensely help Sri Lanka Telecom and their manufacturing organization to producing concrete pole in comparatively less production cost. It is further suggested that usual casting practices will be improved with spun casting mechanism, and reinforcement steel with pre-stressing reinforcements coupled with high strength concrete such as grade 50 concrete. This will result in superior quality high strength, less weight and economical product. Proposed and current pole's geometric properties are compared in Table 10 to show the weight reduction in proposed designs. Initial investment will be necessary to secure new equipments and machineries for production purposes but long term results will be much benefitted to the investors in terms of producing good quality and durable concrete poles with less production cost.

6.2.Recommendations for the future work

Carbon-fiber-reinforced polymer (CFRP) reinforcement shows immense potential in civil engineering applications as an alternative to traditional steel reinforcement because of its unique properties. CFRP is high strength, light weight, noncorrosive, and nonmagnetic. The improved durability of CFRP-reinforced concrete has caused CFRP to gain considerable use and attention in the reinforced concrete field.

Most research on the use of CFRP in concrete structures has focused on the rectangular and tee cross-sectional shapes commonly used in building and bridges. Limited information, however, is available in the literature on circular concrete sections reinforced with CFRP. Terrasi and Lees [20] tested centrifugally cast high-strength concrete poles reinforced with CFRP wires, manufactured in Switzerland. Their weight was about 30% less than for comparable conventional steel reinforced concrete poles.

Members with circular cross sections are commonly used in the precast concrete industry for poles, piles, pipes, and columns for buildings and bridge piers. Round spun concrete poles are used in supporting electric transmission lines, communication towers, stadium lighting, and a variety of other applications. The round cross sections which is dictated by the manufacturing process in the case of spun concrete offers a number of advantages, including a smooth finish, denser concrete material, reduced wind pressure, and improved aesthetics. Carbon-fiber-reinforced polymer (CFRP) composites show potential as a replacement for steel reinforcement because of their corrosion resistance, high strength, and light weight.

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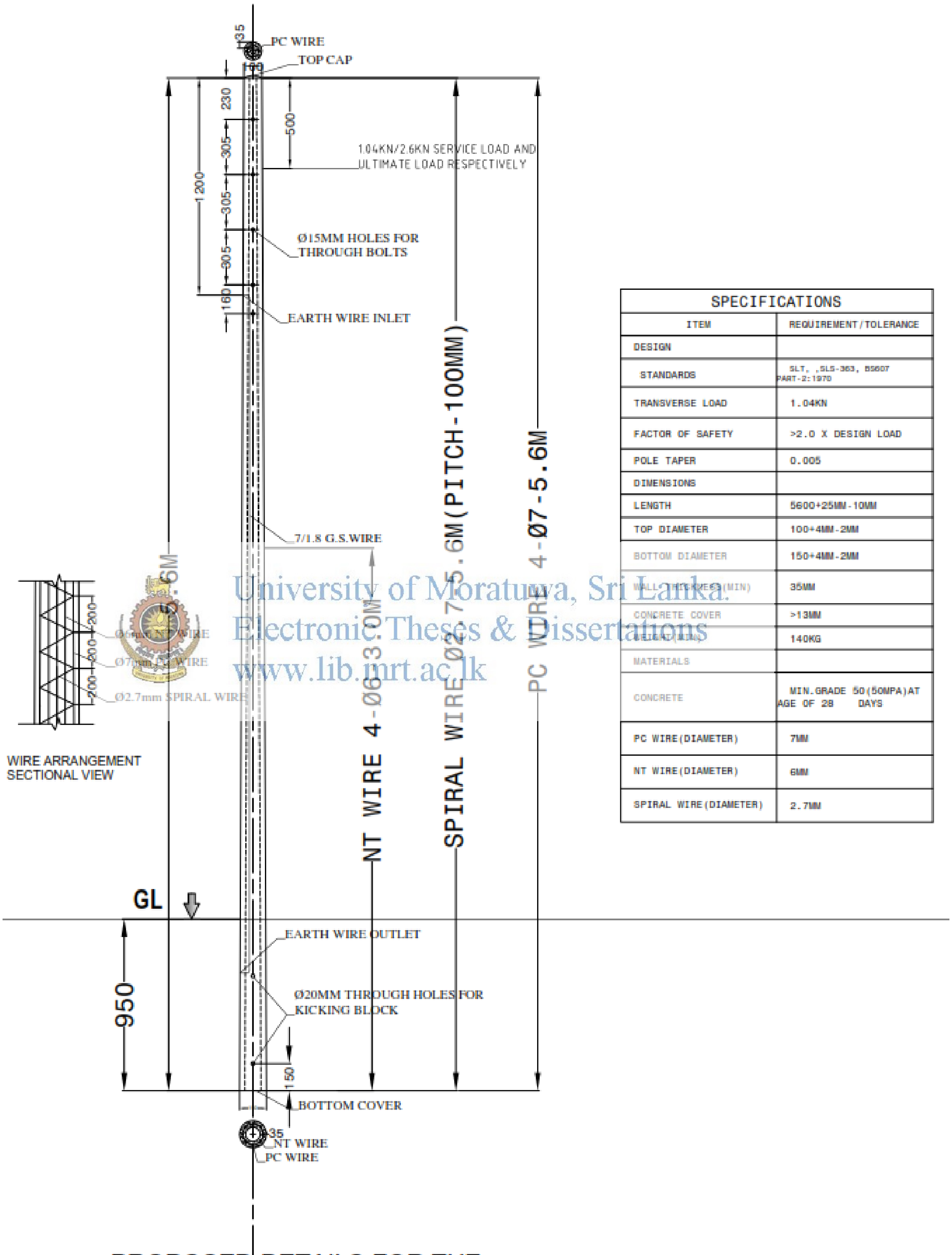
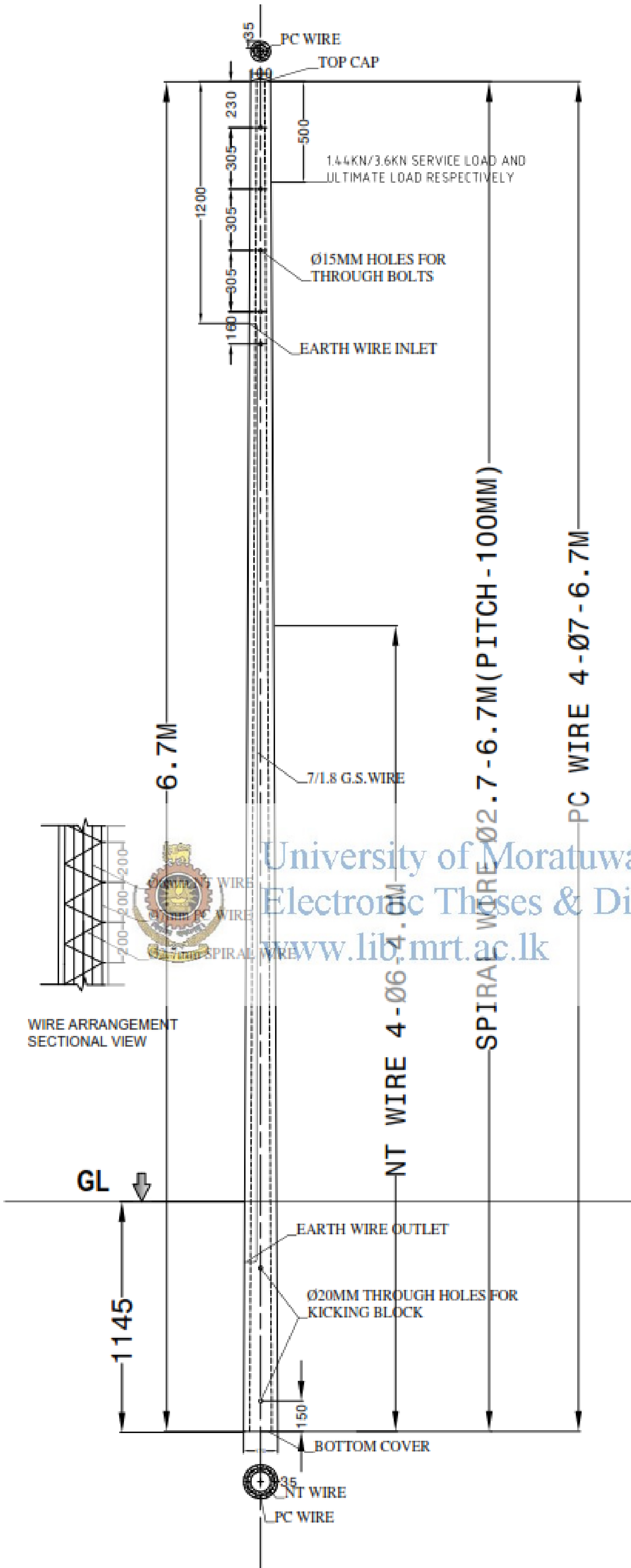
APPENDIXES

APPENDIX – A: Structural drawings of proposed pre-stressed concrete poles



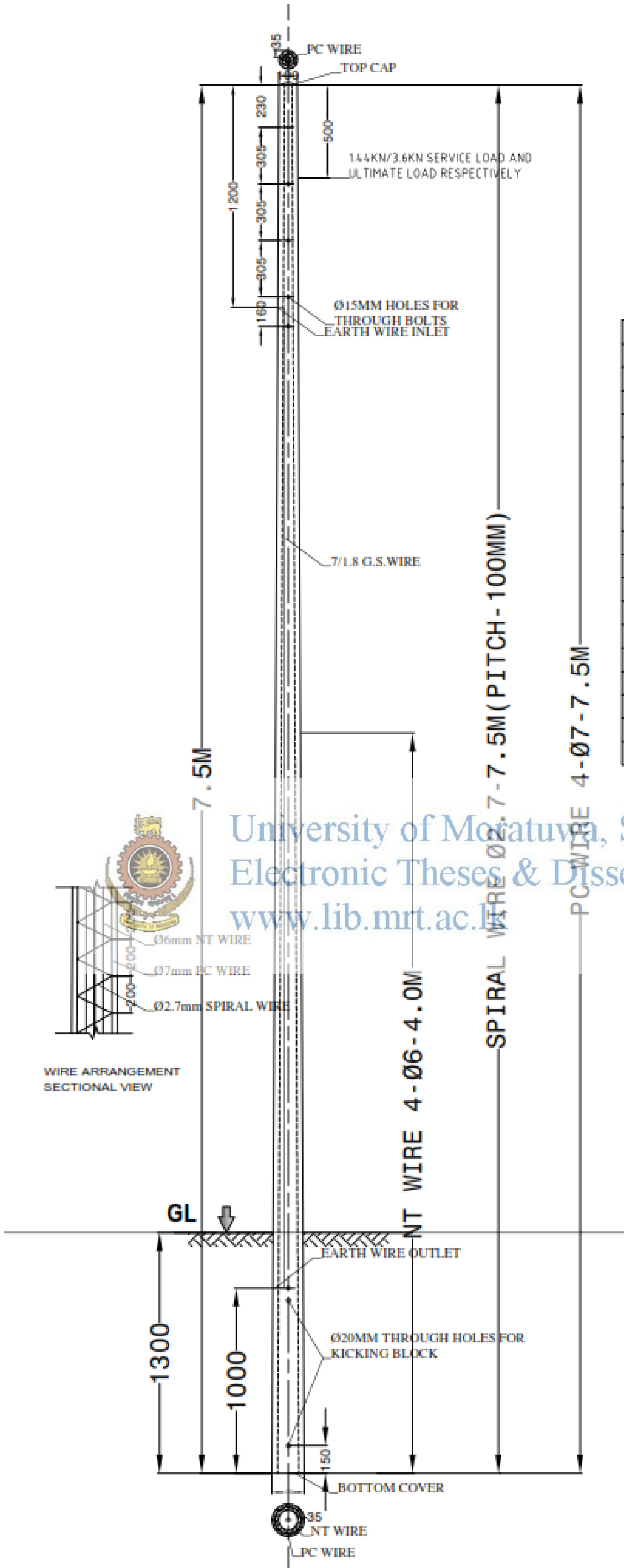
SPECIFICATIONS	
ITEM	REQUIREMENT/TOLERANCE
DESIGN	
STANDARDS	SLT, ,SLS-363, BS607 PART-2:1970
TRANSVERSE LOAD	1.04KN
FACTOR OF SAFETY	>2.0 X DESIGN LOAD
POLE TAPER	0.005
DIMENSIONS	
LENGTH	5600+25MM-10MM
TOP DIAMETER	100+4MM-2MM
BOTTOM DIAMETER	150+4MM-2MM
WALL THICKNESS(MIN)	35MM
CONCRETE COVER	>13MM
WEIGHT(MIN)	140KG
MATERIALS	
CONCRETE	MIN.GRADE 50(50MPA)AT AGE OF 28 DAYS
PC WIRE(DIAMETER)	7MM
NT WIRE(DIAMETER)	6MM
SPIRAL WIRE(DIAMETER)	2.7MM

PROPOSED DETAILS FOR THE
5.6M/140KG PRE-STRESSED SPUN CAST CONCRETE POLE



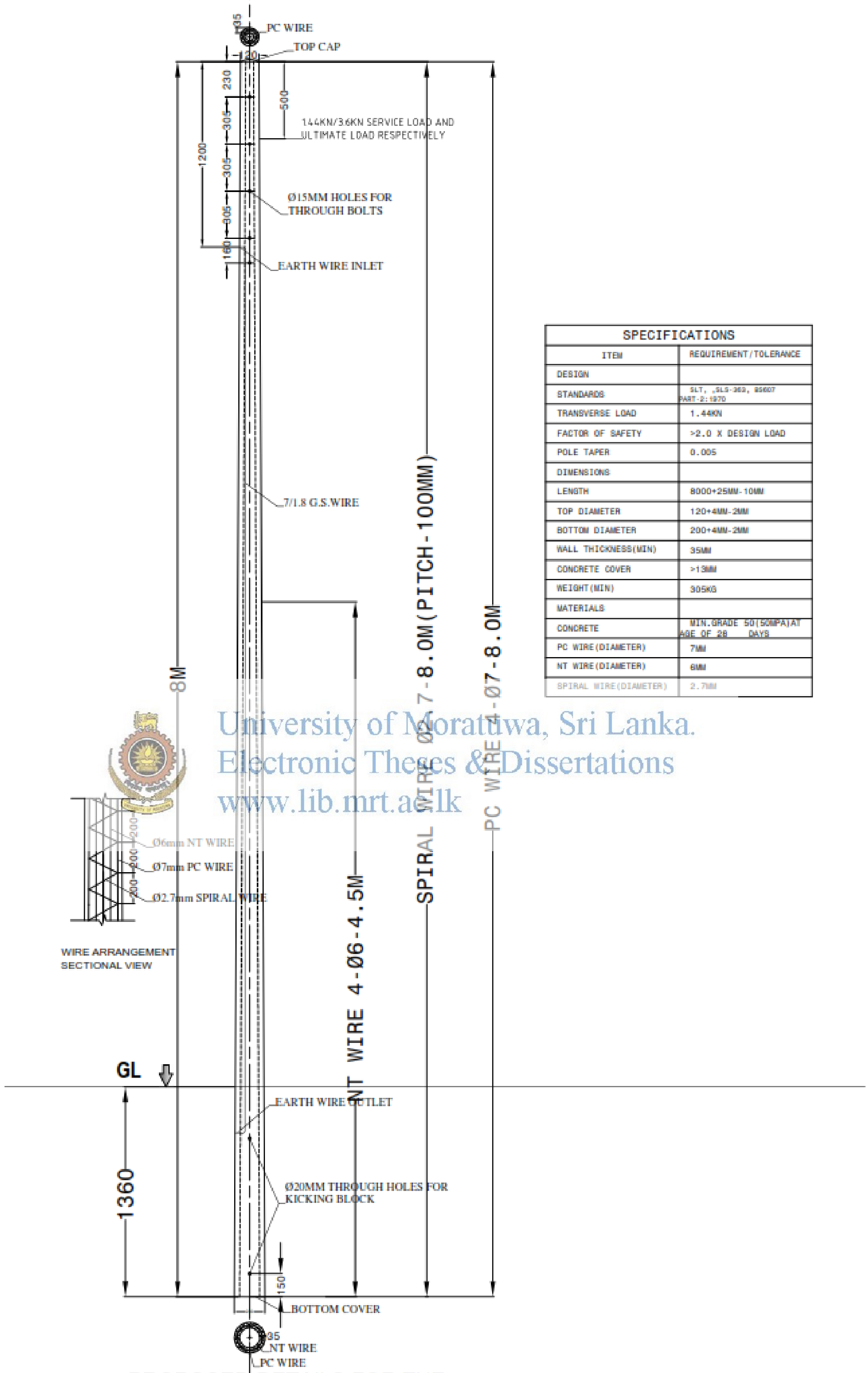
SPECIFICATIONS	
ITEM	REQUIREMENT/TOLERANCE
DESIGN	
STANDARDS	SLT, ,SLS-363, BS607 PART-2:1970
TRANSVERSE LOAD	1.44KN
FACTOR OF SAFETY	>2.0 X DESIGN LOAD
POLE TAPER	0.005
DIMENSIONS	
LENGTH	6700+25MM-10MM
TOP DIAMETER	100+4MM-2MM
BOTTOM DIAMETER	170+4MM-2MM
WALL THICKNESS(MIN)	35MM
CONCRETE COVER	>13MM
WEIGHT(MIN)	184KG
MATERIALS	
CONCRETE	MIN.GRADE 50(50MPA)AT AGE OF 28 DAYS
PC WIRE(DIAMETER)	7MM
NT WIRE(DIAMETER)	6MM
SPIRAL WIRE(DIAMETER)	2.7MM

PROPOSED DETAILS FOR THE
6.7M/184KG PRE-STRESSED SPUN CAST CONCRETE POLE



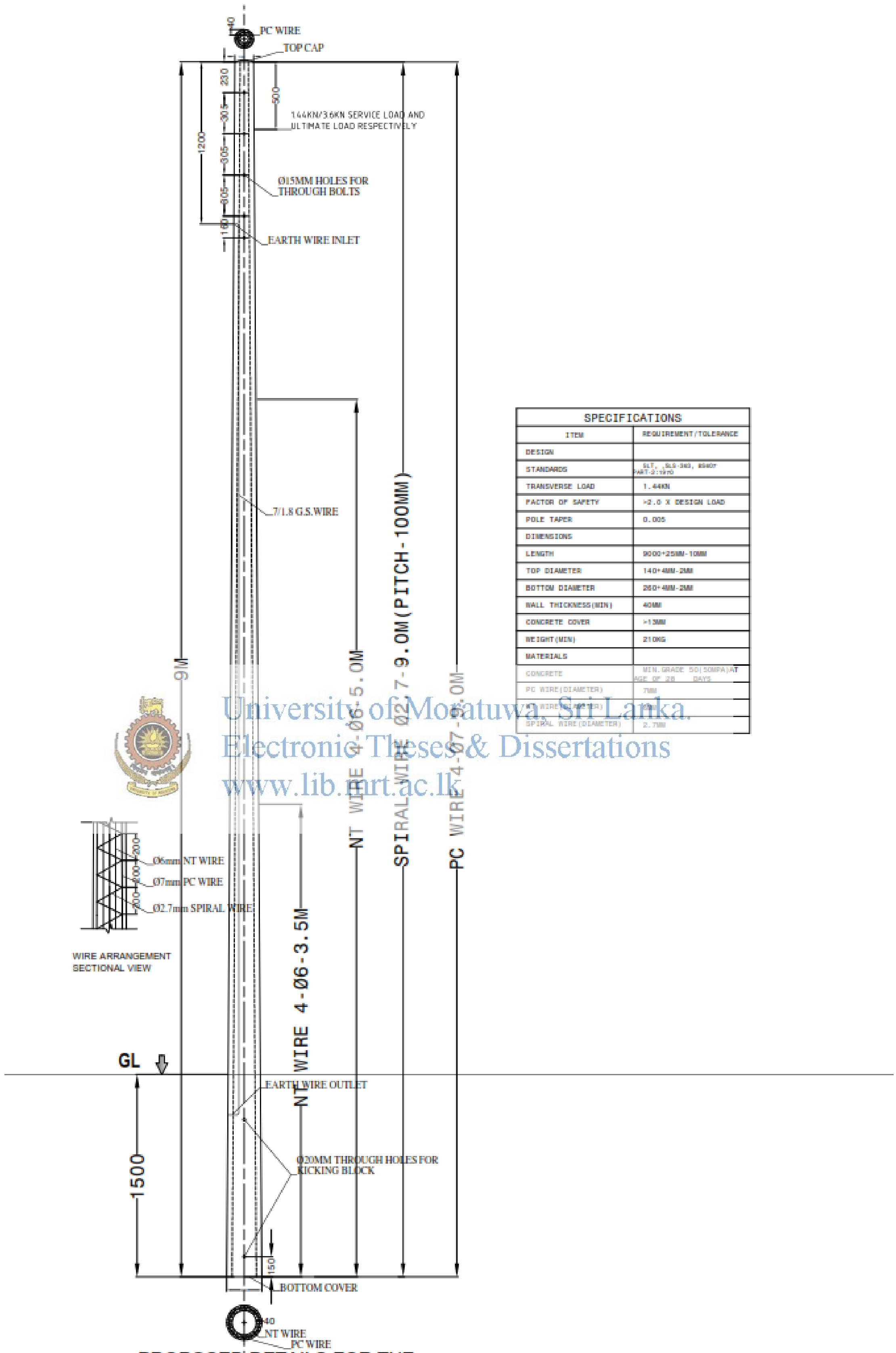
SPECIFICATIONS	
ITEM	REQUIREMENT/TOLERANCE
DESIGN	
STANDARDS	SLT, ,SLS-363, BS607 PART-2:1970
TRANSVERSE LOAD	1.44KN
FACTOR OF SAFETY	>2.0 X DESIGN LOAD
POLE TAPER	0.005
DIMENSIONS	
LENGTH	7500+25MM-10MM
TOP DIAMETER	100+4MM-2MM
BOTTOM DIAMETER	175+4MM-2MM
WALL THICKNESS(MIN)	35MM
CONCRETE COVER	>13MM
WEIGHT(MIN)	210KG
MATERIALS	
CONCRETE	MIN.GRADE 50(50MPA)AT AGE OF 28 DAYS
PC WIRE(DIAMETER)	7MM
NT WIRE(DIAMETER)	6MM
SPIRAL WIRE(DIAMETER)	2.7MM

PROPOSED DETAILS FOR THE
7.5M/210KG PRE-STRESSED SPUN CAST CONCRETE POLE



SPECIFICATIONS	
ITEM	REQUIREMENT/TOLERANCE
DESIGN	
STANDARDS	SLT, ,SLS-363, BS607 PART-2:1970
TRANSVERSE LOAD	1.44KN
FACTOR OF SAFETY	>2.0 X DESIGN LOAD
POLE TAPER	0.005
DIMENSIONS	
LENGTH	8000+25MM-10MM
TOP DIAMETER	120+4MM-2MM
BOTTOM DIAMETER	200+4MM-2MM
WALL THICKNESS(MIN)	35MM
CONCRETE COVER	>13MM
WEIGHT(MIN)	305KG
MATERIALS	
CONCRETE	MIN.GRADE 50(50MPA)AT AGE OF 28 DAYS
PC WIRE(DIAMETER)	7MM
NT WIRE(DIAMETER)	6MM
SPIRAL WIRE(DIAMETER)	2.7MM

PROPOSED DETAILS FOR THE
8.0M/305KG PRE-STRESSED SPUN CAST CONCRETE POLE

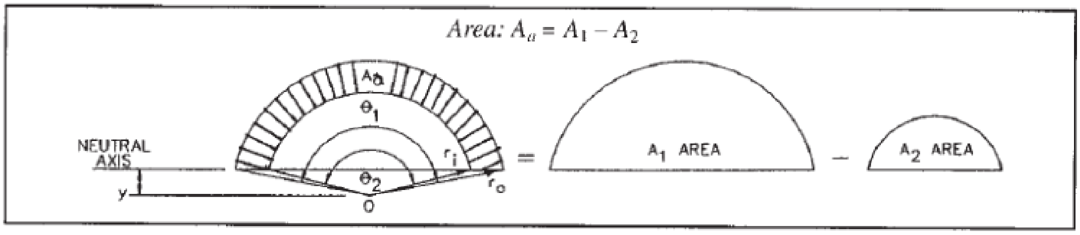


SPECIFICATIONS	
ITEM	REQUIREMENT/TOLERANCE
DESIGN	
STANDARDS	SLT, SLS-363, [illegible]
TRANSVERSE LOAD	1.44KN
FACTOR OF SAFETY	>2.0 X DESIGN LOAD
POLE TAPER	0.005
DIMENSIONS	
LENGTH	9000+25MM-10MM
TOP DIAMETER	140+4MM-2MM
BOTTOM DIAMETER	260+4MM-2MM
WALL THICKNESS(MIN)	40MM
CONCRETE COVER	>13MM
WEIGHT(MIN)	210KG
MATERIALS	
CONCRETE	MIN.GRADE 50(50MPA)AT AGE OF 28 DAYS
PC WIRE(DIAMETER)	7MM
[illegible]	[illegible]
SPIRAL WIRE(DIAMETER)	2.7MM

PROPOSED DETAILS FOR THE
9.0M/502KG PRE-STRESSED SPUN CAST CONCRETE POLE

APPENDIX – B: Area and centroid of annulus [4]

AREA AND CENTROID OF ANNULUS



Area of Annulus

1. Determination of A_1 area.

Consider S, half of the area A_1:

$$r_o^2 = x^2 + y^2,\ \text{hence}\ y = \sqrt{r_o^2 - x^2}$$

$$S = \int_d^{r_o} dA = \int_d^{r_o} y\,dx = \int_d^{r_o} \sqrt{r_o^2 - x^2}\,dx$$

Let: $x = r_o \cos\phi$ and $dx = (-r_o \sin\phi)d\phi$

Therefore:

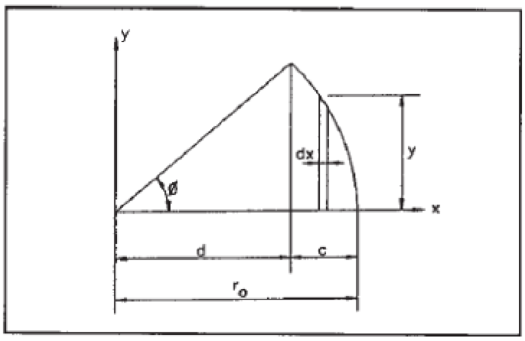
$$S = -\int_\phi^o \sqrt{r_o^2 - r_o^2\cos^2\phi}\,(r_o \sin\phi)d\phi$$

$$= -\int_\phi^o r_o^2 \sin\phi \sin\phi\,d\phi$$

$$= -r_o^2 \int_\phi^o \sin^2\phi\,d\phi$$

$$= -r_o^2 \left(\frac{\phi}{2} - \frac{\sin^2\phi}{4}\right)_\phi^o$$

$$A_1 = \frac{r_o^2}{2}(2\phi - \sin 2\phi)\ \text{or}\ A_1 = \frac{r_o^2}{2}(\Theta_1 - \sin\Theta_1)$$



in which the central angle $\Theta_1 = 2\phi$:

$$\Theta_1/2 = \tan^{-1}\left(\sqrt{r_o^2 - y^2}/y\right)$$

2. Determination of A_2 area.

$$A_2 = \frac{r_i^2}{2}(\Theta_2 - \sin\Theta_2)$$

with radius r_i and central angle Θ_2:

$$\Theta_2/2 = \tan^{-1}\left(\sqrt{r_i^2 - y^2}/y\right)$$

Centroid of Annulus

The centroid of the annulus is the point of intersection of two axes which place the body in equilibrium. Axis 1 bisects the area of the annulus and Axis 2 divides the annulus into two equal areas.

Area of annulus:

$$A_a = \tfrac{1}{2}r_o^2(\Theta_1 - \sin\Theta_1) - \tfrac{1}{2}r_i^2(\Theta_2 - \sin\Theta_2)$$

where

$$\Theta_1/2 = \tan^{-1}\left(\sqrt{r_o^2 - y^2}/y\right)$$

and

$$\Theta_2/2 = \tan^{-1}\left(\sqrt{r_i^2 - y^2}/y\right)$$

Area above Axis 2:

$$A'_a = \tfrac{1}{2}r_o^2(\Theta_3 - \sin\Theta_3) - \tfrac{1}{2}r_i^2(\Theta_4 - \sin\Theta_4)$$

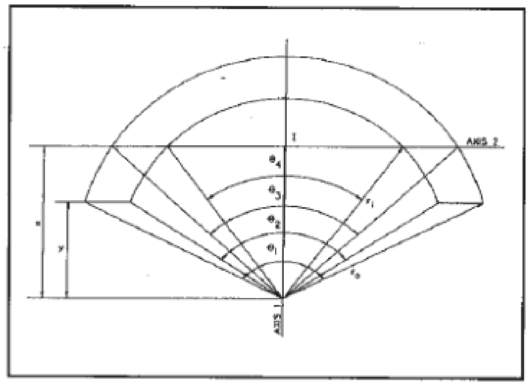
where

$$\Theta_3/2 = \tan^{-1}\left(\sqrt{r_o^2 - x^2}/x\right)$$

and

$$\Theta_4 = \tan^{-1}\left(\sqrt{r_i^2 - x^2}/x\right)$$

Since y is known, x is found by trial and error until $A'_a = \tfrac{1}{2}A_a$.



APPENDIX – C: Characteristic of pre-stressing steel [4]

Galvanized stress relieved strand

Nominal strand diameter mm	in.	Grade MPa	ksi	Minimum breaking load kN	lbs	Minimum load at 1 percent extension kN	lbs	Nominal steel area* mm²	sq in.
9.53	3/8	1725	250	94.5	21,250	75.6	17,000	54.84	0.085
11.11	7/16	1725	250	127.7	28,700	102.1	22,950	74.19	0.115
12.70	1/2	1725	250	169.9	38,200	136.1	30,000	98.71	0.153
12.70	1/2	1860	270	183.7	41,300	156.1	35,100	98.71	0.153

* Steel area prior to galvanizing.

Uncoated stress relieved strand ASTM A416

Nominal strand diameter		Grade		Minimum breaking load		Minimum load at 1 percent extension				Nominal steel area	
						Normal relaxation		Low relaxation			
mm	in.	MPa	ksi	kN	lbs	kN	lbs	kN	lbs	mm²	sq in.
7.94	5/16	1725	250	64.5	14,000	54.7	12,300	58.1	13,050	37.42	0.058
7.94	5/16	1860	270	71.2	16,000	60.5	13,600	64.1	14,400	38.06	0.059
9.53	3/8	1860	270	102.3	23,000	87.0	19,550	92.1	20,700	54.84	0.085
11.11	7/16	1860	270	137.9	31,000	117.2	26,350	124.1	27,900	74.19	0.115
12.70	1/2	1860	270	183.7	41,300	156.1	35,100	165.3	37,170	98.71	0.153
15.24	0.6	1860	270	260.7	58,600	221.5	49,800	234.6	52,740	140.00	0.217

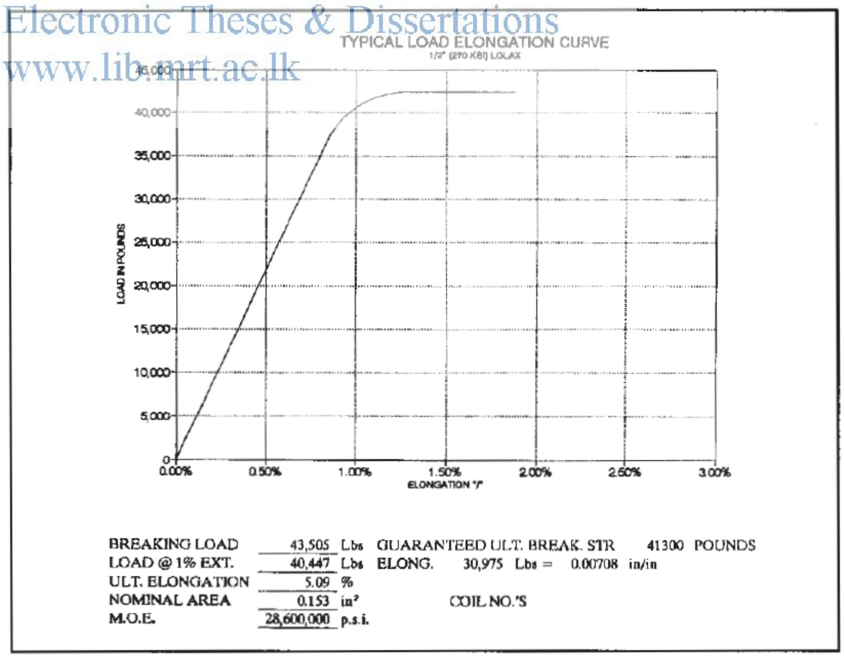
Uncoated stress relieved wire ASTM A421

Nominal wire diameter mm	in.	Grade MPa	ksi	Minimum breaking load kN	lbs	Minimum load at 1 percent extension kN	lbs
5	0.196	1655	240	33.6	7550	28.6	6418
6.35	1/4	1655	240	52.4	11,780	44.5	10,013
7	0.276	1620	235	62.5	14,050	53.1	11,943

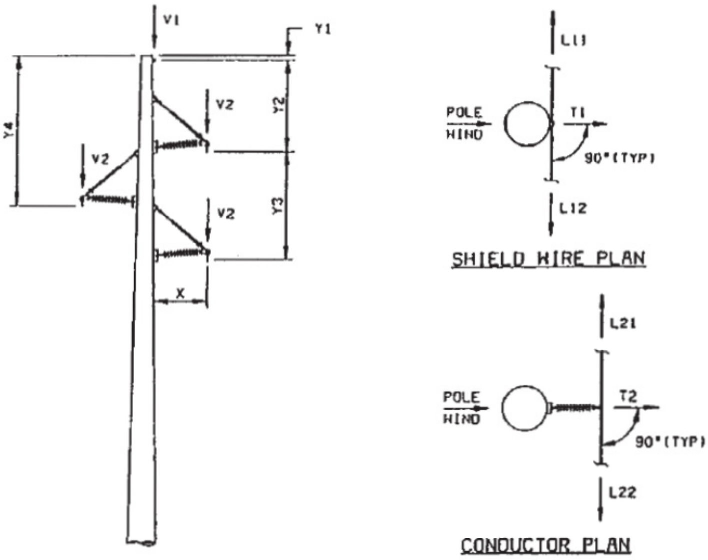
Table 3.2.2. Permissible stresses of prestressing steel.

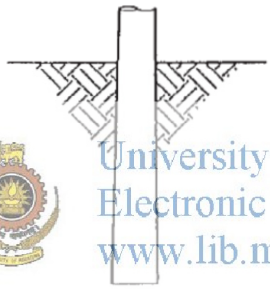
1	Due to jacking force but not greater than $0.80f_{pu}$ or maximum value recommended by manufacturer of prestressing steel or anchorages.	$0.94f_{py}$
2	Immediately after prestress transfer but not greater than $0.74f_{pu}$.	$0.82f_{py}$
3	Post-tensioning steel at anchorages and couplers immediately after anchorage.	$0.70f_{pu}$

Fig. 3.2.1. Load elongation curve for 1/2 in. (12.5 mm) diameter stress relieved seven-wire strand Grade 270.



APPENDIX – D: Typical load tree for concrete pole [4]





LOAD TABLE

LOAD DESCRIPTION	LOAD CASE 1	2	3	4	5
POLE WIND	0.036	0.023	0.026	0.000	0.009
TRANSVERSE LOAD - T1	[illegible]	0.33	0.32	0.08	0.15
VERTICAL LOAD - V1	[illegible]	0.25	[illegible]	0.50	0.17
LONGITUDINAL LOAD - L11	2.75	2.60	1.03	1.24	2.36
LONGITUDINAL LOAD - L12	2.75	2.60	1.03	1.44	2.36
TRANSVERSE LOAD - T2	1.42	0.86	0.86	0.04	0.38
VERTICAL LOAD - V2	0.51	0.69	0.46	2.80	0.46
LONGITUDINAL LOAD - L21	6.05	5.36	4.25	2.02	4.87
LONGITUDINAL LOAD - L22	6.05	5.36	4.25	2.02	4.87

CASE DESCRIPTION

CASE 1 - ASCE EXTREME WIND, VERTICAL OLF = 1.1
CASE 2 - NESC LIGHT, VERTICAL OLF = 1.50
CASE 3 - NESC EXTREME WIND, VERTICAL OLF = 1.0
CASE 4 - CONSTRUCTION LOAD, VERTICAL OLF = 1.0
CASE 5 - SPECIFIED CRACKING LOAD, VERTICAL OLF = 1.0

NOTES

1. WIRE LOADS ARE IN KIPS. POLE WIND IS IN KIPS PER SQUARE FOOT.
2. LOADS SHOWN INCLUDE OVERLOAD FACTORS.
3. EACH LOADING CASE SHALL BE CONSIDERED INDEPENDENTLY.

APPENDIX – E: Structural calculations for the 8m, 7.5m, 6.7m and 5.6m poles

8m high Pre-stressed Pole

Analysis of critical section of the pole (at ground support position)		
r_1 (Radios of outer circle) =	100	mm
r_2 (Radios of inner circle) =	65	mm
Thickness of pole =	35.00	mm
Gross concrete area at base A_g	18,142.70	mm^2
Assume C= Neutral axis depth by trial and error	106.50	mm
β_1= Parameter to calculate rectangular concrete compressive stress block	0.69	
$\beta_1 C$ =	73.49	mm
f'_c = Concrete compressive strength	50.00	N/mm^2
$f_r=0.62\sqrt{f'_c}$ Modulus of rupture of concrete	4.38	N/mm^2
y_t = Distance from centroid axis to extreme tensile fibre	93.50	mm
I_g = Gross moment of inertia of the section	6.45E-05	m^4
Finding area of Annulus		
Φ_1 =	3.27	radians
Φ_2 =	3.34	radians
Area A_1 =	17,007.05	mm^2
Area A_2 =	7,480.20	mm^2
Therefore Annulus area $A_a = A_1 - A_2$	**9,526.84**	mm^2
Finding centroid of annulus A_a		
Assume distance to the centroid from the centre x=	15.00	mm
Φ_3 =	2.84	radians

Φ_4 =	2.68	radians
A'_a =	**8,015.19**	mm^2
Find the "x" so that $A_a \approx 2A'_a$		
Cross sectional area of Φ7mm strand A_{psi} =	38.48	mm^2
Number of strands	4	No's
Modulus of elasticity of pre-stressing steel E_s =	205.00	kN/mm^2
Modulus of elasticity of normal steel E_y =	200.00	kN/mm^2
Modulus of elasticity of concrete E_c =	31.75	kN/mm^2
f_{py} = Specified yield stress of pre-stressing steel	1620.00	N/mm^2
Total pre-stressing force per strand = F_{py}	62.34	kN
Minimum breaking load F_{pu} =	62.50	kN
Therefore permissible pre-stressing force per strand = lesser of $0.80f_{pu}$ and $0.94f_{py}$ (assume 10% loss due to relaxation)=	50.00	kN
Assessment of transmission length: $l_t = K_t\Phi/\sqrt{f_{ci}}$	593.97	mm
where K_t is a coefficient for tendons =	600.00	
Average diameter of the section	160.00	mm
Average cross sectional area of concrete	20106.19	mm^2
Exposed perimeter of the section	502.65	mm
Effective section thickness of concrete(under immersed conditions)	600.00	mm
Elastic deformation of concrete at the age of stress transfer	2.51	mm
Creep strain $\mathcal{E}_{cc}$=stress x ø/E_t	4.70E-04	
Therefore creep deformation of concrete	3.76	mm
where ø = creep coefficient	1.50	
E_t = Modulus of elasticity of concrete at the age of t = E_c		

Design as class 3 member with 0.1mm crack width at ultimate loading

For grade 50 concrete for limiting the crack width to 0.1mm

Design flexural stress for class 3 member f_r =	4.80	N/mm^2
Design compressive stress at extreme fibre should not exceed $0.5f_{ci}$		
where f_{ci} is the concrete strength at transfer		
$0.5\ f_{ci}$ =	25.00	N/mm^2
Concrete stresses due to effective pre-stresses = < $\lvert 0.5f_{ci} \rvert$	-11.02	N/mm^2
Concrete stresses due to bending: assume compression " - "		
Concrete compression at compression zone	-5.47	N/mm^2
Therefore maximum concrete compression = < $\lvert 0.5f_{ci} \rvert$	-16.49	N/mm^2
Maximum concrete tension for class 3 member f_r =	4.80	N/mm^2
Strain at extreme fibre at tension zone =	1.51E-04	
Calculation of steel stresses and moment about neutral axis :		
Area of pre-stressing strand =	38.48	mm^2
Strands stresses due to effective pre-stresses =	1299.22	N/mm^2
Strands stresses-1 due to bending at tension zone=	-2.15	N/mm^2
Moment about neutral axis	-0.65	kNm
Strands stresses-2 due to bending at tension zone=	25.19	N/mm^2
Moment about neutral axis	9.30	kNm
Normal steel stresses due to bending =	16.76	N/mm^2
Area of normal steel =	56.55	mm^2
Moment about neutral axis	0.05	kNm
Total moment about neutral axis due to steel tensions	**8.70**	kNm
Concrete compression $C_c = 0.85f_c'A_a$	404.89	kN
Centroid distance	21.50	mm
Moment about neutral axis due to concrete compression	**8.71**	kNm

Trial and error to find the value of "C" so that above two figures are almost equal		
Zero tension moment $M_0 = P\, I_g/(A_g\, y_t)$	**7.61**	kNm
Cracking moment $M_{cr} = P\, I_g/(A_g\, y_t)+f_r\, I_g/y_t$	**10.92**	kNm
Service moment of the pole =< M_{cr}	**10.80**	kNm
Therefore pole design is satisfactory for given service requirements		

7.5m high Pre-stressed Pole

Analysis of critical section of the pole (at ground support position)		
r_1 (Radios of outer circle) =	87.5	mm
r_2 (Radios of inner circle) =	52.5	mm
Thickness of pole =	35.00	mm
Gross concrete area at base A_g	15,393.80	mm^2
Assume C= Neutral axis depth by trial and error	104.50	mm
β_1= Parameter to calculate rectangular concrete compressive stress block	0.69	
$\beta_1 C$ =	72.11	mm
f'_c = Concrete compressive strength	50.00	N/mm^2
$f_r = 0.62\sqrt{f'_c}$ Modulus of rupture of concrete	4.38	N/mm^2
y_t = Distance from centroid axis to extreme tensile fibre	70.50	mm
I_g = Gross moment of inertia of the section	4.01E-05	m^4
Finding area of Annulus		
Φ_1 =	3.53	radians
Φ_2 =	3.80	radians
Area A_1 =	14,982.59	mm^2
Area A_2 =	6,082.80	mm^2
Therefore Annulus area $A_a = A_1 - A_2$	**8,899.78**	mm^2
Finding centroid of annulus A_a		
Assume distance to the centroid from the centre x=	-0.30	mm
Φ_3 =	3.15	radians
Φ_4 =	3.15	radians

A'_a =	**7,717.90**	mm^2
Find the "x" so that $A_a \approx 2A'_a$		
Cross sectional area of Φ7mm strand A_{psi} =	38.48	mm^2
Number of strands	4	No's
Modulus of elasticity of pre-stressing steel E_s =	205.00	kN/mm^2
Modulus of elasticity of normal steel E_y =	200.00	kN/mm^2
Modulus of elasticity of concrete E_c =	31.75	kN/mm^2
f_{py} = Specified yield stress of pre-stressing steel	1620.00	N/mm^2
Total pre-stressing force per strand = F_{py}	62.34	kN
Minimum breaking load F_{pu} =	62.50	kN
Therefore permissible pre-stressing force per strand = lesser of $0.80f_{pu}$ and $0.94f_{py}$ = (assume 10% loss due to relaxation)	50.00	kN
Assessment of transmission length: $l_t = K_t\Phi/\sqrt{f_{ci}}$	593.97	mm
where K_t is a coefficient for tendons =	600.00	
Average diameter of the section	137.50	mm
Average cross sectional area of concrete	16198.84	mm^2
Exposed perimeter of the section	431.97	mm
Effective section thickness of concrete(under immersed conditions)	600.00	mm
Elastic deformation of concrete at the age of stress transfer	2.92	mm
Creep strain $\mathcal{E}_{cc}$=stress x ø/E_t	5.83E-04	
Therefore creep deformation of concrete	4.37	mm
where ø = creep coefficient	1.50	
E_t = Modulus of elasticity of concrete at the age of t = E_c		

Design as class 3 member with 0.1mm crack width at ultimate loading

For grade 50 concrete for limiting the crack width to 0.1mm

Design flexural stress for class 3 member f_r =	4.80	N/mm^2
Design compressive stress at extreme fibre should not exceed $0.5f_{ci}$		
where f_{ci} is the concrete strength at transfer		
$0.5\ f_{ci}$ =	25.00	N/mm^2
Concrete stresses due to effective pre-stresses = < $\lvert 0.5f_{ci} \rvert$	-12.99	N/mm^2
Concrete stresses due to bending: assume compression " - "		
Concrete compression at compression zone	-7.11	N/mm^2
Therefore maximum concrete compression = < $\lvert 0.5f_{ci} \rvert$	-20.11	N/mm^2
Maximum concrete tension for class 3 member f_r =	4.80	N/mm^2
Strain at extreme fibre at tension zone =	1.51E-04	
Calculation of steel stresses and moment about neutral axis :		
Area of pre-stressing strand =	38.48	mm^2
Strands stresses due to effective pre-stresses =	1299.22	N/mm^2
Strands stresses-1 due to bending at tension zone=	-7.47	N/mm^2
Moment about neutral axis	-1.69	kNm
Strands stresses-2 due to bending at tension zone=	23.30	N/mm^2
Moment about neutral axis	8.02	kNm
Normal steel stresses due to bending =	13.94	N/mm^2
Area of normal steel =	56.55	mm^2
Moment about neutral axis	0.03	kNm
Total moment about neutral axis due to steel tensions	**6.35**	kNm
Concrete compression $C_c = 0.85f_c'A_a$	378.24	kN
Centroid distance	16.70	mm
Moment about neutral axis due to concrete compression	**6.32**	kNm

Trial and error to find the value of "C" so that above two figures are almost equal		
Zero tension moment $M_0 = P\ I_g/(A_g\ y_t)$	**7.38**	kNm
Cracking moment $M_{cr} = P\ I_g/(A_g\ y_t)+f_r\ I_g/y_t$	**10.11**	kNm
Service moment of the pole =< M_{cr}	**10.08**	kNm
Therefore pole design is satisfactory for given service requirements		

6.7m high Pre-stressed Pole

Analysis of critical section of the pole (at ground support position)		
r_1 (Radios of outer circle) =	85	mm
r_2 (Radios of inner circle) =	50	mm
Thickness of pole =	35.00	mm
Gross concrete area at base A_g	14,844.03	mm^2
Assume C= Neutral axis depth by trial and error	97.00	mm
β_1= Parameter to calculate rectangular concrete compressive stress block	0.69	
$\beta_1 C$ =	66.93	mm
f'_c = Concrete compressive strength	50.00	N/mm^2
$f_r = 0.62\sqrt{f'_c}$ Modulus of rupture of concrete	4.38	N/mm^2
y_t = Distance from centroid axis to extreme tensile fibre	73.00	mm
I_g = Gross moment of inertia of the section	3.61E-05	m^4
Finding area of Annulus		
Φ_1 =	3.42	radians
Φ_2 =	3.63	radians
Area A_1 =	13,382.21	mm^2
Area A_2 =	5,115.37	mm^2
Therefore Annulus area $A_a = A_1 - A_2$	**8,266.84**	mm^2
Finding centroid of annulus A_a		
Assume distance to the centroid from the centre x=	7.00	mm
Φ_3 =	2.98	radians
Φ_4 =	2.86	radians

A'_a =	**6,931.07**	mm^2
Find the "x" so that $A_a \approx 2A'_a$		
Cross sectional area of Φ7mm strand A_{psi} =	38.48	mm^2
Number of strands	4	No's
Modulus of elasticity of pre-stressing steel E_s =	205.00	kN/mm^2
Modulus of elasticity of normal steel E_y =	200.00	kN/mm^2
Modulus of elasticity of concrete E_c =	31.75	kN/mm^2
f_{py} = Specified yield stress of pre-stressing steel	1620.00	N/mm^2
Total pre-stressing force per strand = F_{py}	62.34	kN
Minimum breaking load F_{pu} =	62.50	kN
Therefore permissible pre-stressing force per strand = lesser of $0.80f_{pu}$ and $0.94f_{py}$ (assume 10% loss due to relaxation) =	50.00	kN
Assessment of transmission length: $l_t = K_t\Phi/\sqrt{f_{ci}}$	593.97	mm
where K_t is a coefficient for tendons =	600.00	
Average diameter of the section	135.00	mm
Average cross sectional area of concrete	14844.03	mm^2
Exposed perimeter of the section	424.12	mm
Effective section thickness of concrete(under immersed conditions)	600.00	mm
Elastic deformation of concrete at the age of stress transfer	2.84	mm
Creep strain ε_{cc}=stress x ø/E_t	6.36E-04	
Therefore creep deformation of concrete	4.26	mm
where ø = creep coefficient	1.50	
E_t = Modulus of elasticity of concrete at the age of t = E_c		

Design as class 3 member with 0.1mm crack width at ultimate loading

For grade 50 concrete for limiting the crack width to 0.1mm		
Design flexural stress for class 3 member f_r =	4.80	N/mm^2

Design compressive stress at extreme fibre should not exceed $0.5f_{ci}$

where f_{ci} is the concrete strength at transfer

$0.5\ f_{ci} =$	25.00	N/mm^2
Concrete stresses due to effective pre-stresses = < $\lvert 0.5f_{ci} \rvert$	-13.47	N/mm^2
Concrete stresses due to bending: assume compression " - "		
Concrete compression at compression zone	-6.38	N/mm^2
Therefore maximum concrete compression = < $\lvert 0.5f_{ci} \rvert$	-19.85	N/mm^2
Maximum concrete tension for class 3 member f_r =	4.80	N/mm^2
Strain at extreme fibre at tension zone =	1.51E-04	

Calculation of steel stresses and moment about neutral axis :

Area of pre-stressing strand =	38.48	mm^2
Strands stresses due to effective pre-stresses =	1299.22	N/mm^2
Strands stresses-1 due to bending at tension zone=	-5.09	N/mm^2
Moment about neutral axis	-1.20	kNm
Strands stresses-2 due to bending at tension zone=	23.56	N/mm^2
Moment about neutral axis	7.76	kNm
Normal steel stresses due to bending =	14.80	N/mm^2
Area of normal steel =	56.55	mm^2
Moment about neutral axis	0.03	kNm
Total moment about neutral axis due to steel tensions	**6.60**	kNm
Concrete compression $C_c = 0.85f_c'A_a$	351.34	kN
Centroid distance	19.00	mm
Moment about neutral axis due to concrete compression	**6.68**	kNm

Trial and error to find the value of "C" so that above two figures are almost equal

Zero tension moment $M_0 = P\, I_g/(A_g\, y_t)$	**6.66**	kNm
Cracking moment $M_{cr} = P\, I_g/(A_g\, y_t)+f_r\, I_g/y_t$	**9.03**	kNm
Service moment of the pole $=< M_{cr}$	**8.93**	kNm

Therefore pole design is satisfactory for given service requirements

5.6m high Pre-stressed Pole

Analysis of critical section of the pole (at ground support position)

r_1 (Radios of outer circle) =	75	mm
r_2 (Radios of inner circle) =	40	mm
Thickness of pole =	35.00	mm
Gross concrete area at base A_g	12,644.91	mm^2
Assume C= Neutral axis depth by trial and error	70.00	mm
β_1= Parameter to calculate rectangular concrete compressive stress block	0.69	
$\beta_1 C$ =	48.30	mm
f'_c = Concrete compressive strength	50.00	N/mm^2
$f_r=0.62\sqrt{f'_c}$ Modulus of rupture of concrete	4.38	N/mm^2
y_t = Distance from centroid axis to extreme tensile fibre	80.00	mm
I_g = Gross moment of inertia of the section	2.28E-05	m^4
Finding area of Annulus		
Φ_1 =	3.01	radians
Φ_2 =	2.89	radians
Area A_1 =	8,086.29	mm^2
Area A_2 =	2,114.32	mm^2
Therefore Annulus area $A_a = A_1 - A_2$	**5,971.97**	mm^2
Finding centroid of annulus A_a		
Assume distance to the centroid from the centre x=	33.50	mm
Φ_3 =	2.22	radians
Φ_4 =	1.16	radians

A'_a =	**3,790.47**	mm^2
Find the "x" so that $A_a \approx 2A'_a$		
Cross sectional area of Φ7mm strand A_{psi} =	38.48	mm^2
Number of strands	4	No's
Modulus of elasticity of pre-stressing steel E_s =	205.00	kN/mm^2
Modulus of elasticity of normal steel E_y =	200.00	kN/mm^2
Modulus of elasticity of concrete E_c =	31.75	kN/mm^2
f_{py} = Specified yield stress of pre-stressing steel	1620.00	N/mm^2
Total pre-stressing force per strand = F_{py}	62.34	kN
Minimum breaking load F_{pu} =	62.50	kN
Therefore permissible pre-stressing force per strand = lesser of $0.80f_{pu}$ and $0.94f_{py}$ (assume 10% loss due to relaxation)=	50.00	kN
Assessment of transmission length: $l_t = K_t\Phi/\sqrt{f_{ci}}$	593.97	mm
where K_t is a coefficient for tendons =	600.00	
Average diameter of the section	125.00	mm
Average cross sectional area of concrete	9817.48	mm^2
Exposed perimeter of the section	392.70	mm
Effective section thickness of concrete(under immersed conditions)	600.00	mm
Elastic deformation of concrete at the age of stress transfer	3.59	mm
Creep strain ε_{cc}=stress x ø/E_t	9.62E-04	
Therefore creep deformation of concrete	5.39	mm
where ø = creep coefficient	1.50	
E_t = Modulus of elasticity of concrete at the age of t = E_c		

Design as class 3 member with 0.1mm crack width at ultimate loading

For grade 50 concrete for limiting the crack width to 0.1mm

Design flexural stress for class 3 member f_r =	4.80	N/mm^2

Design compressive stress at extreme fibre should not exceed $0.5f_{ci}$

where f_{ci} is the concrete strength at transfer

$0.5\ f_{ci}$ =	25.00	N/mm^2
Concrete stresses due to effective pre-stresses = < $\lvert 0.5f_{ci} \rvert$	-15.82	N/mm^2
Concrete stresses due to bending: assume compression " - "		
Concrete compression at compression zone	-4.20	N/mm^2
Therefore maximum concrete compression = < $\lvert 0.5f_{ci} \rvert$	-20.02	N/mm^2
Maximum concrete tension for class 3 member f_r =	4.80	N/mm^2
Strain at extreme fibre at tension zone =	1.51E-04	

Calculation of steel stresses and moment about neutral axis :

Area of pre-stressing strand =	38.48	mm^2
Strands stresses due to effective pre-stresses =	1299.22	N/mm^2
Strands stresses-1 due to bending at tension zone=	1.94	N/mm^2
Moment about neutral axis	0.50	kNm
Strands stresses-2 due to bending at tension zone=	24.21	N/mm^2
Moment about neutral axis	6.75	kNm
Normal steel stresses due to bending =	17.25	N/mm^2
Area of normal steel =	56.55	mm^2
Moment about neutral axis	0.04	kNm
Total moment about neutral axis due to steel tensions	**7.29**	kNm
Concrete compression $C_c = 0.85f_c'A_a$	253.81	kN
Centroid distance	28.50	mm
Moment about neutral axis due to concrete compression	**7.23**	kNm

Trial and error to find the value of "C" so that above two figures are almost equal

Zero tension moment $M_0 = P\, I_g/(A_g\, y_t)$	**4.52**	kNm
Cracking moment $M_{cr} = P\, I_g/(A_g\, y_t)+f_r\, I_g/y_t$	**5.89**	kNm
Service moment of the pole =< M_{cr}	**5.30**	kNm

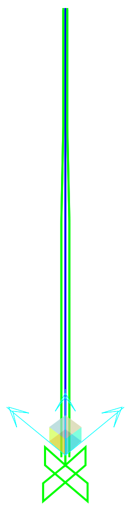
Therefore pole design is satisfactory for given service requirements

APPENDIX – F: SAP2000 Finite elements analytical results.

SAP2000 Analysis Report for 9m pre-stressed pole

Model geometry

This section provides model geometry information, including items such as joint coordinates, joint restraints, and element connectivity.



Finite element model

1.1. Joint coordinates

Table 1: Joint Coordinates, Part 1 of 2

Joint	CoordSys	CoordType	XorR mm	Y mm	Z mm	SpecialJt	GlobalX mm
1	GLOBAL	Cartesian	0.00	0.00	0.00	No	0.00
2	GLOBAL	Cartesian	0.00	0.00	9000.00	No	0.00

Table 1: Joint Coordinates, Part 2 of 2

Joint	GlobalY mm	GlobalZ mm	GUID
1	0.00	0.00	
2	0.00	9000.00	

1.2. Joint restraints

Table 2: Joint Restraint Assignments

Joint	U1	U2	U3	R1	R2	R3

Joint	U1	U2	U3	R1	R2	R3
1	Yes	Yes	Yes	Yes	Yes	Yes

1.3. Element connectivity

Table 3: Connectivity - Frame, Part 1 of 2

Frame	JointI	JointJ	IsCurved	Length mm	Centroid X mm	Centroid Y mm	Centroid Z mm
1	1	2	No	9000.00	0.00	0.00	4500.00

Table 3: Connectivity - Frame, Part 2 of 2

Frame	GUID
1	

Table 4: Frame Section Assignments, Part 1 of 2

Frame	SectionType	AutoSelect	AnalSect	DesignSect	MatProp
1	Nonprismatic	N.A.	Tapered	Tapered	Default

Table 4: Frame Section Assignments, Part 2 of 2

Frame	NPSectType	NPSectLen mm	NPSectRD
1	Default		

Table 5: Connectivity - Tendon

Tendon	JointI	JointJ	Length mm	GUID
2	1	2	9000.20	
3	1	2	9000.20	
4	1	2	9000.20	

Tendon	JointI	JointJ	Length mm	GUID
5	1	2	9000.20	

Table 6: Connectivity - Tendon

Tendon	JointI	JointJ	Length mm	GUID
2	1	2	9000.20	
3	1	2	9000.20	
4	1	2	9000.20	
5	1	2	9000.20	

2. Material properties

This section provides material property information for materials used in the model.

Table 7: Material Properties 02 - Basic Mechanical Properties

Material	UnitWeight N/mm3	UnitMass N-s2/mm4	E1 N/mm2	G12 N/mm2	U12	A1 1/C
A416Gr270	7.6973E-05	7.8490E-09	196500.60			1.1700E-05
A615Gr60	7.6973E-05	7.8490E-09	199947.98			1.1700E-05
A992Fy50	7.6973E-05	7.8490E-09	199947.98	76903.07	0.300000	1.1700E-05
Gr50	2.4000E-05	2.4473E-09	24855.58	10356.49	0.200000	9.9000E-06

Table 8: Material Properties 03a - Steel Data, Part 1 of 2

Material	Fy N/mm2	Fu N/mm2	EffFy N/mm2	EffFu N/mm2	SSCurveOpt	SSHysType	SHard	SMax
A992Fy50	344.74	448.16	379.21	492.98	Simple	Kinematic	0.015000	0.110000

Table 8: Material Properties 03a - Steel Data, Part 2 of 2

Material	SRup	FinalSlope
A992Fy50	0.170000	-0.100000

Table 9: Material Properties 03b - Concrete Data, Part 1 of 2

Material	Fc N/mm2	LtWtConc	SSCurveOpt	SSHysType	SFc	SCap	FinalSlope	FAngle Degrees
Gr50	50.00	No	Mander	Takeda	0.002219	0.005000	-0.100000	0.000

Table 9: Material Properties 03b - Concrete Data, Part 2 of 2

Material	DAngle Degrees	TimeType	TimeE	TimeCreep	TimeShrink	CreepType
Gr50	0.000	CEB-FIP 90	Yes	Yes	Yes	Full Integration

Table 10: Material Properties 03e - Rebar Data, Part 1 of 2

Material	Fy N/mm2	Fu N/mm2	EffFy N/mm2	EffFu N/mm2	SSCurveOpt	SSHysType	SHard	SCap
A615Gr60	413.69	620.53	455.05	682.58	Simple	Kinematic	0.010000	0.090000

Table 10: Material Properties 03e - Rebar Data, Part 2 of 2

Material	FinalSlope	UseCTDef
A615Gr60	-0.100000	No

Table 11: Material Properties 03f - Tendon Data

Material	Fy N/mm2	Fu N/mm2	SSCurveOpt	SSHysType	FinalSlope	TimeType	TimeRelax	RelaxType
A416Gr270	1689.91	1861.58	270 ksi	Kinematic	-0.100000	CEB-FIP 90	Yes	Full Integration

3. Section properties

This section provides section property information for objects used in the model.

3.1. Frames

Table 12: Frame Section Properties 01 - General, Part 1 of 6

SectionName	Material	Shape	t3 mm	t2 mm	tf mm	tw mm
140Dia	Gr50	SD Section				
260Dia	Gr50	SD Section				
Tapered		Nonprismatic				

Table 12: Frame Section Properties 01 - General, Part 2 of 6

SectionName	t2b mm	tfb mm	Area mm2	TorsConst mm4	I33 mm4	I22 mm4	I23 mm4
140Dia			12566.37	35970477.10	18221237.39	18221237.39	0.00
260Dia			27646.02	341054347.	172787595.9	172787595.9	0.00
Tapered							

Table 12: Frame Section Properties 01 - General, Part 3 of 6

SectionName	AS2 mm2	AS3 mm2	S33 mm3	S22 mm3	Z33 mm3	Z22 mm3	R33 mm
140Dia	9864.01	9864.01	260303.39	260303.39	417285.43	417285.43	38.079
260Dia	19334.73	19334.73	1329135.35	1329135.35	1938528.53	1938528.53	79.057
Tapered							

Table 12: Frame Section Properties 01 - General, Part 4 of 6

SectionName	R22 mm	ConcCol	ConcBeam	Color	TotalWt N	TotalMass N-s2/mm	FromFile
140Dia	38.079	No	No	Gray8Dark	0.00	0.000	No
260Dia	79.057	No	No	Magenta	0.00	0.000	No
Tapered				Blue			

Table 12: Frame Section Properties 01 - General, Part 5 of 6

SectionName	AMod	A2Mod	A3Mod	JMod	I2Mod	I3Mod	MMod

140Dia	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
260Dia	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
Tapered							

Table 12: Frame Section Properties 01 - General, Part 6 of 6

SectionName	WMod	GUID	Notes
140Dia	1.000000		Added 5/5/2015 4:24:24 PM
260Dia	1.000000		Added 5/5/2015 3:29:11 PM
Tapered			Added 5/5/2015 4:27:35 PM

Table 13: Frame Section Properties 05 - Nonprismatic, Part 1 of 2

SectionName	NumSegments	SegmentNum	StartSect	EndSect	LengthType	AbsLength mm
Tapered	1	1	260Dia	140Dia	Absolute	9000000.00

Table 13: Frame Section Properties 05 - Nonprismatic, Part 2 of 2

SectionName	VarLength	EI33Var	EI22Var
Tapered		Linear	Linear

3.2. Tendons

Table 14: Tendon Section Definitions, Part 1 of 4

TendonSect	ModelOpt	PreType	Material	Specify	Diameter mm	Area mm2	TorsConst mm4	I mm4
PC7	Elements	Prestress	A416Gr270	Diameter	7.000	38.48	235.72	117.86

Table 14: Tendon Section Definitions, Part 2 of 4

TendonSect	AS mm2	Color	TotalWt N	TotalMass N-s2/mm	AMod	A2Mod	A3Mod	JMod
PC7	34.64	Magenta	106.64	0.011	1.000000	1.000000	1.000000	1.000000

Table 14: Tendon Section Definitions, Part 3 of 4

TendonSect	I2Mod	I3Mod	MMod	WMod	GUID
PC7	1.000000	1.000000	1.000000	1.000000	

Table 14: Tendon Section Definitions, Part 4 of 4

TendonSect	Notes
PC7	

4. Load patterns

This section provides loading information as applied to the model.

4.1. Definitions

Table 15: Load Pattern Definitions

LoadPat	DesignType	SelfWtMult	AutoLoad	GUID	Notes
DEAD	DEAD	1.000000			
LIVE	LIVE	0.000000			
PRESTRESS	PRESTRESS	0.000000			

5. Load cases

This section provides load case information.

5.1. Definitions

Table 16: Load Case Definitions, Part 1 of 3

Case	Type	InitialCond	ModalCase	BaseCase	DesTypeOpt	DesignType	DesActOpt
DEAD	LinStatic	Zero			Prog Det	DEAD	Prog Det
MODAL	LinModal	Zero			Prog Det	OTHER	Prog Det
LIVE	LinStatic	Zero			Prog Det	LIVE	Prog Det
PRESTRESS	LinStatic	Zero			Prog Det	PRESTRESS	Prog Det

Table 16: Load Case Definitions, Part 2 of 3

Case	DesignAct	AutoType	RunCase	CaseStatus	GUID
DEAD	Non-Composite	None	Yes	Finished	
MODAL	Other	None	Yes	Finished	
LIVE	Short-Term Composite	None	Yes	Finished	
PRESTRESS	Long-Term Composite	None	Yes	Finished	

Table 16: Load Case Definitions, Part 3 of 3

Case	Notes
DEAD	
MODAL	
LIVE	
PRESTRESS	

5.2. Static case load assignments

Table 17: Case - Static 1 - Load Assignments

Case	LoadType	LoadName	LoadSF
DEAD	Load pattern	DEAD	1.000000
LIVE	Load pattern	LIVE	1.000000

Case	LoadType	LoadName	LoadSF
PRESTRESS	Load pattern	PRESTRESS	1.000000

5.3. Response spectrum case load assignments

Table 18: Function - Response Spectrum - User

Name	Period Sec	Accel	FuncDamp
UNIFRS	0.000000	1.000000	0.050000
UNIFRS	1.000000	1.000000	

6. Load combinations

This section provides load combination information.

Table 19: Combination Definitions, Part 1 of 3

ComboName	ComboType	AutoDesign	CaseType	CaseName	ScaleFactor	SteelDesign
COMB(ULS)	Linear Add	No	Linear Static	DEAD	1.400000	None
COMB(ULS)			Linear Static	LIVE	2.500000	
COMB(ULS)			Linear Static	PRESTRESS	1.000000	
COMB(SLS)	Linear Add	No	Linear Static	DEAD	1.000000	None
COMB(SLS)			Linear Static	LIVE	1.000000	
COMB(SLS)			Linear Static	PRESTRESS	1.000000	
ENV(ULS)	Envelope	No	Response Combo	COMB(ULS)	1.000000	None
ENV(SLS)	Envelope	No	Response Combo	COMB(SLS)	1.000000	None

Table 19: Combination Definitions, Part 2 of 3

ComboName	CaseName	ConcDesign	AlumDesign	ColdDesign
COMB(ULS)	DEAD	None	None	None
COMB(ULS)	LIVE			
COMB(ULS)	PRESTRESS			
COMB(SLS)	DEAD	None	None	None

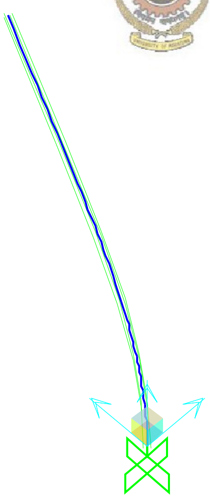
ComboName	CaseName	ConcDesign	AlumDesign	ColdDesign
COMB(SLS)	LIVE			
COMB(SLS)	PRESTRESS			
ENV(ULS)	COMB(ULS)	Strength	None	None
ENV(SLS)	COMB(SLS)	None	None	None

Table 19: Combination Definitions, Part 3 of 3

ComboName	CaseName	GUID	Notes
COMB(ULS)	DEAD		
COMB(ULS)	LIVE		
COMB(ULS)	PRESTRESS		
COMB(SLS)	DEAD		
COMB(SLS)	LIVE		
COMB(SLS)	PRESTRESS		
ENV(ULS)	COMB(ULS)		
ENV(SLS)	COMB(SLS)		

7. Structure results

This section provides structure results, including items such as structural periods and base reactions.



Finite element model deformed shape

7.1. Mass summary

Table 20: Assembled Joint Masses

Joint	MassSource	U1	U2	U3	R1	R2	R3

		N-s2/mm	N-s2/mm	N-s2/mm	N-mm-s2	N-mm-s2	N-mm-s2
1	MSSSRC1	0.049	0.049	0.049	0.00	0.00	0.00
2	MSSSRC1	0.025	0.025	0.025	0.00	0.00	0.00
~1	MSSSRC1	0.092	0.092	0.092	0.00	0.00	0.00
~2	MSSSRC1	0.083	0.083	0.083	0.00	0.00	0.00
~3	MSSSRC1	0.074	0.074	0.074	0.00	0.00	0.00
~4	MSSSRC1	0.065	0.065	0.065	0.00	0.00	0.00
~5	MSSSRC1	0.055	0.055	0.055	0.00	0.00	0.00
~6	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00
~7	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~8	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~9	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~10	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~11	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~12	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00
~13	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00
~14	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~15	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~16	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~17	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~18	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~19	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00
~20	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00
~21	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~22	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~23	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~24	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~25	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~26	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00
~27	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00
~28	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~29	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~30	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~31	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~32	MSSSRC1	4.531E-04	4.531E-04	4.531E-04	0.00	0.00	0.00
~33	MSSSRC1	2.266E-04	2.266E-04	2.266E-04	0.00	0.00	0.00

7.2. Modal results

Table 21: Modal Participating Mass Ratios, Part 1 of 3

OutputCase	StepType	StepNum	Period Sec	UX	UY	UZ	SumUX	SumUY
MODAL	Mode	1.000000	0.486022	0.59	3.903E-06	0.00	0.59	3.903E-06
MODAL	Mode	2.000000	0.486022	3.903E-06	0.59	0.00	0.59	0.59
MODAL	Mode	3.000000	0.102229	1.988E-03	0.22	0.00	0.59	0.81
MODAL	Mode	4.000000	0.102229	0.22	1.988E-03	0.00	0.81	0.81

OutputCase	StepType	StepNum	Period Sec	UX	UY	UZ	SumUX	SumUY
MODAL	Mode	5.000000	0.041583	2.964E-06	9.219E-02	0.00	0.81	0.90

Table 21: Modal Participating Mass Ratios, Part 2 of 3

OutputCase	StepType	StepNum	SumUZ	RX	RY	RZ	SumRX	SumRY
MODAL	Mode	1.000000	0.00	4.046E-06	0.61	0.00	4.046E-06	0.61
MODAL	Mode	2.000000	0.00	0.61	4.046E-06	0.00	0.61	0.61
MODAL	Mode	3.000000	0.00	0.12	1.086E-03	0.00	0.73	0.61
MODAL	Mode	4.000000	0.00	1.086E-03	0.12	0.00	0.73	0.73
MODAL	Mode	5.000000	0.00	0.11	3.467E-06	0.00	0.84	0.73

Table 21: Modal Participating Mass Ratios, Part 3 of 3

OutputCase	StepType	StepNum	SumRZ
MODAL	Mode	1.000000	0.00
MODAL	Mode	2.000000	0.00
MODAL	Mode	3.000000	0.00
MODAL	Mode	4.000000	0.00
MODAL	Mode	5.000000	0.00

7.3. Base reactions

Table 22: Base Reactions, Part 1 of 3

OutputCase	CaseType	StepType	GlobalFX N	GlobalFY N	GlobalFZ N	GlobalMX N-mm	GlobalMY N-mm	GlobalMZ N-mm
ENV(ULS)	Combination	Max	-3529.20	4.401E-14	197146.21	-2.344E-10	-29423259.9	3.763E-11
ENV(ULS)	Combination	Min	-3529.20	4.401E-14	197146.21	-2.344E-10	-29423259.9	3.763E-11
ENV(SLS)	Combination	Max	-1411.68	4.379E-14	195439.64	-2.332E-10	-11769304.0	1.505E-11

OutputCase	CaseType	StepType	GlobalFX N	GlobalFY N	GlobalFZ N	GlobalMX N-mm	GlobalMY N-mm	GlobalMZ N-mm
ENV(SLS)	Combination	Min	-1411.68	4.379E-14	195439.64	-2.332E-10	-11769304.0	1.505E-11

Table 22: Base Reactions, Part 2 of 3

OutputCase	StepType	GlobalX mm	GlobalY mm	GlobalZ mm	XCentroidFX mm	YCentroidFX mm	ZCentroidFX mm	XCentroidFY mm
ENV(ULS)	Max	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENV(ULS)	Min	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENV(SLS)	Max	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENV(SLS)	Min	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 22: Base Reactions, Part 3 of 3

OutputCase	StepType	YCentroidFY mm	ZCentroidFY mm	XCentroidFZ mm	YCentroidFZ mm	ZCentroidFZ mm
ENV(ULS)	Max	0.00	0.00	0.00	0.00	0.00
ENV(ULS)	Min	0.00	0.00	0.00	0.00	0.00
ENV(SLS)	Max	0.00	0.00	0.00	0.00	0.00
ENV(SLS)	Min	0.00	0.00	0.00	0.00	0.00

Table 23: Joint Displacements, Part 2 of 2

Joint	OutputCase	StepType	R3 Radians
1	ENV(ULS)	Max	0.000000
1	ENV(ULS)	Min	0.000000
1	ENV(SLS)	Max	0.000000
1	ENV(SLS)	Min	0.000000
2	ENV(ULS)	Max	2.687E-19
2	ENV(ULS)	Min	2.687E-19
2	ENV(SLS)	Max	1.075E-19
2	ENV(SLS)	Min	1.075E-19
~1	ENV(ULS)	Max	-1.731E-20
~1	ENV(ULS)	Min	-1.731E-20
~1	ENV(SLS)	Max	0.000000
~1	ENV(SLS)	Min	0.000000
~2	ENV(ULS)	Max	-1.029E-19
~2	ENV(ULS)	Min	-1.029E-19
~2	ENV(SLS)	Max	-4.115E-20

Joint	OutputCase	StepType	R3 Radians
~2	ENV(SLS)	Min	-4.115E-20
~3	ENV(ULS)	Max	8.153E-20
~3	ENV(ULS)	Min	8.153E-20
~3	ENV(SLS)	Max	3.261E-20
~3	ENV(SLS)	Min	3.261E-20
~4	ENV(ULS)	Max	3.242E-19
~4	ENV(ULS)	Min	3.242E-19
~4	ENV(SLS)	Max	1.297E-19
~4	ENV(SLS)	Min	1.297E-19
~5	ENV(ULS)	Max	2.440E-19
~5	ENV(ULS)	Min	2.440E-19
~5	ENV(SLS)	Max	9.761E-20
~5	ENV(SLS)	Min	9.761E-20
~6	ENV(ULS)	Max	-2.113E-12
~6	ENV(ULS)	Min	-2.113E-12
~6	ENV(SLS)	Max	-8.602E-13
~6	ENV(SLS)	Min	-8.602E-13
~7	ENV(ULS)	Max	-2.113E-12
~7	ENV(ULS)	Min	-2.113E-12
~7	ENV(SLS)	Max	-8.602E-13
~7	ENV(SLS)	Min	-8.602E-13
~8	ENV(ULS)	Max	-2.113E-12
~8	ENV(ULS)	Min	-2.113E-12
~8	ENV(SLS)	Max	-8.602E-13
~8	ENV(SLS)	Min	-8.602E-13
~9	ENV(ULS)	Max	-2.113E-12
~9	ENV(ULS)	Min	-2.113E-12
~9	ENV(SLS)	Max	-8.602E-13
~9	ENV(SLS)	Min	-8.602E-13
~10	ENV(ULS)	Max	-2.113E-12
~10	ENV(ULS)	Min	-2.113E-12
~10	ENV(SLS)	Max	-8.602E-13
~10	ENV(SLS)	Min	-8.602E-13
~11	ENV(ULS)	Max	-2.113E-12
~11	ENV(ULS)	Min	-2.113E-12
~11	ENV(SLS)	Max	-8.602E-13
~11	ENV(SLS)	Min	-8.602E-13
~12	ENV(ULS)	Max	-2.113E-12
~12	ENV(ULS)	Min	-2.113E-12
~12	ENV(SLS)	Max	-8.602E-13
~12	ENV(SLS)	Min	-8.602E-13
~13	ENV(ULS)	Max	-0.000173
~13	ENV(ULS)	Min	-0.000173
~13	ENV(SLS)	Max	-0.000069
~13	ENV(SLS)	Min	-0.000069
~14	ENV(ULS)	Max	-0.000131
~14	ENV(ULS)	Min	-0.000131
~14	ENV(SLS)	Max	-0.000052
~14	ENV(SLS)	Min	-0.000052
~15	ENV(ULS)	Max	-0.000059
~15	ENV(ULS)	Min	-0.000059
~15	ENV(SLS)	Max	-0.000024
~15	ENV(SLS)	Min	-0.000024
~16	ENV(ULS)	Max	2.621E-08

Joint	OutputCase	StepType	R3 Radians
~16	ENV(ULS)	Min	2.621E-08
~16	ENV(SLS)	Max	1.048E-08
~16	ENV(SLS)	Min	1.048E-08
~17	ENV(ULS)	Max	0.000055
~17	ENV(ULS)	Min	0.000055
~17	ENV(SLS)	Max	0.000022
~17	ENV(SLS)	Min	0.000022
~18	ENV(ULS)	Max	0.000096
~18	ENV(ULS)	Min	0.000096
~18	ENV(SLS)	Max	0.000038
~18	ENV(SLS)	Min	0.000038
~19	ENV(ULS)	Max	0.000111
~19	ENV(ULS)	Min	0.000111
~19	ENV(SLS)	Max	0.000044
~19	ENV(SLS)	Min	0.000044
~20	ENV(ULS)	Max	2.756E-16
~20	ENV(ULS)	Min	2.756E-16
~20	ENV(SLS)	Max	1.102E-16
~20	ENV(SLS)	Min	1.102E-16
~21	ENV(ULS)	Max	2.756E-16
~21	ENV(ULS)	Min	2.756E-16
~21	ENV(SLS)	Max	1.102E-16
~21	ENV(SLS)	Min	1.102E-16
~22	ENV(ULS)	Max	2.756E-16
~22	ENV(ULS)	Min	2.756E-16
~22	ENV(SLS)	Max	1.102E-16
~22	ENV(SLS)	Min	1.102E-16
~23	ENV(ULS)	Max	2.756E-16
~23	ENV(ULS)	Min	2.756E-16
~23	ENV(SLS)	Max	1.102E-16
~23	ENV(SLS)	Min	1.102E-16
~24	ENV(ULS)	Max	2.756E-16
~24	ENV(ULS)	Min	2.756E-16
~24	ENV(SLS)	Max	1.102E-16
~24	ENV(SLS)	Min	1.102E-16
~25	ENV(ULS)	Max	2.756E-16
~25	ENV(ULS)	Min	2.756E-16
~25	ENV(SLS)	Max	1.102E-16
~25	ENV(SLS)	Min	1.102E-16
~26	ENV(ULS)	Max	2.756E-16
~26	ENV(ULS)	Min	2.756E-16
~26	ENV(SLS)	Max	1.102E-16
~26	ENV(SLS)	Min	1.102E-16
~27	ENV(ULS)	Max	-5.407907
~27	ENV(ULS)	Min	-5.407907
~27	ENV(SLS)	Max	-2.163163
~27	ENV(SLS)	Min	-2.163163
~28	ENV(ULS)	Max	-5.407949
~28	ENV(ULS)	Min	-5.407949
~28	ENV(SLS)	Max	-2.163180
~28	ENV(SLS)	Min	-2.163180
~29	ENV(ULS)	Max	-5.408021
~29	ENV(ULS)	Min	-5.408021
~29	ENV(SLS)	Max	-2.163208

Joint	OutputCase	StepType	R3 Radians
~29	ENV(SLS)	Min	-2.163208
~30	ENV(ULS)	Max	-5.408080
~30	ENV(ULS)	Min	-5.408080
~30	ENV(SLS)	Max	-2.163232
~30	ENV(SLS)	Min	-2.163232
~31	ENV(ULS)	Max	-5.408135
~31	ENV(ULS)	Min	-5.408135
~31	ENV(SLS)	Max	-2.163254
~31	ENV(SLS)	Min	-2.163254
~32	ENV(ULS)	Max	-5.408176
~32	ENV(ULS)	Min	-5.408176
~32	ENV(SLS)	Max	-2.163270
~32	ENV(SLS)	Min	-2.163270
~33	ENV(ULS)	Max	-5.408191
~33	ENV(ULS)	Min	-5.408191
~33	ENV(SLS)	Max	-2.163276
~33	ENV(SLS)	Min	-2.163276

Table 24: Joint Reactions, Part 1 of 2

Joint	OutputCase	CaseType	StepType	F1 N	F2 N	F3 N	M1 N-mm	M2 N-mm
1	ENV(ULS)	Combination	Max	-3600.00	5.627E-14	6229.41	1.770E-09	-30600000.0
1	ENV(ULS)	Combination	Min	-3600.00	5.627E-14	6229.41	1.770E-09	-30600000.0
1	ENV(SLS)	Combination	Max	-1440.00	5.605E-14	4449.58	1.772E-09	-12240000.0
1	ENV(SLS)	Combination	Min	-1440.00	5.605E-14	4449.58	1.772E-09	-12240000.0

Table 24: Joint Reactions, Part 2 of 2

Joint	OutputCase	StepType	M3 N-mm
1	ENV(ULS)	Max	5.936E-11
1	ENV(ULS)	Min	5.936E-11
1	ENV(SLS)	Max	2.374E-11

Joint	OutputCase	StepType	M3 N-mm
1	ENV(SLS)	Min	2.374E-11

10. Material take-off

This section provides a material take-off.

Table 26: Material List 2 - By Section Property

Section	ObjectType	NumPieces	TotalLength mm	TotalWeight N
Tapered	Frame	1	9000.00	4342.94

APENDIX – G: Manufacturer's cost analysis for current pre-casting concrete poles

Rate analysis for the casting of conventional telecom post submitted to the Sri Lankan Telecom by one of the sub-contracting company on October 2012 are summarized below under different height categories of poles.

5.6m High Pole:

Description	Unit	Qty	Rate(Rs.)	Amount(Rs.)
Cement	bags	0.410	855.00	312.99
Metal(¾")	cube	0.014	7000.00	99.40
Sand	cube	0.007	10,000.00	70.00
¾" G.I Pipe	ft	0.50	101.60	50.80
Nuts & Bolts	each	1.00	80.00	80.00
Welding Rods	pkts	0.02	225.00	4.50
Mould Oil	ltrs	0.25	175.00	43.75
Cost of Mould	each	0.0005	75,000.00	37.50
Electricity and Water	Item	1.00	32.00	32.00
Welder	day	0.021	875.00	18.38
Curing 28Days	Item	1.00	32.00	32.00
12mm Tor Steel	ft	0.00	37.60	0.00
10mm Tor Steel	ft	109.00	25.91	2,521.60
6mm Mild Steel	kg	2.64	140.00	330.00
Binding Wire	kg	0.35	140.00	43.75
Bar Bender	hrs	1.50	196.87	295.31
Concreter	hrs	0.60	284.37	170.62
			Basic Cost per pole	**4,142.59**

6.7m High Pole:

Description	Unit	Qty	Rate(Rs.)	Amount(Rs.)
Cement	bags	1.15	855.00	877.90
Metal(¾")	cube	0.04	7000.00	280.00
Sand	cube	0.02	10,000.00	200.00
¾" G.I Pipe	ft	0.50	101.60	50.80
Nuts & Bolts	each	1.00	80.00	80.00
Welding Rods	pkts	0.02	225.00	4.50
Mould Oil	ltrs	0.30	175.00	52.50
Cost of Mould	each	0.0005	81,250.00	40.63
Electricity and Water	Item	1.00	32.00	32.00
Welder	day	0.021	875.00	18.38
Curing 28Days	Item	1.00	32.00	32.00
12mm Tor Steel	ft	88.15	37.60	2,959.32
10mm Tor Steel	ft	49.00	25.91	1,133.56
6mm Mild Steel	kg	5.20	140.00	650.00
Binding Wire	kg	0.40	140.00	50.00
Bar Bender	hrs	2.00	196.87	393.74
Concreter	hrs	0.80	284.37	227.50
			Basic Cost per pole	**7,082.82**

7.5m High Pole:

Description	Unit	Qty	Rate(Rs.)	Amount(Rs.)
Cement	bags	1.38	855.00	1,053.48
Metal(¾")	cube	0.05	7000.00	343.00
Sand	cube	0.025	10,000.00	250.00
¾" G.I Pipe	ft	0.50	101.60	50.80
Nuts & Bolts	each	1.00	80.00	80.00
Welding Rods	pkts	0.02	225.00	4.50
Mould Oil	ltrs	0.04	175.00	70.00
Cost of Mould	each	0.0005	81,250.00	40.63
Electricity and Water	Item	1.00	32.00	32.00
Welder	day	0.021	875.00	18.38
Curing 28Days	Item	1.00	32.00	32.00
12mm Tor Steel	ft	98.70	37.60	3,313.36
10mm Tor Steel	ft	58.00	25.91	1,341.54
6mm Mild Steel	kg	5.90	140.00	737.50
Binding Wire	kg	0.66	140.00	82.50
Bar Bender	hrs	2.50	196.87	492.18
Concreter	hrs	1.00	284.37	284.37
			Basic Cost per pole	**8,226.23**

8.0m High Pole:

Description	Unit	Qty	Rate(Rs.)	Amount(Rs.)
Cement	bags	1.51	855.00	1,152.72
Metal(¾")	cube	0.05	7000.00	378.00
Sand	cube	0.03	10,000.00	280.00
¾" G.I Pipe	ft	0.50	101.60	50.80
Nuts & Bolts	each	1.00	80.00	80.00
Welding Rods	pkts	0.02	225.00	4.50
Mould Oil	ltrs	0.40	175.00	70.00
Cost of Mould	each	0.0005	93,750.00	46.88
Electricity and Water	Item	1.00	32.00	32.00
Welder	day	0.021	875.00	18.38
Curing 28Days	Item	1.00	32.00	32.00
12mm Tor Steel	ft	105.00	37.60	3,525.00
10mm Tor Steel	ft	58.00	25.91	1,341.54
6mm Mild Steel	kg	6.45	140.00	806.25
Binding Wire	kg	0.75	140.00	93.75
Bar Bender	hrs	2.50	196.87	492.18
Concreter	hrs	1.00	284.37	284.37
			Basic Cost per pole	**8,688.59**

9.0m High Pole:

Description	Unit	Qty	Rate(Rs.)	Amount(Rs.)
Cement	bags	1.84	855.00	1,404.64
Metal(¾”)	cube	0.06	7000.00	448.00
Sand	cube	0.034	10,000.00	340.00
¾” G.I Pipe	ft	0.50	101.60	50.80
Nuts & Bolts	each	1.00	80.00	80.00
Welding Rods	pkts	0.03	225.00	6.75
Mould Oil	ltrs	0.50	175.00	87.50
Cost of Mould	each	0.0005	106,250.00	53.13
Electricity and Water	Item	1.00	32.00	32.00
Welder	day	0.021	875.00	18.38
Curing 28Days	Item	1.00	32.00	32.00
12mm Tor Steel	ft	118.08	37.60	3,964.11
10mm Tor Steel	ft	71.76	25.91	1,660.09
6mm Mild Steel	kg	7.34	140.00	917.50
Binding Wire	kg	0.40	140.00	50.00
Bar Bender	hrs	3.00	196.87	590.61
Concreter	hrs	1.15	284.37	327.03
			Basic Cost per pole	**10,062.53**