DEVELOPMENT OF ADVANCED DESIGN CRITERIA FOR STRONGER LATTICE TOWERS

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Degree of Doctor of Philosophy

Department of Civil Engineering

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Thesis submitted in partial fulfillment of the requirements for the degree Doctor of Philosophy of Engineering in Civil Engineering

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Declaration of the candidate & Supervisors

I declare that this is my own work and this thesis does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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The above candidate has carried out research for the PhD thesis under our supervision.

Name of the supervisors: 1. Prof. M.T.R. Jayasinghe

2. Dr. C.S. Lewangamage

Signature of the supervisors: 1. Date:

2.

Abstract

The key lateral loads acting on a freestanding telecommunication/broadcasting tower are due to wind effects though occasionally seismic forces also can act. Though earthquake design guidelines are well covered directly or indirectly in many tower designs standards, telecommunication/broadcasting towers in Sri Lanka and in Indian subcontinent are not specifically designed for earthquake induced forces. Therefore, a detailed study was undertaken to determine the probable structural performances that can be expected from commonly adopted Four leg and Three leg self supporting lattice towers. For this study, the earthquake levels that can be possibly expected in Sri Lanka and in the South Asian region have been considered.

A key parameter that can directly affect the seismic performance is subsoil conditions. Hence, a probable range of subsoil conditions have been considered in this study as certain subsoil conditions could amplify seismic waves under certain conditions. For a range of tower heights that are generally used, Response Spectrum analysis techniques have been used to assess the probable performance of lattice towers. An assessment was also made with equivalent static method to determine the applicability of it over a wide range of conditions.

One of the key observations has been that the earthquake induced stresses on key members of lattice towers could be of lower magnitude than due to effects of wind. The parameters that could affect the seismic behaviour of a lattice tower are the subsoil stratum and the natural period of vibration of the lattice tower.

The main finding of seismic analysis of lattice towers is that for the likely seismic hazard levels in Sri Lanka, key elements of towers will be subjected to much lower stress levels than induced by the winds of design magnitudes. However, if more severe earthquakes that could occur in the South Asian region are considered, there is a possibility for the earthquake induced forces to reach structurally significant levels. However, still such forces are less than the wind induced forces.

For both earthquake and wind induced lateral loads, one of the key elements that needs careful attention is the connection of the tower to the foundation through baseplates. Baseplates are crucial elements related to structural stability of self supporting lattice towers. It acts as the interface between the tower body and the foundation.

Due to structural characteristics of towers, uplift forces induced on baseplates govern the design of the baseplates. This is a rare case to occur on baseplate of buildings and other conventional structures. Hence, guidelines published in design codes and standard text books for design of baseplates are not really applicable for self-standing towers. This is a gray area that needs attention. Hence, an extensive study was carried out in this regard using Yield line theory to develop a design guideline. The developed theory was verified through a detailed experimental investigation and a finite element modeling using computer aided non liner modeling techniques. The formulae developed were modified for industrial applications considering all practical deviations which could not be addressed under fundamental theory using extensive parametric analyses carried out with the Finite Element models. The findings have been presented as design guidelines in the form of equations that can be used by structural design engineers undertaking free standing tower designs.

Keywords: Telecommunication towers, earthquakes, response spectrum analysis, baseplates, finite element analysis, design guidelines

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Finally, there is one outstanding person behind my whole successful journey even though he is not physically present now. It is his vision and passion that drove me to this peak. I dedicate this thesis to him, to my dear farther.

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List of Abbreviations

Abbreviation	Meaning
4G	Fourth generation of broadband cellular network technology
5G	Fifth generation of broadband cellular network technology
ANSI	American National Standards Institute
BNBC	Bangladesh National Building Code
BS	British Standards
GDP	Gross domestic product
EHV	Extra High Voltage
GSHAP	Global Seismic Hazard Assessment Programme
GSM	Global System for Mobile Communications
GSMB	Geology Survey and Mining Bureau of Sri Lanka
IS	Indian Standards
LTE	Long-Term Evolution
NBC	National Building Code of Nepal
MMI VIII	Modified Mercalli intensity scale 8
PGA	Peak Ground Acceleration
PSHA	Probabilistic Seismic Hazard Assessment
SODF	Single Degree of Freedom
SPT	Standard Penetration Test
SRSS	Square Root Sum of Squares
TRCSL	Telecommunication Regulatory Commission of Sri Lanka
TIA	Telecommunications Industry Association

Chapter 1- Introduction

1.1 Background

Telecommunication is one of the fastest growing industries during last few decades in the world. The worldwide telecommunication services revenue is reported as US$2.2 trillion in 2015 and as per predictions, it will grow up to US$2.4 trillion in 2019. In Sri Lankan context, the direct contribution of Post and Telecommunication services to GDP is about US$ 1.6 billion (about 2% of GDP as Annual report of Central Bank of Sri Lanka 2015). In addition to this, the telecommunication also plays a key role in product and service sectors too as voice and data connectivity is essential in modern world. As per Telecommunication Regulatory Commission of Sri Lanka (TRCSL) statistics, total phone subscribers are almost 30 million (2.5 million Fixed line subscribers and 27.2 million Mobile subscribers) in Sri Lanka. Accordingly, the Cellular Mobile Subscription per 100 inhabitants is high as 123.7. Also, Broadband penetration is high as 27% of population and Sri Lanka is the first South Asian country that successfully carried out emerging Pre 5G field trial tests.

Since most of these emerging technologies are wireless technologies (such as GSM, 4G LTE, etc.), telecommunication towers play a key role in the telecommunication network development. However, even in pre era to the mobile technology (the mobile phone technology was introduced to Sri Lanka in 1991) telecommunication towers had played a vital part by facilitating Point to point Radio connectivity between regional switching stations especially in remote parts of the country. Hence, telecommunication towers of the country too have a great history as incumbent fixed line telecommunication operator, Sri Lanka Telecom PLC, has a history of over hundred years. There are some towers which are more than 35 years old and those are still in service without any structural issues. Presently, total count of telecommunication towers is few thousands.

In addition to telecommunication towers, broadcasting towers can also be seen which are used by the radio and television broadcasters. The structural characteristics of these towers are very much similar to telecommunication towers and the number of broadcasting towers is comparatively less compared to telecommunication towers in a country.

Usually, telecommunication/broadcasting towers are slender tall structures and various materials can be used for the construction. However, structural steel is the most common material used for the tower construction in world wide.

1.1.1 Seismic effects on towers

Traditionally, wind forces are the predominant forces that govern the structural design of telecommunication towers. Seismic forces may also be a concern for these types of structures but still seismic analysis and design in the context of telecommunication tower field is not generally practiced in Sri Lanka. Even in global context it has been paid lesser attention as many tower design codes do not address it specifically. However, ensuring structural integrity of telecommunication towers under all extreme events like earthquakes is quite important as communication play a vital role in operations like rescue missions, disaster management, etc. in such an event. During 2004 Tsunami, failure of telecommunication facilities was experienced and it seriously affected the disaster management process. Similar situations have been reported in global context too in various publications (Fenwick, 2011, Ballantyne, 2006).

Quite a few tower damages/failures have been reported in various countries under earthquakes. Most infamous incident is deformation of Tokyo broadcasting tower during 2011 earthquake (see Fig. 1.1).

(a) A typical view Tokyo tower

(b) Deformed top part of Tokyo tower after earthquake in 2011

Fig. 1.1- Deformation of Tokyo tower due to earthquake in 2011(www. jurilog.jp)

Also, failures of telecommunication towers have been reported in earthquake events like 2009 earthquake in Indonesia (see Fig. 1.2) and 2011 earthquake in Haiti as well. However, detail forensic engineering studies relevant to these failures are not available. This indicates the importance of investigating the structural performance of telecommunication towers, which has not been addressed especially under regional context of South Asia.

Fig. 1.2 – A tower reported to be collapsed in Pandang earthquake 2009, Indonesia (Razak, 2015)

1.1.2 Baseplate design of lattice towers

All free standing self supporting towers act as vertical cantilevers. Therefore, when subjected to lateral forces due to wind and earthquakes, tower legs can be subjected to tension. Forces/stresses developed in tower legs are usually transferred to the foundation through a baseplate, which is connected to the foundation by anchor bolts. Therefore, baseplates are vital components of the structural system of a tower and failure of a baseplate may lead to a catastrophic failure due to the structural significance of it. Fig. 1.3 shows a photograph of a typical baseplate.

However, a proper design guideline for the design of baseplate is not available in almost all tower design codes. Due to the specific structural behaviour of a telecommunication towers that contrast to the behaviour of a typical steel column of a building, tower baseplates behave in a unique way. Therefore, usage of typical baseplate design guidelines specified in text books and steel building design codes are unacceptable even though certain designers may use these design guidelines without proper interpretation.

Fig. 1.3- Typical tower baseplate

When considering failures of lattice telecommunication towers, considerable number of towers has been reported to have failed at baseplate or near baseplates. Fig. 1.4 shows a failed transmission tower in Western Washington State, USA in 1999 (Eidinger & Kemper, 2012). Therefore, the need of complete design guideline for design of baseplates can be clearly identified. However, such detailed design guidelines are not presently available, especially where, tower legs subject to uplift forces.

Fig 1.4 - Failed power transmission tower at Washington, USA in 1999 (Eidinger & Kemper, 2012)

1.2 Objectives

The main objectives of this dissertation are of two folds:

1. Investigate the structural performances of self-supporting steel lattice towers under regional seismic loading conditions
2. Develop design guidelines for baseplates with governing uplift load condition

1.3 Methodology

For the investigation of structural performances of self-supporting lattice towers under seismic loadings, selected four leg and three leg towers representing different commonly used height ranges were analyzed using Response spectrum techniques to assess seismic performance. To cover the whole range of seismic effects, minor to severe seismic hazard levels and a whole spectrum of sub soil characteristics, which also significantly affects the seismic loadings, were considered in the analysis. 3D computer aided finite element modeling was carried out using actual data of selected existing towers in this regard.

For the development of a design guideline for baseplate with four anchor bolts, which is the most common anchor bolt arrangement in towers, Yield line theory was adopted to derive the basic design formula. The formulae derived were firstly verified with a laboratory testing with scale down physical models. For further validation/verification and for introduction of necessary correction factors for practical adaptation, necessary parametric studies were carried out using 3D computer aided nonliner finite element analysis. Simplified semi empirical formulae were also introduced for design office applications with acceptable accuracy at the end considering the complex nature of developed theoretical formulae.

1.4 Significance and innovation of the research

Following significance and innovations were addressed in this research:

- Seismic performances of Three leg and Four leg self supporting lattice towers under South Asian seismic hazard levels
- Comparison of wind and seismic performances of Three leg and Four leg self supporting lattice towers
- Variation of seismic forces/stress on Three leg and Four leg self supporting lattice towers with sub soil properties
- Development of Yield line model for tower baseplates under governing uplift loading
- Experimental investigation of tower baseplates with governing uplift loading
- Numerical modeling of tower baseplates under governing uplift loading
- Development of design theory for design baseplates under governing uplift loading

1.5 Outline of the thesis

Chapter one is the introduction and it discusses the background, objectives, methodology and briefly presents the main findings of the study.

Chapter two presents various literature available related to this study.

Chapter three presents seismic analysis of self supporting lattice telecommunication towers. Seismic analysis of both Three leg and Four leg self supporting lattice towers with regional seismic parameters and various sub soil conditions are discussed in this chapter.

Chapter four focuses on investigation on behaviour of lattice tower baseplates under uplift forces based on Yield line theory. Experimental and numerical validation of developed theory is also discussed.

Chapter five discusses the development of design formulae for lattice tower baseplates. Modification of fundamental design methodology to suit practical applications considering other effects is the prime discussion in this chapter.

Chapter six is the final chapter of this dissertation and it focuses on Conclusion of the study and recommendations.

Chapter 2 – Literature review

2.1 Structural aspects of telecommunication/broadcasting towers

Various structural forms are used for steel telecommunication/broadcasting towers constructions. Those structural forms are as follows:

1. Guy Masts
2. Monopoles
3. Poles with supporting struts
4. Self supporting lattice towers

2.1.1 Guy Masts

Guyed mast is a structure, which is pin connected to base and braced with guys (see Fig. 2.1). Center mast can be formed by a triangular or a square lattice structure or as a pole and it may be in tapered or straight shape in vertical profile. Generally, guyed masts are lighter in weight and relatively inexpensive when compared with other structural forms, however it requires a large ground area. This is due to the space required for cable guy wire stays, which are to be anchored at least in three directions since these wires can only take tensile forces.

Typical front view ***pin connection at base***

Fig. 2.1 – A typical guy mast (at Anamaduwa SLT premises)

This structural form can be used even for very tall structures and there was a Guy Mast tower of 180 m in Jaffna, Sri Lanka. It was the tallest telecommunication/broadcasting tower in the country until it was dismantled few years back. However, this may not be a feasible structural form if the required windshield area of antennas is large. Movable trailer mounted guy masts are also in operation now. The structural formation of these can also be considered as Guy mast towers. Percentage of Guy mast towers out of total telecommunication towers in the country is about 15%.

2.1.2 Monopoles

A single self-supporting vertical pole without guy wires or supporting struts is referred to as a monopole (see Fig. 2.2). Normally, a monopole is formed by tapered steel tubes that fit over each other to form a stable pole. Usually, these tubes have cylindrical or multi sided shapes like octagonal, hexagonal cross sections and these are produced out of galvanized steel or high tensile steel. In some instances, the entire pole is constructed by tubes of the same size and tubes are connected to each other by using connecting plies.

Fig. 2.2 – A typical Monopole at Jaffna

Heavy and large foundations are needed for these towers as structural effects caused by wind forces are transferred to foundation as a bending moment. Usually this structural form is used for relatively shorter towers (especially as roof top towers) and the tallest tower in this structural from is about 40 m in Sri Lanka. Tall monopole towers are comparatively few in Sri Lanka other than short rooftop monopole towers (in the range of 3 m to 10 m). However, it is gaining popularity for 30 m to 40 m height range due to factors like requirement of relatively less ground area, aesthetically pleasing appearance (even it may be camouflage as a tree), etc.

2.1.3 Poles with supporting struts

A vertical pole with inclined supporting struts (at least two or more) is another structural form especially used in roof top tower construction (see Fig. 2.3). This structural form has been developed to overcome the shortcomings of structural forms such as monopoles and guyed masts.

Fig. 2.3 – A typical pole with supporting struts (SLT rooftop tower at Galle)

Bending Moments develop at the base level of the pole can be significantly reduced by providing supporting struts to the pole. Therefore, this structural form can be used when it is required to erect a pole of greater height especially in rooftop where it is necessary to control the bending moment transferred to the roof structures. Also, installation of struts only in two directions is enough to stabilize the pole effectively, since struts can take both tensile and compressive forces. However, this is not possible if cables are used instead of struts (as in guyed mast) since cables can only take tensile forces.

However, application of this structural form for Greenfield towers is extremely rare in the country. One of the probable reasons for this may be slenderness problem that can arise in supporting struts with the increase of the height of the pole.

2.1.4 Self Supporting Lattice towers

Fig. 2.4 shows a photograph of a typical self-supporting lattice tower. Self-supporting lattice towers are the most commonly adopted structural form for telecommunication and broadcasting towers. A self-supporting lattice tower behaves as a vertical cantilever structure, when the whole structure is considered as one unit. However, it can be considered as a space truss in the structural analysis and design for convenience.

Fig. 2.4 – A typical self-supporting tower (at Kilinochchi SLT premises)

This structural form can be considered as the strongest form of tower structure (Pavan Kumar et al., 2017) and it can be designed to withstand very large wind loads, where it is not possible to use any other structural form. However, self supporting lattice towers are comparatively expensive compared with other structural forms. The space required at the base level is comparatively less, when compared with structural forms such as guyed masts.

Three or Four legs are usually used for self supporting lattice towers. Plan view of the tower becomes a triangle or a square based on the number of legs. There is no specific structural rule to select number of legs for a tower. However, based on the non structural factors like aesthetic aspects, fabrications/detailing aspects, antenna mounting requirements, etc. designers/clients select three or four leg for their towers. In Sri Lankan context, number of four leg towers are proportionately high compared to three leg towers and further, self supporting towers have a share of more than 70%(considering both three leg and four leg towers) out of all telecommunication/broadcasting towers in the country based on an own survey carried out in this regard.

2.2 Seismic effects on telecommunication/broadcasting towers

As discussed in Chapter 1, telecommunication/broadcasting towers play a vital role in their respective telecommunication/broadcasting networks by facilitating various wireless transmissions. Therefore, a failure of a telecommunication/broadcasting tower in a disaster at crucial location can become a serious concern as it could cutoff the entire communication link between the affected area and the rest of the country. It can hamper the essential communication needs of rescue operations, sending awareness messages/warnings to the people in affected areas, etc. Such a situation had been experienced by the country in the past and certain areas had lost their entire communication links for few days in 2004 Tsunami due to failure of telecommunication towers (Fig. 2.5 shows a picture of a collapsed telecommunication tower at Hambantota during 2004 Tsunami).

Fig 2.5 – Collapsed self supporting tower at Hambantota, Sri Lanka in 2004 Tsunami (www.hikenow.net)

Similar situations have been reported globally as well. Further, failure of towers itself can cause life losses or great property damages to adjacent properties as these are tall structures compared to residential and ordinary structures.

Traditionally, according to tower design codes (ANSI/TIA-222-G(2005), EN 1993-3-1(2006), IS 802(part 1/Sec 1):1995, etc.) wind forces are the predominant forces that govern the structural design of telecommunication towers . Hence, these design codes have presented methods to calculate wind force on tower body and antennas. In Sri Lankan context, it was found that almost all towers in the country have been designed considering the wind forces and have not been checked for the seismic forces (by an own survey done regarding design calculations of telecommunication towers) as Sri Lanka was considered as a country free from seismic activities until recently. However, this attitude amongst the professional community of the country regarding earthquake had started to change especially after earth tremors reported in 1993 and 2004 Tsunami and importance of considering seismic effects for structural designs was emphasized. Accordingly, especially for high rise buildings in the country, the seismic design is gradually becoming a must but, seismic analysis for telecommunication towers is still not practiced.

The telecommunication/broadcasting towers especially at crucial locations shall be considered as post disaster structures and it is a good practice that such structures should be designed considering all possible natural and non-natural disasters. Earthquakes are one of such natural disasters, where it can cause devastating damages to the structures. Failure of telecommunication towers may completely cutoff the essential communications systems as that was experienced in Sri Lanka in 2004 Tsunami.

2.2.1 Seismic hazard levels in Sri Lanka and South Asia

2.2.1.1 Seismic hazard levels in Sri Lanka

Sri Lanka was considered as a country free from earthquakes until recently at least by general public. Even the scientific community paid a relatively low attention to this matter until 2004 Tsunami, where it became an alarm call for the entire society on this subject.

However, even written records are available as old as 1615 regarding earthquakes that had taken place in the city of Colombo. This has been reported in a four (04) page pamphlet published in Lisbon in 1616. According to historical records, the death toll of that incident was more than 2000 and number of damaged houses was more than 200. This earthquake was thought to be of magnitude 6.5 M_L (Richter magnitude scale) or of intensity MMI VIII (This represents severe earthquake damage in Modified Mercalli intensity scale). In additions to above event, many earthquakes of low magnitudes have been reported in and around Sri Lanka. Fig. 2.6 shows earthquakes plotted in a geographical map of the region by Peiris, 2007 in his research on seismic hazard assessment on Sri Lanka.

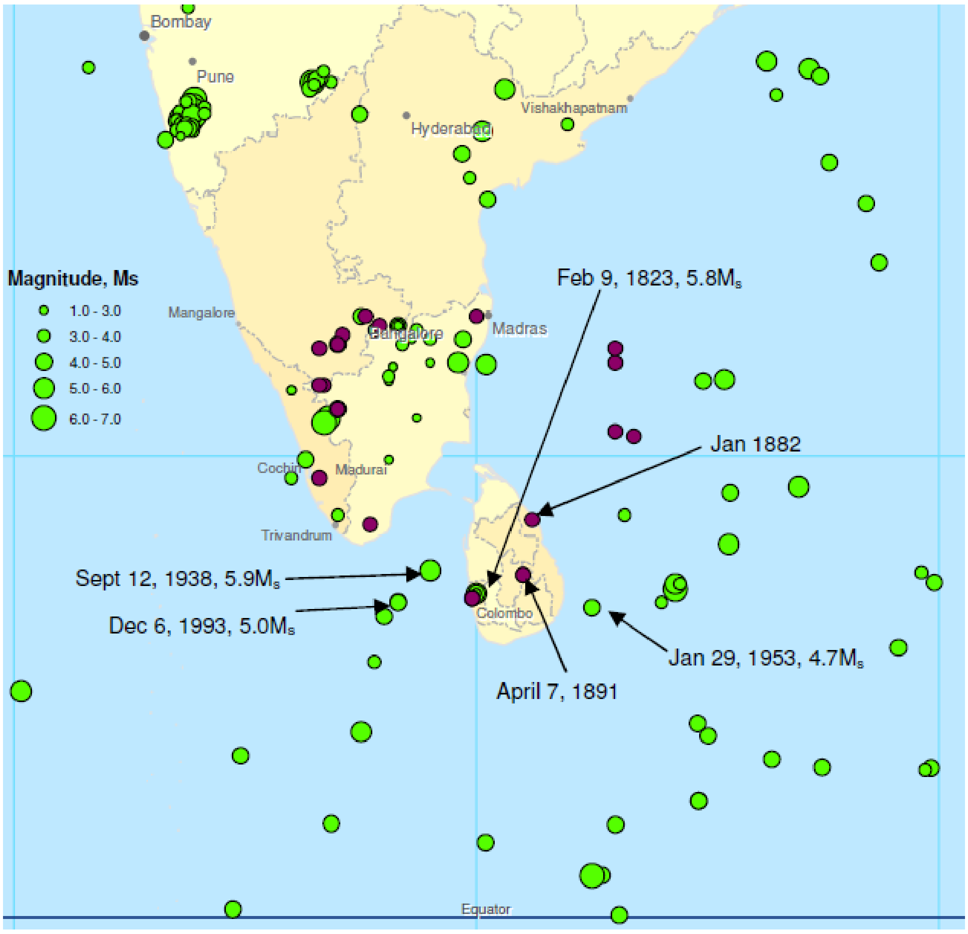


Fig. 2.6 -Earthquakes reported in and around Sri Lanka (Peiris, 2017)

One of such events that were felt by most Sri Lankans on December 6, 1993 was of magnitude 5.0 M_S. The epicenter of that earthquake was located about 170 km West of Colombo (see Fig. 2.6). This event however did not result in any property damage or loss of life. In additions to that few minor tremors (more than five incidents) were reported from May to December in 2014 in Ampara area. As per the records of Geological Survey and Mining Bureau of Sri Lanka (GSMB), the magnitudes of those tremors are in the range of 3-4 of Richter scale and no significant damages were reported due to such tremors.

Nevertheless, 2004 Tsunami that caused the deaths of over 40,000 and devastating property damages to the costal belt starting from Jaffna to Colombo along the eastern coast was the eye-opener for scientific and engineering community regarding seismic hazard levels and seismic resisting structures. This Tsunami was generated by a massive undersea earthquake having a magnitude of 9.0 of Richter scale occurred close to island of Sumatra.

Accordingly, structural designers started to consider seismic effects especially for high rise buildings and other important structures. However, there was no seismic Code of Practice for the country to follow (even still not available). At least, specific Peak Ground Acceleration (PGA) value and other country specific structural engineering parameters relevant to earthquakes were not available then. Therefore, various designers started to select these values based on various guidelines for their analysis and in the meantime researchers started to work on the matter to develop such parameters.

One of such earliest research is a research done by Peiris, 2007. He developed a PGA value and uniform hazard response spectra (UHRS) for Colombo by probabilistic seismic hazard assessment (PSHA). He had considered a zone bounded by 0ºN to 15ºN and longitudes 75ºE to 85ºE and seismic events recorded from 1800 to 2007 (having minimum magnitude of $4.0M_s$) for the study. According to this study, a PGA of 0.026 g was recommended for 10% probability of exceedance in 50years or 475year return period (for rock sites). However, there were lots of arguments about the accuracy of this value.

Subsequently, many research were carried out regarding the seismic parameters of Sri Lanka for structural analysis and design. One of such research was carried out by Uduweriya et al., 2014. They had carried out a Probabilistic Seismic Hazard Assessment (PSHA) for Colombo City with the objective of developing a suitable response spectrum curve. As per the results of the study, it had recommended a PGA value of 0.1 g at rock site for 10% of probability of exceedance in 50 years or 475 years return period for Colombo City.

During same time period, Dananjaya et al., 2014 had also carried out a study to develop PGA values for major cities of the country. However, they had adopted a Numerical simulation approach different to the PSHA using a finite difference computer programme. As per the results of their study, three seismic zones have been proposed for the country (see Table 2.1 and Fig. 2.7) and three different PGA values have been specified for those zones.

Table 2.1 – PGA values for Sri Lanka (Dananjaya et al., 2014)

Zone	PGA value at rock site for 10% of probability of exceedance in 50 years
Zone 1	0.12
Zone 2	0.07
Zone 3	0.08

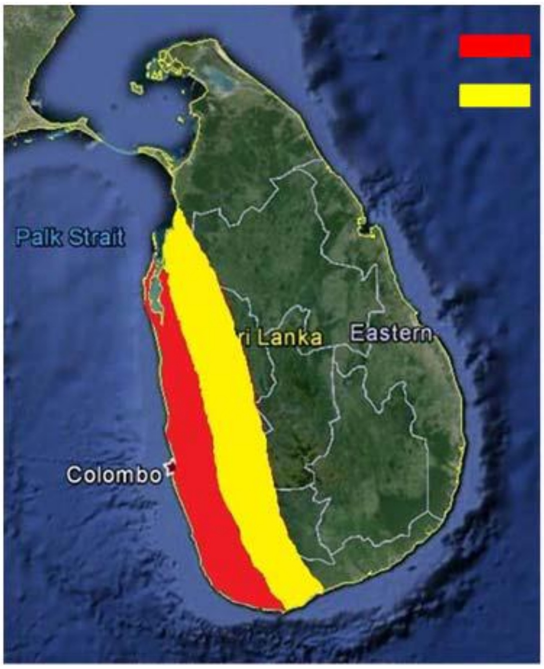


Fig. 2.7 – Zonal distribution of PGA values for Sri Lanka (Dananjaya et al., 2014)

As per their zonal map, entire Western costal belt (including Colombo and its sub urban areas) is in the Zone 1 and recommended PGA value for that zone is 0.12 g, which is quite close to the value proposed by Uduweriya et al., 2014.

The most recent publication on this subject has been published by Venkatesan et al., 2017. It has presented a response spectrum curve for Sri Lanka for seismic analysis considering a PGA value of 0.08 g and further specified that Sri Lanka should consider as a low-moderate seismic region rather than a low seismic region.

In summary, as per above recent research data, recommended PGA value for Sri Lanka varies from 0.07 g to 0.12 g. Since seismic code for the country is still not available, it would be prudent to use a PGA value of 0.1 g (for rock sites) for the analysis of the telecommunication towers under Sri Lankan conditions.

2.2.1.2 Seismic hazard levels in South Asia

Even though seismic hazard levels of Sri Lanka are relatively low as a country, South Asian region as whole is not so blessed from Mother Nature regarding earthquakes. Countries like Nepal, Bhutan and northern part of India and Pakistan which locate close to tectonic plate boundaries between Eurasian plate and Indian plate are under severe seismic threats (see Fig. 2.8). "Gorkha" earthquake in Nepal (2015) and Kashmir earthquake of 2005 are few examples for this threat. These catastrophic earthquakes cause large number of human losses and massive physical destructions causing socioeconomic crises for respective countries.

***Fig. 2.8 – Tectonic plate map of South Asia** (**www.geologyin.com**)*

Fig. 2.9 shows part of Global Seismic Hazard map relevant to South Asia prepared by Global Seismic Hazard Assessment Program (GSHAP). It also shows the high seismic vulnerability of South Asian countries located close to tectonic plate boundaries.

Therefore, these countries adopt seismic design procedures for their structures. Peak Ground Accelerations (PGA) values specified by respective seismic design code of practices main cities of these countries are given in Table 2.2.

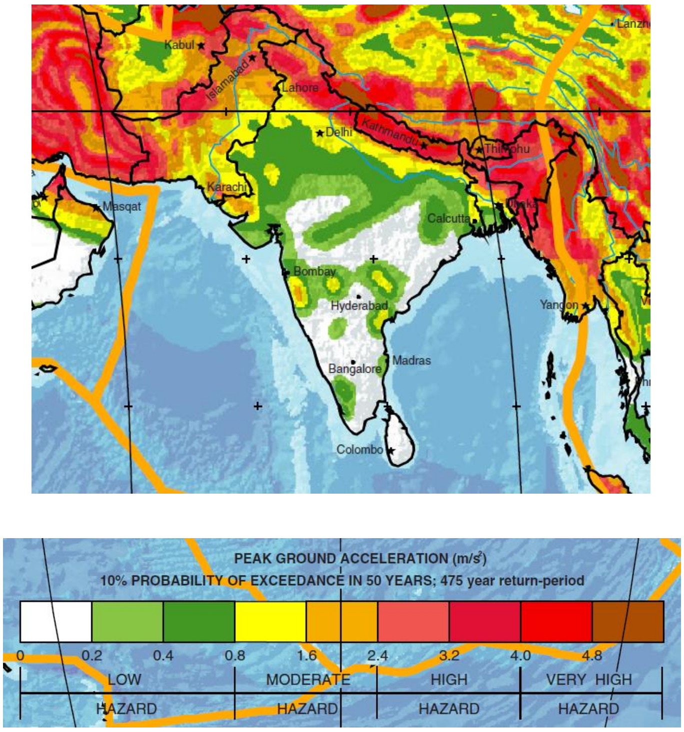


Fig. 2.9 - Global Seismic Hazard map relevant to South Asia prepared by GSHAP (www.gfz-potsdam.de/en/GSHAP, 1999)

Table 2.2 - PGA values of cities of South Asia given in design codes

City	Country	PGA specified by seismic code (g)	Reference seismic code
New Delhi	India	0.24	IS-1893, 2002
Mumbai	India	0.16	IS-1893, 2002
Islamabad	Pakistan	0.25	Building Code of Pakistan (Seismic Provision)-2007
Karachchi	Pakistan	0.16	Building Code of Pakistan (Seismic Provision)-2007
Katmandu	Nepal	Z = 1.0* PGA value is not defined	NBC 105 : 1994
Dhaka	Bangladesh	0.18	BNBC -206
Thimpu	Bhutan	0.19	IS-1893, 2002 (no separate code is available)

2.2.2 Previous studies of seismic effects on telecommunication/broadcasting towers

A recent survey carried out on earthquake-induced damage to telecommunications towers (1999-2011) by Osgoie, 2012 have highlighted considerable number of incidences where telecommunication tower had experienced structural or non-structural damages by earthquakes during above time period. Further, Osgoie, 2012 have quoted a previous research done by Schiff, 1999 where, 16 instances of structural damage related to towers under 7 important earthquakes between 1949 and 1998 have been reported. In addition, that a study done by Razak, 2015 have mentioned about few tower failures in Indonesia during recent earthquakes. However, detail discussions about damages, structural repercussions, etc. of those occasions have not been discussed in any of these studies. One probable reason may be, even if the details are collected regarding damages, structural repercussions, etc. by the experts of respective telecommunication operators who owns these towers, those data would not make available publically due to various strategic business related concerns.

There are various studies carried out regarding the structural performances of the telecommunication/broadcasting towers too in various countries as early as 1973. Komo and Kimura, 1973 had done a study aimed mainly at collecting information on tower vibration mode shapes, natural frequencies and damping properties. A real case study of a tower that was instrumented when the 1968 Off-Tokachi earthquake occurred was presented. Mikus, 1994 studied the seismic response of self-supporting telecommunication towers using modal superposition analysis. The aim of this preliminary study was to improve understanding of the responses of towers to earthquakes.

The first attempt to propose equivalent static method for seismic analysis of lattice self-supporting towers was made by Gálvez, 1995. This method had been developed based on model superposition theory of lowest three flexural vibration modes. Further, Gálvez & McClure, 1995 had proved that the contributions of both the second and third flexural modes are also significant under seismic assessment.

Khedr, 1998 had done a comprehensive research on seismic analysis of lattice towers. Inaccuracy of using static equivalent equations developed for building structures to assess seismic performances of lattice towers have been highlighted in his research. Unique structural characteristics of lattice towers that differed from building structures had been highlighted. Theory of Response spectrum analysis is briefly described here:

Simplified Single Degree Of Freedom (SDOF) Equation of Motion under the effect of base excitation is

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\ddot{y}_s(t) \quad (2.1)$$

where,

$\ddot{y}_s$	= The base acceleration
m	= Mass
k	= Stiffness
c	= Viscous damping constant
$\ddot{u}$, $\dot{u}$ and u	= Relative acceleration, velocity and displacement receptively, with respect to the moving base and are functions of the t

This equation can be solved using Duhamel's integral, and the solution is given by:

$$u(t) = \frac{1}{m\omega_D}\int_0^t -m\ddot{y}_s(\tau)e^{-\xi\omega(t-\tau)}\sin\omega_D(t-\tau)d\tau \quad (2.2)$$

Where

ξ	= Viscous damping ratio = c / 2ωm
ω and ω_D	= Undamped and damped circular frequency of vibration

For smaller damping values:

$$u(t) = \frac{1}{\omega}\int_0^t -m\ddot{y}_s(\tau)e^{-\xi\omega(t-\tau)}\sin\omega(t-\tau)d\tau \quad (2.3)$$

By taking first derivation

$$\dot{u}(t) = \int_0^t \ddot{y}(\tau)e^{-\xi\omega(t-\tau)}\cos\omega(t-\tau) - \varepsilon\int_0^t \ddot{y}(\tau)e^{-\xi\omega(t-\tau)}\sin\omega_D(t-\tau)d\tau \quad (2.4)$$

By substituting above two equations, absolute acceleration $\ddot{u}^t$ can be defined as:

$$\ddot{u}^t(t) = \omega(2\varepsilon^2 - 1)\int_0^t \ddot{y}_s(\tau)e^{-\varepsilon\omega(t-\tau)}\sin\omega(t-\tau)d\tau + 2\omega\xi\int_0^t \ddot{y}_s(\tau)e^{-\xi\omega(t-\tau)}\cos\omega(t-\tau)d\tau \quad (2.5)$$

The absolute maximum values of the quantities given by (2.3), (2.4) and (2.5) are the spectral relative displacement $S_d(\xi,T)$, the spectral relative velocity $S_v(\xi,T)$, and the spectral absolute acceleration $S_a(\xi,T)$.

Pseudo velocity response spectrum $S_{pv}(\xi,T)$ is defined as

$$S_{pv}(\xi,T) = \left|\frac{1}{\omega}\int_0^t -m\ddot{y}_s\,(\tau)e^{-\xi\omega(t-\tau)}\sin\omega\,(t-\tau)d\tau\right|_{max} \tag{2.6}$$

The relative spectral displacement

$$S_d(\xi,T) = \frac{S_{pv}(\xi,T)}{\omega} \tag{2.7}$$

For small damping ratios (ξ <0.1),

$$S_a(\xi,T) = \omega\, S_{pv}(\xi,T) \tag{2.8}$$

Which is called as Pseudo acceleration and called as $S_{pa}(\xi,T)$. Pseudo acceleration can be used to estimate the maximum elastic forces.

Equation of motion of a continuous system can be expressed in following form;

$$f_I(x,t)+ f_D(x,t) + f_S(x,t) = 0 \tag{2.9}$$

The terms in eq. 2.9 are inertia force, damping force and elastic force respectively.

By considering displacement of this system to a single mode shape i for instance and applying a virtual displacement of the form ;

$$\delta u = \phi_i(x)\delta y \tag{2.10}$$

Then, eq. 2.9 can be rewritten as ;

$$f_{I,i}\,\delta y + f_{D,i}\,\delta y + f_{S,i}\delta y = 0 \tag{2.11}$$

Where,

$$f_{I,i} = \int f_I(x,t)\phi_i\,(x)dx \tag{2.12}$$

$$f_{D,i} = \int f_D(x,t)\phi_i\,(x)dx \tag{2.13}$$

$$f_{S,i} = \int f_S(x,t)\phi_i\,(x)dx \tag{2.14}$$

are the generalized inertia force, the generalized damping force and generalized elastic force for mode I, respectively. But inertia force depends on total acceleration. Hence,

$$f_I(x,t) = m(x)\phi_1(x)\ddot{y}(t) + m(x)\ddot{y}_s(t) \tag{2.15}$$

By substituting in eq. 2.12

$$f_{I,i} = \ddot{y}(t)\int \phi_i(x)m(x)\phi_i(x)dx + \ddot{y}_s(t)\int m(x)\phi_i(x)dx \tag{2.16}$$

$$f_{I,i} = \ddot{y}(t)M_i + \ddot{y}_s(t)L_i \tag{2.17}$$

Where M_i and L_i are the generalized mass and the excitation factors of mode i.

Further, if C_i and K_i are defined as the generalized damping and the generalized stiffness of mode i;

$$f_{D,i} = C_i\,\ddot{y} \tag{2.18}$$

$$f_{S,i} = K_i\,y \tag{2.19}$$

Hence, eq. 2.12 can be rewritten as

$$M_i\;\ddot{y}(t) + C_i\dot{y}(t) + K_i\,y(t) = -L_i\;\ddot{y}_s(t) \tag{2.20}$$

By dividing Mi and substituting $C_i = 2\omega_i\xi_i\,M_i$ and neglecting sign of right hand side,

$$\ddot{y}(t) + 2\omega_i\xi_i y(t) + \omega_i^2\,y(t) = \frac{L_i}{M_i}\ddot{y}_s(t) \tag{2.21}$$

Based on response spectrum technique, maximum acceleration profile of mode i is;

$$\ddot{u}_{imax}(x) = \phi_i(x)\frac{L_i}{M_i}\;S_{pa}(\xi_i, T_i) \tag{2.22}$$

The contribution of mode i to the maximum elastic force acting on the structure is given by:

$$f_{S,i\,max}(x) = m(x)\phi_i(x)\frac{L_i}{M_i}\;S_{pa}(\xi_i, T_i) \tag{2.23}$$

Using this maximum elastic force, bending moment and shear forces of the structure under each mode can be separately calculated. Equations for modal shear force (V_i) and (M_{fi}) any relative position of x is given by:

$$V_i(x) = \int_1^x m\,(x)\phi_i(x)\frac{L_i}{M_i}\ S_{pa}\,(\xi_i, T_i)dx \tag{2.24}$$

$$M_{fi}(x) = \int_1^x m\,(x)\phi_i(x)\frac{L_i}{M_i}\ S_{pa}\,(\xi_i, T_i)xdx \tag{2.25}$$

Model contributions can be combined using various mathematical techniques and Square Root Sum of Squares (SRSS) is the most widely used technique for this type of structures under response spectrum analysis. Accordingly, the total response can be expressed as:

$$V(x) = \sqrt{\sum_{i=1}^{n} V_i\ (x)^2} \tag{2.26}$$

$$Mf(x) = \sqrt{\sum_{i=1}^{n} V_{fi}\ (x)^2} \tag{2.27}$$

However, Khedr, 1999 proposed simple static equivalent method on the definition of a horizontal acceleration profile a(x) that can give the same value of shear or bending moment at each tower section as the value obtained from the Square Root Sum of Squares (SRSS) responses combination of the three lowest flexural modes. Schematic representation of the method is given in Fig. 2.10.

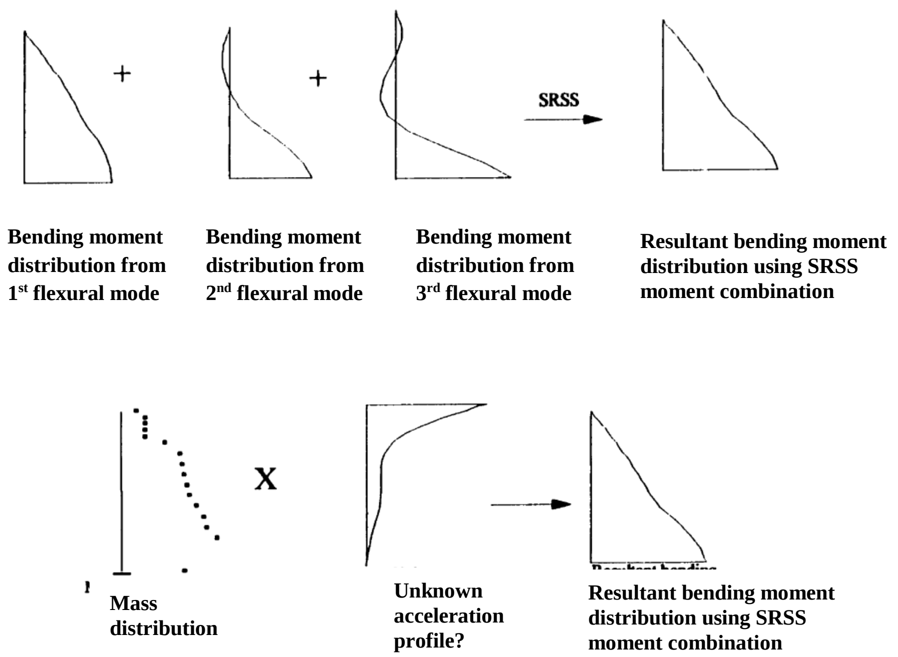


Fig. 2.10 - Schematic representation of the method developed by Khedr, 1999

The a(x) is defined as:

$$V(x) = \int_1^x a(x)m(x)dx = \sqrt{\sum_{i=1}^{3} V_i \ (x)^2} \qquad (2.28)$$

$$Mf(x) = \int_1^x a(x)m(x)\, x\, dx = \sqrt{\sum_{i=1}^{3} V_{fi} \ (x)^2} \qquad (2.29)$$

Khedr, 1998 developed curves to calculate *a(x)* for self supporting lattice towers based on analysis of ten (10) selected towers having heights from 30 m to 121 m. He had analyzed all these towers using Response Spectrum technique using three selected (03) accelerogrames considering first three (03) flexural modes. Results had also verified with acceptable accuracy of the computer aided Response Spectrum analysis. Therefore, this method can be considered as a more accurate Static Equivalent analysis method for self supporting lattice towers.

Publications of Amiri & Boostan, 2002 and Amiri et al., 2007 have focused on structural performances of lattice towers under earthquake loading. However, these studies have been predominantly done based on the Iranian seismic code and their local seismic parameters. Therefore, findings of those research cannot be directly applied to our regional conditions but can only be used as initial guideline.

There are few other research papers published by Vidaya, 2015, Long, 2018, etc. relevant to seismic effects of lattice telecommunication/broadcasting towers. However, direct relevancy of the findings of such work is questionable due to the fact that seismic effects are greatly influenced by location specific characteristics and tower geometry parameters.

2.2.3 Seismic analysis procedures in tower design codes

Most of the reputed tower design codes such as EN 1993-3-2, IS 802 do not directly discuss about seismic analysis. In, EN 1993-3-2 has directed to EN 1998-6, which covers earthquake resistance of Towers, Mast and Chimneys. In EN 1998-6, two analytical methods namely Lateral Force method and Modal response spectrum analysis have been discussed in detail. The first method is an equivalent static method while second one involves Response spectrum analysis as the name implies. Nevertheless, these discussions are more generic as this code covers various other structures in addition to steel lattice towers.

IS 802 also does not cover seismic analysis. Further, it has not given any direction for any other codes relevant to seismic analysis. In IS 1893 (Part 4), which covers earthquake resistance design of Industrial structures including Stack –like structures, discussed the seismic analysis and design of transmission and communication structures. It discussed about all forms of seismic analytical technique such as Equivalent static analysis, Response Spectrum analysis and Time history analysis. However, it also has a generic approach as it is not limited to steel lattice towers.

In the case of ANSI/TIA-222-G, a comprehensive seismic analysis procedure has been presented which is specific to lattice towers. Four (04) different analytical methods have been specified in this code:

(1) Equivalent lateral force procedure
(2) Equivalent Modal analysis procedure
(3) Modal analysis procedure
(4) Time -History analysis

The method 2 of the list, which is Equivalent Modal analysis procedure, is quite similar to the method proposed by Khedr, 1998. Even though it is a Static equivalent method, acceleration coefficients are calculated separately for considered levels using set of curves based on fundamental frequency of structures and spectral acceleration parameters. This approach had been developed based on the work of Khedr, 1998 and other researchers who worked on developing a more accurate Static Equivalent method for seismic analysis of self supporting towers. Detailed seismic analysis procedure of ANSI/TIA-222-G is discussed in Section 3.1.

2.3 Wind data of Sri Lanka and South Asia

Usually, design of lattice communication/broadcasting towers are governed by wind forces due to tall light weight nature of these structures. Due to geographical location of South Asian Subcontinent, Sri Lanka is usually subjected to two main monsoons namely North East monsoon and South West Monsoon. Cyclones are also common in this part of the world and frequency of occurrence of cyclones in Bay of Bengal is higher compared to Arabian Sea due to wind, sea currents and geographical factors. Therefore, countries like Bangladesh and North Eastern part of India suffer from cyclones regularly compared to other countries in the region. Still, cyclones are not infrequent in other areas as well and Cyclone Vardah in 2016 that badly affected

Southern India (Ayyappan et al., 2017), 1978 cyclone affected Eastern part of Sri Lanka (Design of buildings for high winds, Sri Lanka, 1980) and Cyclone Onil in 2004 that affected Pakistan and North West part of India are few good examples. Global factors like activation of El Niño and La Niña effects with Global warming have intensified the cyclonic attacks in the region and frequency of occurrence and scale of destructions has also intensified.

After a catastrophic cyclone that affected the eastern part of Sri Lanka in 1978, Ministry of Local-government, Housing and Construction introduced a guideline called "Design of buildings for high winds, Sri Lanka" (1980) and it was considered as the governing wind design document in the country until very recently.

This guideline had defined three wind zones for the country as given in Fig. 2.11 and given separate 3s design Gust wind speed values under 50 year Return period for the above zones. The above wind speed values are given in Table 2.3.

Table 2.3 – Design wind speeds of Sri Lanka (Design of buildings for high winds, Sri Lanka, 1980)

Wind Zone	Post disaster structures (ms^{-1})	Normal structures (ms^{-1})
Zone 1	54	50
Zone 2	47	42
Zone 3	38	33

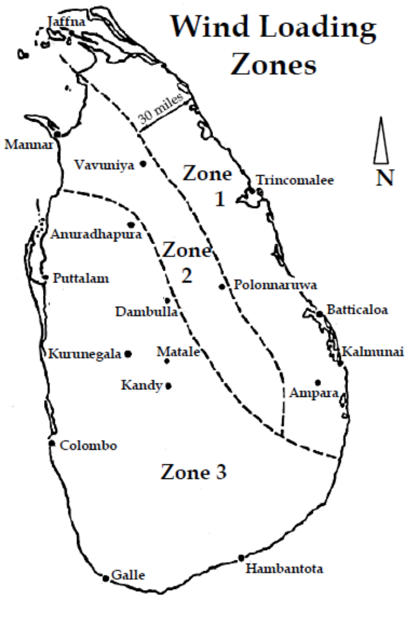


Fig. 2.11 - Design Wind zones of Sri Lanka (Design of buildings for high winds, Sri Lanka, 1980)

Recently, Maduranaga and Lewangamage, 2018 had carried out a research on development of updated wind loading map for Sri Lanka considering the shortcomings in the publication “Design of buildings for high winds, Sri Lanka” (1980). The basic wind speed contours and wind loading zones for Sri Lanka for different averaging times and return periods based on recoded historical wind data have been developed. The proposed wind speeds and wind loading zones are given in Table 2.4 and Fig. 2.12 respectively.

Table 2.4 – Basic design wind speed for zones proposed by Maduranaga and Lewanagamage, *2018*

Wind loading zone	Basic wind speed (ms^{1})							
	3-Second gust				10-Minute average			
	Return period (years)				Return period (years)			
	5	10	**50**	100	5	10	**50**	100
I	40	43	**50**	54	28	31	**35**	38
II	34	36	**42**	45	23	25	**29**	31
III	32	34	**38**	40	20	21	**23**	24

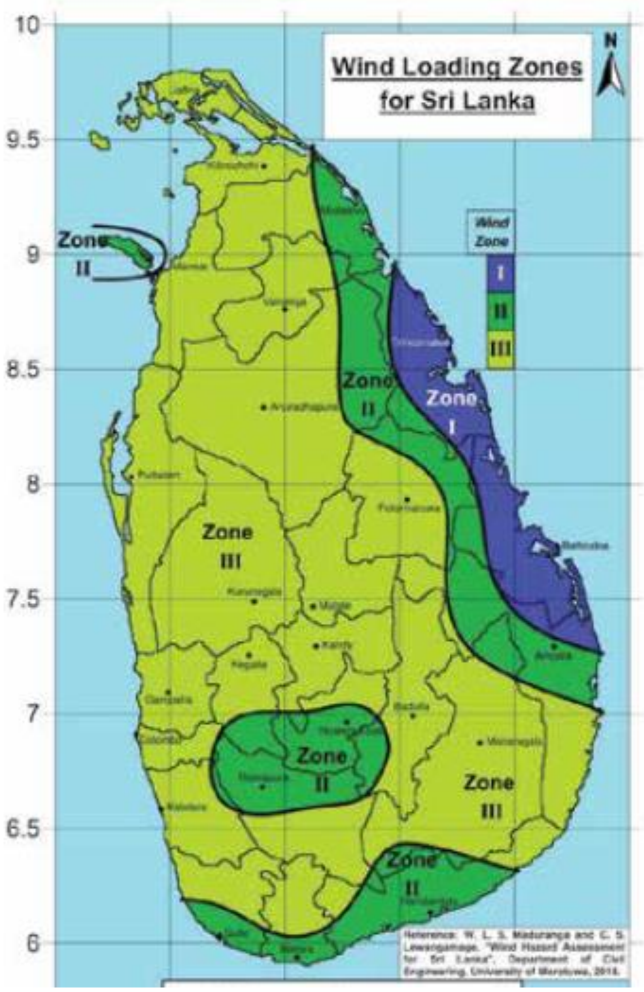


Fig. 2.12 – Wind loading zones proposed by Maduranaga and Lewangamage , 2018

When the wind zone map proposed by Maduranaga and Lewanagamage, 2018 is compared with previous wind zone map developed in 1980, some changes can be observed. One significant change is defining of a new wind zone (named as Zone II) having considerably different geographical boundaries Zone II of previous wind zone map as indicated in Fig. 2.12. However, design 3-second gust wind speed under 50 year return period of zone I remains 50 m/s as in the previous code. The design 3-second gust wind speed under 50 year return period of zone III has increased to 38 m/s from 33 m/s, the value specified in the previous code.

Similarly, all other countries in the region have developed their own wind design codes and 3 second Gust wind speed has used in some of these codes to specify design wind speeds. Table 2.5 gives a summary of 3-second Gust wind speed data specified in different code of practices.

Table 2.5– Design wind data in South Asian countries

Country	Maximum 3-second Gust wind speed specified (m/s)	Minimum 3-second Gust wind speed specified (m/s)	Code of practice
India	55	33	IS 875 :2002
Sri Lanka	50	33	Design of buildings for high winds, Sri Lanka(1980)
Pakistan	58	33	Building Code of Pakistan 2007
Nepal	55	47	NBC 104:1994
Bangladesh	80	41.4	BNBC 1993
Bhutan /Maldives	Specific data is not available		

2.4 Base connection of towers

2.4.1 Different type of connections

Base connection of lattice tower acts as the interface between tower body (super structure) and foundation (sub structure). Therefore, it is the most important connection of a lattice tower. A base connection should have an adequate structural capacity to withstand under all vertical and transverse force/stress combinations transfer from the tower body. Two (02) main techniques are used in tower industry for base connections (see Fig. 2.13):

1. Embedding of a sufficient length of tower leg in the stem column
2. Use of steel baseplates with anchor bolts.

The first method is being widely used in the electrical transmission structures of many Asian countries. The second method is being used in the telecommunication/ broadcasting towers because of the flexibility it offers during the erection of the tower and structural inadequacy of first method in case of taller towers due to higher forces/stresses.

a) Directly embedded the leg members in footing (www.gsengineering.lk)

b) Baseplate with anchor bolts (SLT tower at Dambulla)

Fig. 2.13 - Base connection of towers

The predominant force that governs the design of a baseplate of a tower is the upward or downward forces acting on the baseplate based on the wind direction. In addition to this vertical force, a relatively small transverse force would also act on the baseplate (see Fig. 2.14).

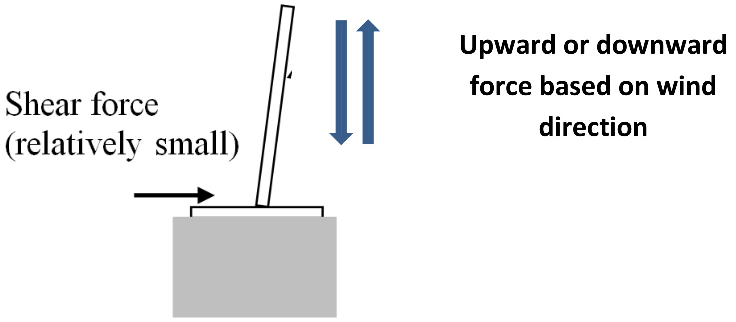


Fig. 2.14- Main forces at Base connection

Almost always it is the uplift forces which govern the design of a tower baseplate since when there are downward forces, the concrete grout filling below the baseplate would also help to control the deformation of the baseplate due to those forces.

2.4.2 Failures of towers at base connection

Quite a few failures at baseplate or base level have been recorded worldwide, even though publications on forensic engineering studies done on lattice towers that had failed are rare. Following are few examples for such failures:

1. Failure of a power transmission tower at Western Washington State in February 23, 1999. Deformed and uprooted base plate of tension side of the tower can be clearly observed (see Fig. 2.15).

Fig. 2.15 –A failed power transmission tower at Washington, USA (Eidinger & Kemper, 2012)

2. Failure of directly embedded power transmission tower of 765 KV Anta-Phagis S/C line of Central Electricity Authority of India on 14/03/2015. Here also the failure has taken place from the tension side of the tower (see Fig. 2.16).

Fig. 2.16 – A collapsed directly embedded power transmission tower of 765KV Anta- Phagis S/C line in India (Standing Committee report on Failures of tower, CEA, India, 2016)

3. Failure of PG&E electric transmission tower located in Moss Landing, California on 18/10/2015. The tower has been constructed with direct embedding technique. The forensic analysis has concluded that the failure had occurred due to misalignment of embedded part of leg member with the design inclination during construction (see Fig. 2.17).

Fig. 2.17- Failed PG&E electric transmission tower located in Moss Landing, CA, 2015 (Brian and Rayan, 2015)

This clearly indicates the importance of designing the baseplate of tower.

2.4.3 Design codes formulae for baseplate design

As explained in Section 2.4.1, uplift load case on the baseplate governs the design of baseplates of a lattice tower, which is not unlikely in case of a baseplate of a column in a building. However, tower design codes do not mention any specific guidelines for the design of baseplates. Design guidelines/rules given for column baseplate designs in steel design codes and standard text book are not directly applicable to towers since most of these theories had been developed considering downward forces on baseplates as it is the usual case of column baseplates. In case of downward compressive force transfer, the grout or concrete below the baseplate mainly contributes for stress transfer. However, under uplift force, stress transfer would take place mainly through anchor bolts and contribution of grout or concrete is negligible. Hence, design theories for uplift load cases should be developed accordingly.

In BS 5950: Part I (1990), which is relevant to structural design of steel buildings, has specified a simple empirical formulae given as Eq. 2.30 and Eq. 2.31 developed based on uniform distribution of stress on the underside of plates.

For I, H, Channel, box and RHS,

$$Min. thickness\ of\ plate \quad t = \left(\frac{2.5}{P_{YP}}\ w\ (a^2 - 0.3\ b^2\right)^{0.5} \tag{2.30}$$

For circular solid or hollow sections,

$$Min. thickness\ of\ plate \quad t = \left(\frac{D_P}{2.4P_{YP}}\ w\ (D_P - 0.9\ D\right)^{0.5} \tag{2.31}$$

Where,

w - Pressure on the underside of the plate assuming uniform distribution

P_{YP} - Design strength of the plate

a - Greater projection of the plate beyond the column

b - Lesser projection of the plate beyond the column

D_p - Length of the side or Diameter of cap of baseplate

D - Diameter of column

BS 5950: Part I (1990) has stated that bearing pressure on underside of a baseplate could be calculated using elastic theory and baseplate thickness could be calculated by applying General bending theory.

In Steel Designers Manual - 6th Edition published by Steel Construction Institute (Davidson and Owens, 2003) recommended an effective area method for baseplates where projections of the plate from column (where it cannot assume uniform stress distribution) would be large.

However, all of these theories are not directly applicable for tower baseplates where uplift load case governs the design.

In Base Plate and Anchor Rod Design- 2nd Edition American Institute of Steel Construction (Fisher and Kloiber, 2006) provides a comprehensive discussion of baseplate design and analysis. Theories presented under compressive loads and bending are very much similar to the theories presented in European design codes and text books. In addition to above cases, it discussed about baseplates under tension as well. According to the procedure specified in this text book, design of plate can be carried out assuming cantilever bending action of the plate caused by anchor bolts forces. Either one way or two way bending has to be considered based on geometry and number of anchor bolts of the plate.

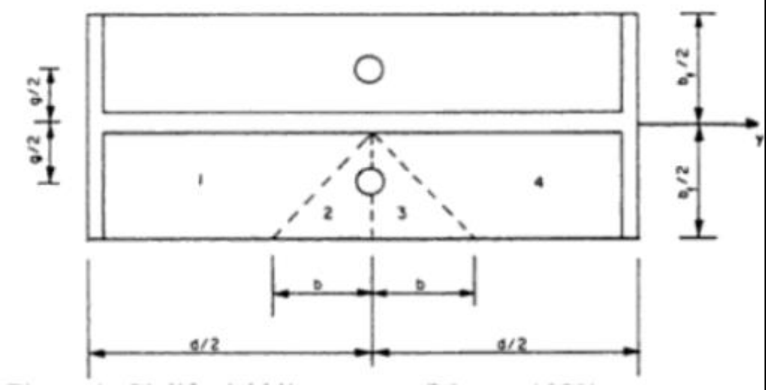
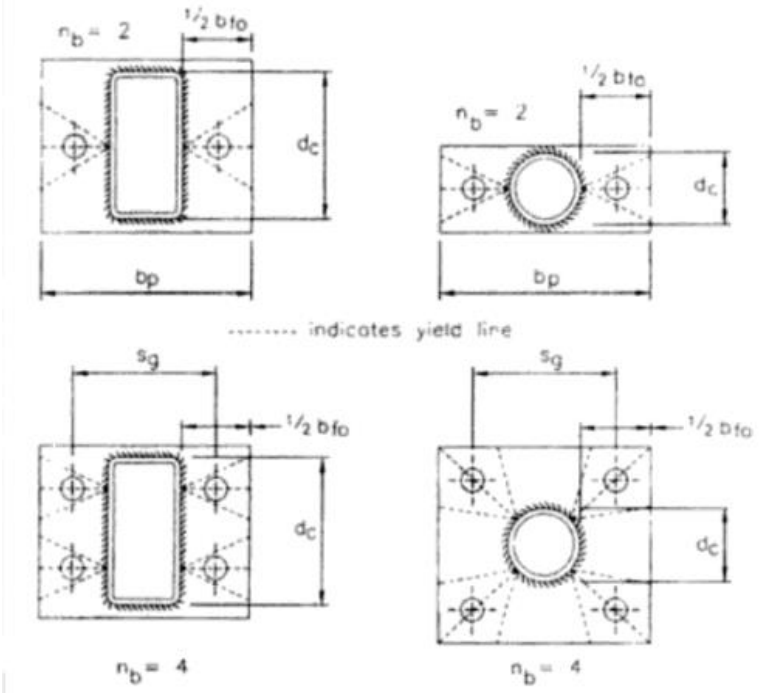
Even though this approach can be used to design baseplates of towers as well, analysis will not be a simple and accurate due to special features of tower baseplates like non-uniform distribution forces amongst anchor bolts, two way bending of plate due to geometry of baseplates, etc.

2.4.4 Present practice of baseplate design of transmission towers and research work on baseplates under uplift

Due to non-availability of specific design guideline for baseplate design of transmission towers, Finite element modelling is the most appropriate method for analysing the baseplates of towers which are under uplift loading although it is a tedious exercise. Hence, it is rarely being used by designers. The other alternative method for this is adaptation of approximate analytical methods based on plate bending. However, inherent deficiencies of structural modelling done using this type of approximate analytical methods can lead to costly overdesigns or in the worst case, structurally inadequate baseplates.

Only few research papers have been published on the design of tower baseplates and column baseplates subjected to uplift forces. However, these guidelines may still not be directly applicable to the design of tower baseplates because of the specific physical shapes of tower members, baseplates, bolt arrangements, etc. Murray's study (1983) which is one of the earliest studies done in this regard, discusses the behaviour of a column baseplate of a steel "H " section fixed with two anchor bolts and which have been subjected to uplift forces and it presents a formula to calculate the thickness of the baseplate. Hogman & Thomas, 1984 and Syam & Chapman, 1996 also have studied the behaviour of baseplates that are under uplift forces. All these studies have used Yield line theory to develop formulae to calculate the thickness of the baseplates under respective cases. Based on identified yield line patterns and energy optimization theory, design formulae have been attained. Table 2.6 shows the summery of findings of such research.

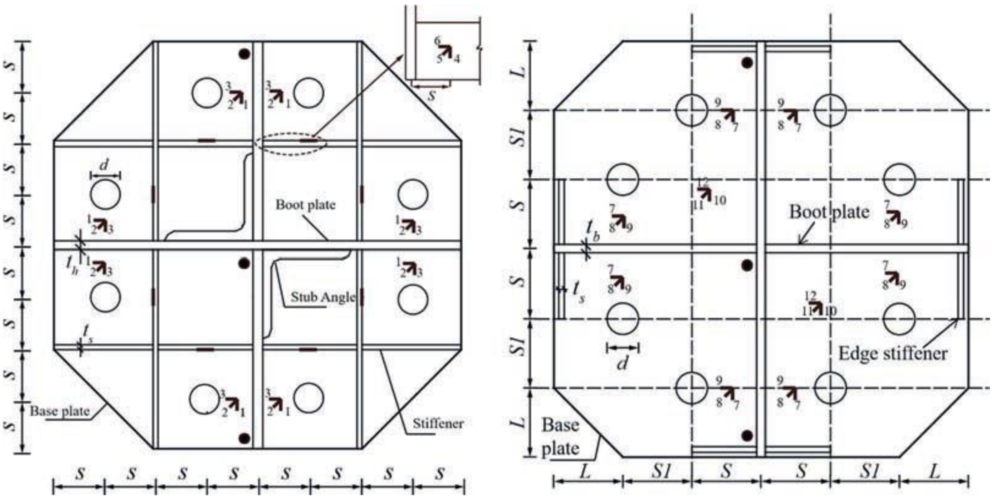
Table 2.6 - Present design theory of baseplate subjected to uplift

Case	Graphical detail and design formula	Developer and year
"I" section with two bolts subjected to uplift with two anchor bolts	 $t = (1.414P_u g/4b_f F_y)^{1/2}$ when $1.414\ \ b_f <= d$ $t = [P_u g / F_y (d^2+2b_f^2)]^{1/2}$ when $1.414\ \ b_f > d$ P_u - Total uplift force on the baseplate g - Anchor bolt gauge f_y - Yield stress of baseplate t - Thickness of baseplate	Murray (1983)
I sections, channels, rectangular, square and circular hollow sections with multiple anchor bolts	 $\varnothing Ns = \varnothing 4b_{fo}f_{yp}t_p^2\, n_b / 2\sqrt{2}\, s_g$ when $\sqrt{2}b_{fo} <= d_c$ $\varnothing Ns=\varnothing f_{yp}(d_c+2b_{fo}^2)t_p^2 n_b/2s_g\, d_c$ when $\sqrt{2}b_{fo} > d_c$ $\varnothing Ns$ - Design capacity of steel baseplate f_{yp} - Yield stress of component t_p - Thickness of baseplate $\varnothing = 0.9$ $b_{fo} = b_p - b$	Hogan and Thomas (1994) Syam and Chapman (1996)

Mills et al., 1997 have carried out an experimental investigation and finite element simulation of the formula proposed by Hogman & Thomas, 1984. According to the results of their studies, validity of such simple formula for vast range of structural member profiles and anchor bolt arrangements have been questioned. The changing of governing yield line patterns with the different bolt configurations and member profiles have been highlighted. Willibald, 2002 had also carried out a study on Bolted Connections for Rectangular Hollow Sections under Tensile Loading and his research findings also highlighted changes of governing yield line patterns with different bolt configurations and questionability of using simple design formulas developed based on one dimensional yield line patterns for complicated baseplate arrangements. However, the findings of these studies are not directly applicable to telecommunication/transmission towers since the structural shapes of the members of these towers are unique and different from those of typical column sections. Anchor bolt arrangements, connection patterns, etc. are also different in lattice towers compared with columns of buildings.

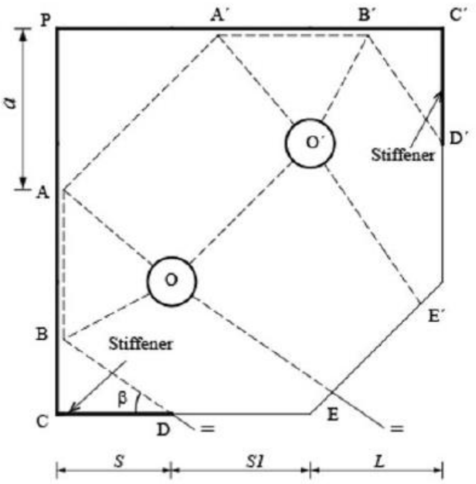
In recent study done by Liu et al., 2018 has focused on the behaviour of the baseplates of Extra High Voltage (EHV) transmission towers. This study had focused on development of baseplate design formula for two types of EHV tower baseplates. Fig. 2.18 shows the graphical view of such baseplates.

In this study, Yield line analysis has been used to develop formulae and such formulae have been verified through a physical testing of samples. Developed design formulas for Case 1 and Case 2 are given in Fig. 2.19 and Fig. 2.20, respectively. However, configurations of telecommunication/broadcasting tower baseplates differ from these geometries and direct applicability of these formulae for baseplate design of telecommunication/broadcasting towers are limited.



Case 1- Rigid EBBP connections *Case 2- Flexible EBBP connections*

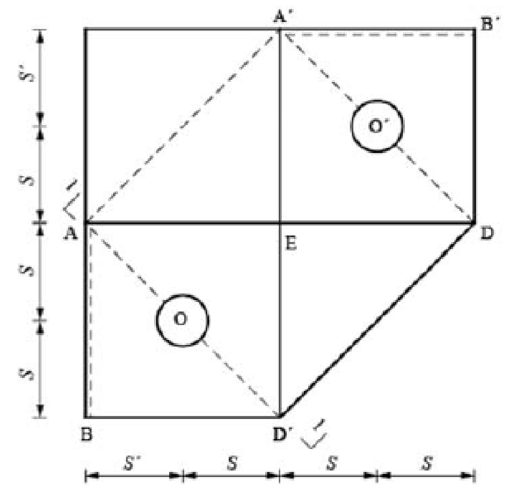
Fig. 2.18-Baseplates of Extra High Voltage (EHV) transmission towers (Liu et al., 2018)



$$\frac{T}{f_y t^2} = 2[\frac{a+S1-d/\sqrt{2}}{2S+S1-a} + \frac{L_{AO}{}^2 - L_{AO}d/2}{S(2S+S1-a)} + \frac{S+S1+L}{S} - \frac{a}{S} - \tan\beta + \frac{L_{BO}{}^2 - L_{BO}d/2}{LS} + \frac{S}{L\cos^2\beta} + \left(\frac{S1+L}{\cos\beta+\sin\beta} - d/2\right)/(L\cos\beta)]$$

Case 1 – Rigid EBBP connections

Fig. 2.19 – Design formula for rigid EBBP connections (Liu et al., 2018)



$$T = \left[\left(10 - \sqrt{2}\frac{d}{S}\right) f_y t^2 + \frac{2 f_{y,s} {h_s}^2 t_s}{S} + \frac{2\sqrt{2}}{3}\frac{f_{y,b} d^3}{S} \right] \Big/ \left(1 + \frac{\sqrt{2}}{3\pi}\frac{d}{S}\right)$$

Case 2 – Flexible EBBP connections

Fig. 2.20 – Design formula for flexible EBBP connections (Liu et al., 2018)

2.5 Summary

In this research, two main areas highlighted in literature review have been earmarked for further study. One area is assessing structural performance of the existing telecommunication/broadcasting towers in the event of probable earthquakes in the region. Detailed analytical research based on regional seismic parameters and sub soil conditions have not been carried out previously. Further, seismic checks are rarely carried out in design stage of the towers. Most of tower codes used in the region have not provided a specific approach. However, considerable part of South Asia is constantly under threat of earthquakes and even countries like Sri Lanka, where it has lesser seismic threat, may be subjected to a minor to moderate level earthquake in any time. Hence, assessment of seismic performance of telecommunication towers in the region is a timely need. Eventhough, there are some studies carried out on structural performances telecommunication/broadcasting towers in global context, findings of such studies are not directly applicable in regional context as those studies have been carried out based on local seismic codes and parameters.

Baseplates of a self supporting lattice tower would be subjected to uplift forces due to design lateral loads (wind or seismic) unlike in the case baseplates of buildings, where uplift is extremely rare. However, specific guideline for design tower baseplates is not given in any of popular tower design codes. Design guidelines given in building design codes and text books are not directly applicable here as uplift load cases govern the design. Eventhough few research publications are available regarding the design of baseplates of buildings and certain tower structures under uplift forces, theories developed in such publications cannot be directly applied to design of common baseplates used for lattice towers as it has unique structural characteristics such as shape of leg members of towers, bolt and stiffner plate arrangements, etc. Development of appropriate design theory for this case would be a fulfillment of essential need in the industry.

Chapter 3 - Seismic analysis of Lattice telecommunication towers

3.1 Seismic analysis procedure

Various codes of practices are available globally for the analysis and design for telecommunication/broadcasting towers. ANSI/TIA-222-G-2005: Structural standard for Antenna supporting Structures and Antenna, EN 1993-3-2 (2006): British Standard Lattice towers and mast and IS 802, 1995: Indian Code of practice for Lattice towers are few examples for tower design codes. In Sri Lanka, ANSI/TIA-222-G-2005: Structural standard for Antenna supporting Structures and Antennas is the most widely used Code of practice for analysis and design of telecommunication/broadcasting towers as there is no local code of practice available.

Therefore, this study is also based predominantly on that code of practice.

For the analysis of towers under seismic loading, four different methods have been given in ANSI/TIA-222-G-2005. Those are:

- Equivalent lateral force procedure
- Equivalent Modal analysis procedure
- Modal analysis procedure
- Time -History analysis

The first two methods of the above are equivalent static methods and the other two are response spectrum and time history analysis procedures, respectively. The height limitations of above analysis procedures are given in Table 3.1 as per ANSI/TIA-222-G-2005.

Table 3.1- Height limitations of analysis procedures (ANSI/TIA-222-G-2005)

Analysis Procedure Method Description [1]	Height Limitations on Analysis Procedure Methods					
	No mass or stiffness irregularities per Table 2-9			With mass or stiffness irregularities per Table 2-9		
	Self-Supporting		Guyed Masts [2]	Self-Supporting		Guyed Masts [2]
	Poles	Latticed		Poles	Latticed	
Equivalent Lateral Force, Method 1 in accordance with 2.7.7	50 ft [15 m]	100 ft [30 m]	No Limit	N/A	N/A	1500 ft [457 m]
Equivalent Modal Analysis, Method 2 in accordance with 2.7.8	No Limit	No Limit	N/A	200 ft [61 m]	600 ft [183 m]	N/A
Modal Analysis, Method 3 in accordance with 2.7.9	No Limit	No Limit	N/A	No Limit	No Limit	N/A
Time-History Analysis, Method 4 in accordance with 2.7.10	No Limit	No Limit	No Limit	No Limit	No Limit	No Limit

Notes:

1. Vertical seismic forces may be ignored for Methods 1, 2 & 3.
2. Method 4 shall be used when the horizontal distance from the base of the structure to any guy anchor point exceeds 1000 ft [305 m].

Modal analysis procedure, which uses Response spectrum technique, was selected as the main procedure for seismic analysis in this study. Finding of time history data for a Time history analysis is not quite practical especially for Sri Lankan condition due to lack of seismicity data. For the comparison of results of Response spectrum analysis, two equivalent static methods have been used as appropriate.

3.1.1 Development of response spectrum curves

The equation given in ANSI/TIA-222-G to develop response spectrum is as follows:

$$T_m = 1/f_m \quad (3.1)$$

For $T_m < 4.0$ sec,

$$S_{am} = S_{DS}(0.4+0.6T_m/T_0) \text{ when } T_m <= T_0 \quad (3.2)$$

$$S_{am} = S_{DS} \text{ when } T_0 < T_m < T_s \quad (3.3)$$

$$S_{am} = S_{D1}/Tm \text{ when } T_m >= T_s \quad (3.4)$$

For T_m>= 4.0 sec

$S_{am} = 4S_{D1}/T_m^2$ (3.5)

$T_0 = 0.2S_{D1}/S_{DS}$ (3.6)

$T_s = S_{D1}/S_{DS}$ (3.7)

Also,

$S_{DS} = 2/3\ F_aS_S$ (3.8)

$S_{D1} = 2/3F_v\ S_1$ (3.9)

Where,

f_m = Frequency of structure for the mode under consideration

S_{am} = Design spectral response acceleration at period T_m for the mode under investigation

S_{DS} = Design spectral response acceleration at short period

S_{D1} = Design spectral response acceleration at period of 1.0 second

S_1 = Maximum considered earthquake spectral response acceleration at 1.0 second

S_s = Maximum considered earthquake spectral response acceleration at short period

F_a = Acceleration based site coefficient based on site class and spectral response acceleration at short period

F_v = Velocity based site coefficient based on site class and spectral response acceleration at 1second

The graphical representation of the response spectrum curves as per above parameters are given in Fig. 3.1.

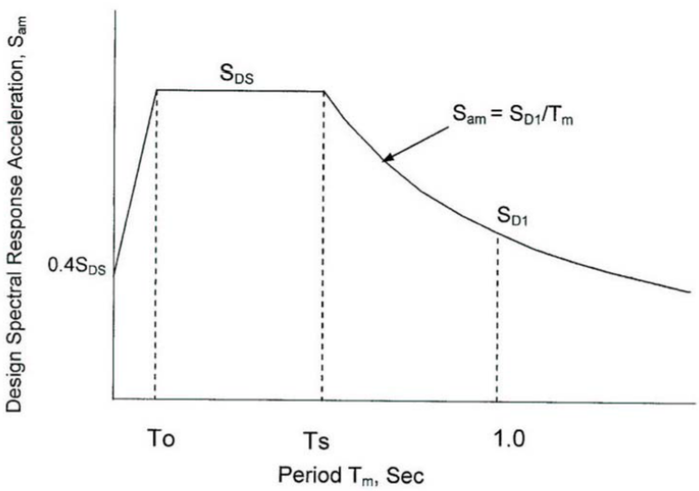


Fig. 3.1 – Response spectrum curve given in with ANSI/TIA-222-G with relevant parameters

Out of the above parameters, S_s and S_1 are site specific seismic spectral acceleration parameters essential for analysis. As discussed in Section 2.2.1, PGA value of 0.1g was selected as the design PGA value for Sri Lanka. However, as per ANSI/TIA-222-G, the Response spectrum curve required for analysis cannot be directly developed using design PGA value and site specific S_s and S_1 values have to be found. An approximate method has been published by Global Seismic Hazard Assessment Programme (GSHAP) to calculate S_s and S_1 based on estimated PGA value of 10% probability of exceedance in 50 years (www.usgs.gov). As per the method:

$$S_s = 5.0 \times \text{Estimated PGA value of 10\% probability of exceedance in 50 years} \tag{3.10}$$

$$S_1 = 2.0 \times \text{Estimated PGA value of 10\% probability of exceedance in 50 years} \tag{3.11}$$

Hence, if PGA value of 0.1 g is considered, S_s and S_1 are 0.5 and 0.2, respectively for the Sri Lankan condition. However, a well recognized US Geological Survey (USGS) website (www.usgs.gov), which publishes these data for all major cities and countries of the world, has specified values of 0.03 and 0.01 respectively for S_1 and S_s. This may be based on the lack of seismicity data in and around Sri Lanka. Therefore, S_s and S_1 values calculated based on 0.1 g PGA was used to represent the local conditions.

As this study is focused to South Asian region under a broad scope, two fairly high seismicity conditions were also considered for the analysis. Accordingly, S_s = 2.14, S_1 = 0.86 (Equivalent PGA is about 0.43 g) and S_s = 1.22, S_1 = 0.49 (Equivalent PGA is about 0.25 g), which represent Nepal and Pakistan, respectively, in USGS website (www.usgs.gov) were selected. In contrast to the previous case, these values are identical with specified design PGA values of local seismic codes and design guidelines of respective countries. In regional context, values relevant to Nepal represent the highest seismicity condition and Pakistan values represent a moderate to severe seismicity condition. Seismicity of other countries of the region such as India, Bhutan and Bangladesh are also covered by these values and considerable variation of seismicity can be observed within those counties from moderate to severe conditions. Situation of Maldives is more or less same to the condition of Sri Lanka.

F_a and F_v are parameters that depend on sub soil characteristics of the location. Effects of sub soil characteristics on seismic forces are quite crucial in certain cases. As an example, in the catastrophic earthquake took place in Kathmandu in 2015, amplification of seismic waves due to existence of thick soft sediments in the Kathmandu basin is reported by Goda et al., 2015. Six different site classes are specified in ANSI/TIA-222-G and those classes are given in Table 3.2.

Table 3.2 – Details of soil conditions

Site Class	Description of upper 30.5 m of soil for the site location
A	Hard rock with 10 ft (3 m) or less of soil overburden
B	Competent rock with moderate fracturing and weathering with 10 ft (3 m) or less of soil overburden
C	very dense soil, soft rock or highly fractured and weathered rock (SPT N >50)
	stiff soil (15 < SPT N < 50)
E	Weak soil -soil profiles over 3 m thick PI >= 30, moisture content >= 40%, S_u < 25 kPa
F	Soil vulnerable to potential failure or collapse under seismic loading

Considering above site classes, appropriate F_a and F_v values were obtained from Table 2-12 and 2-13 of ANSI/TIA-222-G as per the design spectral acceleration values of respective cases.

3.2 Seismic analysis of four leg and three leg towers under normal soil condition

3.2.1 Analytical approach

As discussed in Chapter 2, since self supporting towers are the predominant structural form for towers by fair margin, this study mainly focused on those towers. Accordingly, three and four leg towers having heights of 30 m, 50 m and 80 m (which covers the most common height range of four leg self supporting towers in the country from 30 m to 50 m) were selected to carry out a case study. Line diagrams which show the structural data of those towers are given in Annexure 1.

Similarly, to study about three leg self supporting towers, three towers having heights of 30 m, 45 m and 60 m were selected and Line diagrams of those towers are also given in Annexure 1. Usually three leg towers are not used in situations needing greater heights in Sri Lanka and a three leg tower taller than 60 m is a rare case.

The summery of tower data used for this analysis are given in Table 3.3.

Table 3.3 – Design details of considered towers

Tower height (m)	Type	Design windshield area of tower
80	Four leg	18 m^2 at 80 m level and 9 m^2 at 60 m level
50	Four leg	16 m^2 at 50 m level and 10 m^2 at 30 m level
30	Four leg	10 m^2 at 30 m level and 6 m^2 at 20 m level
60	Three leg	10 m^2 at top most level
45	Three leg	10 m^2 at top most level
30	Three leg	6 m^2 at top most level

SAP2000, which is popular commercial Finite element analysis software, was used to model towers. All structural members of towers including redundant members were incorporated for towers models. Weights and masses (especially for Response spectrum analysis) of non structural elements such as cable ladders, resting platforms, guset plates, connecting plys, nut & bolts, antenna accessories are very important as those have significant contribution to the actual weight and mass of a tower. Hence, according to the actual details of respective towers, above mentioned additional weights and masses were calculated and those were applied to the towers by modifying material density or unit mass of structural members and by adding masses to the SAP2000 model as required. A similar method has been adopted by Amir et al., 2002 in their study on telecommunication towers.

To execute the Response Spectrum analysis of towers, three (03) PGA values (0.1 g, 0.25 g and 0.43 g) discussed in Section 3.2.1 were selected to represent low to sever seismic condition in South Asian region. For the initial analysis, Soil class C was selected as the sub soil condition as in most of the cases (as telecommunication and broadcasting towers are usually located at higher ground elevations for better coverage) towers are erected on very dense soil or soft rock (Refer Table 3.2).

Parameters used to develop Response spectrum curves under above three (03) cases are presented in Table 3.4.

Table 3.4- Seismic parameters considered for development of Response Spectrum Curves

Case	**Relevant country**	**S_s**	**S_1**	**considered soil class**	**F_a**	**F_v**	**Equivalent PGA value (g)**
Case 1	Sri Lanka	0.50	0.20	Class C	1.00	1.00	0.10
Case 2	Pakistan	1.22	0.49	Class C	1.00	1.30	0.25
Case 3	Nepal	2.14	0.86	Class C	1.00	1.30	0.43

Response Spectrum curves developed using above parameters are given in Fig. 3.2.

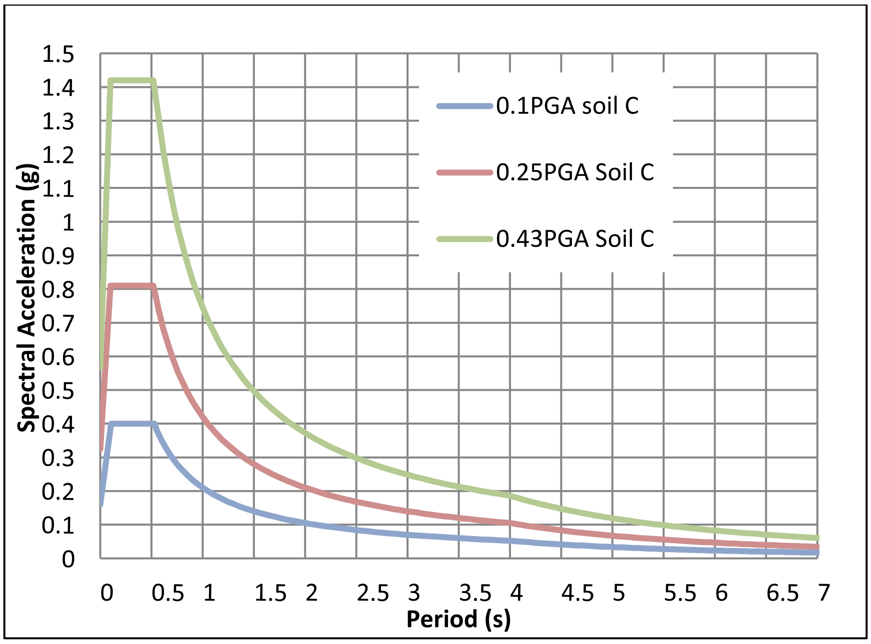


Fig. 3.2 – Response spectrum curves under soil Class C

To compare the results of Response Spectrum with Static Equivalent analysis given in ANSI/TIA-222-G, equivalent static forces of each tower under each case were also calculated using same seismic and soil parameters relevant to respective cases. Then, the calculated equivalent forces distributed throughout the height of the respective towers based on segments defined according to tower geometries as per the specifications given in ANSI/TIA-222-G and applied to the respective SAP2000 models as nodal loads. Equivalent modal analysis method (Method 2) was used for all towers for static equivalent analysis as that method could be used for any self supporting tower without a height restriction. (In case of method 1, it can only be used for towers having heights less than 30 m in case of self supporting towers).

During Response Spectrum analysis, sufficient numbers of mode shapes were always considered for each case to satisfy the code's requirement of achieving at least 85% combined modal mass participation. Accordingly, under Four leg tower case, first 12 modes were considered for 30 m and 50 m towers and first 30 modes were considered for 80 m tower. In three leg tower case, the required modal participation of all three towers (30 m, 45 m and 60 m) were achieved by considering first 12 modes.

Comparison of seismic analysis results with wind analysis results is important as usually telecommunication/broadcasting towers are designed considering wind forces. Hence, each tower was analyzed under two wind speeds given in Table 3.5 selected as lower and upper bound values (based on regional wind data discussed in section 2.4) even though certain areas of South Asia have higher design wind speeds than 50 m/s due to certain geographic and meteorological conditions of such areas.

Table 3.5 – Wind speed data

Case	Considered 3s Gust wind speed (m/s)
Lower bound value	33
Upper bound value	50

According to data in Table 3.5, the lower bound value happens to be the lowest design wind speed that can be considered for structural designs in three countries namely Sri Lanka, Pakistan and India in South Asian region. In case of upper bound value, wind speed of 50 m/s is the highest wind speed that could be considered for Normal structures in Sri Lanka (in Zone 1) and it is the second highest design wind speed for India and Nepal. Therefore, these two wind speeds can be considered as good selections to compare seismic analysis results with Wind analysis results in regional context.
The wind forces on tower body and antennas were calculated as the specifications of ANSI/TIA -222-G. In case of calculation of wind loads on a tower body, tower is divided into segments and loads are calculated based on solidity ratios of respective segments. Finally, calculated loads are applied as nodal loads to the tower model. The

wind load on antennas and accessories were separately calculated and applied to respective nodes in same way.

Following load combinations under each wind and earthquake analysis cases were considered as per the specification of ANSI/TIA-222-G.

For four leg towers:

1. 1.2 D + 1.6 W (in direction perpendicular to a face)
2. 1.2 D + 1.6 W (in diagonal direction)
3. 0.9 D + 1.6 W (in direction perpendicular to a face)
4. 0.9 D + 1.6 W(in diagonal direction)
5. 1.2 D + 1.0 E (in direction perpendicular to a face)
6. 1.2 D + 1.0 E (in diagonal direction)
7. 0.9 D + 1.0 E (in direction perpendicular to a face)
8. 0.9 D + 1.0 E (in diagonal direction)

For three leg towers:

1. 1.2 D + 1.6 W (in direction perpendicular to a face)
2. 1.2 D + 1.6 W (in direction having angle of 30^0 from a face)
3. 1.2 D + 1.6 W (in direction parallel to a face)
4. 0.9 D + 1.6 W (in direction perpendicular to a face)
5. 0.9 D + 1.6 W (in direction having angle of 30^0 from a face)
6. 0.9 D + 1.6 W (in direction parallel to a face)
7. 1.2 D + 1.0 E (in direction perpendicular to a face)
8. 1.2 D + 1.0 E (in direction having angle of 30^0 from a face)
9. 1.2 D + 1.0 E (in direction parallel to a face)
10. 0.9 D + 1.0 E (in direction perpendicular to a face)
11. 0.9 D + 1.0 E (in direction having angle of 30^0 from a face)
12. 0.9 D + 1.0E (in direction parallel to a face)

Under seismic analysis, both Response Spectrum analysis and Static equivalent analysis were performed under each load combination with respect to three selected seismicity conditions. In case of wind, each of above load combinations were considered under selected wind speeds of 33 m/s and 50 m/s.

3.2.2 Results and discussion

At the end of the analysis, results of following under each load combination of the selected towers were extracted as those are the most critical effects that usually governs the design:

1. Maximum Leg member forces - compression
2. Maximum Leg member forces - tension
3. Maximum Support reaction - compression (downward)
4. Maximum Support reaction - tension (uplift)
5. Maximum lateral deflection

The maximum case with respect to each seismicity level and two wind speeds were identified and converted to graphs. Locations where maximum leg member forces were reported are shown in Fig. 3.3 (indicted by red colour ovals).

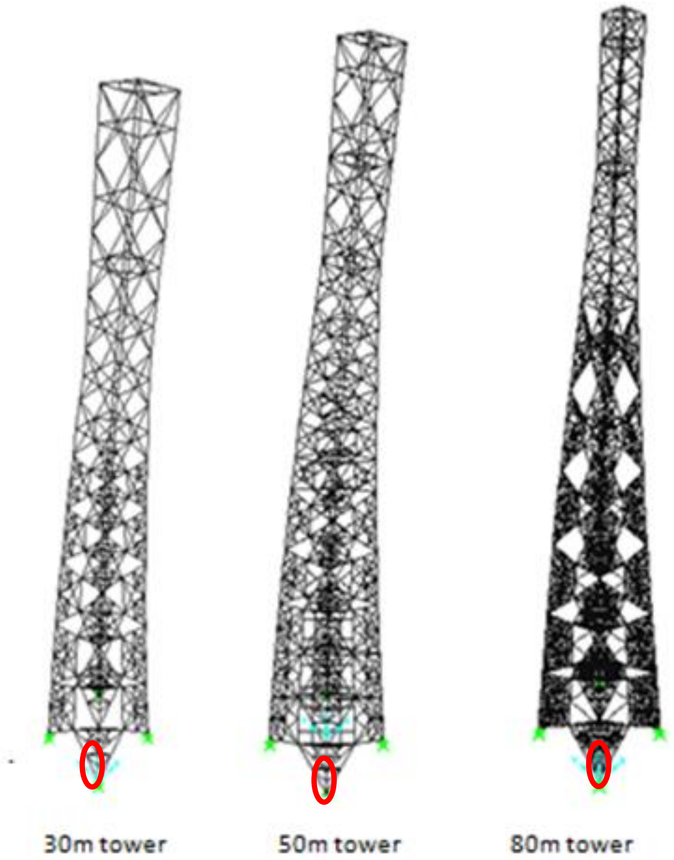
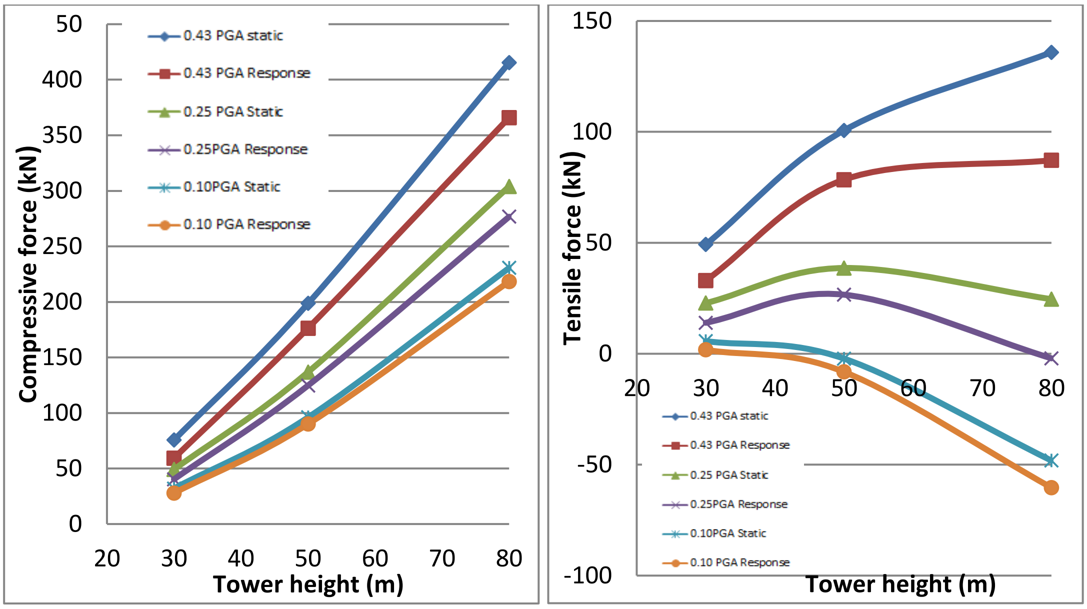


Fig. 3.3 - Locations were maximum leg member forces were reported

Fig. 3.4 shows the maximum leg member force in compression and maximum leg member force in tension of four leg towers. When compared results of Response

Spectrum analysis results with Static equivalent analysis results of a particular seismicity level, forces observed under Static equivalent analysis are high in every tower under every seismicity level. It indicates that the Static equivalent analysis specified in ANSI/TIA-222-G would give conservative results.

Results relevant to equivalent PGA of 0.1 g indicate that seismicity effect under this seismicity level does not able to exceed the forces due to self-weight in heavy towers (50 m and 80 m) and therefore, negative tensile (compressive) forces are observed in Fig. 3.3 (b) in those towers. However, under higher seismic hazard levels, considerable tensile forces can be induced in tower legs.

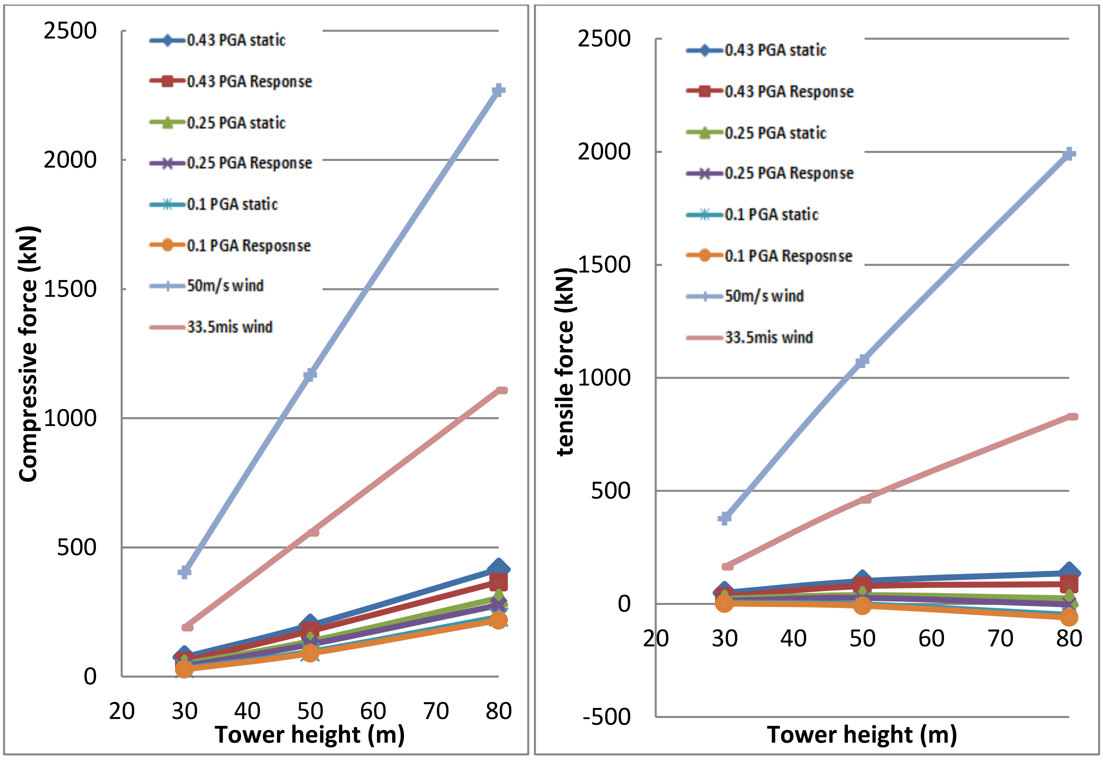


a) Compressive force *b) Tensile force*

Fig. 3.4 – Compressive and Tensile force in leg members under seismic loading with soil class C in four leg towers

As per results, numerical deviations of static equivalent analysis results from Response spectrum analysis results have increased with the height of the tower. The probable reason for this may be effects of higher order vibration modes considered for response spectrum analyses to satisfy the mass participation ratios in taller towers. This is further discussed in Section 3.3.1. When compression and tensile member force results are compared for Response spectrum analysis and Static Equivalent analysis of one particular tower under a particular seismic hazard level, those are almost the same. This is understandable as effect due to any lateral force on this type of structure in compression and tension sides should be same due to axisymmetry. However, trends in tension and compression sides are different to each other due to the effect of self-weight of towers. Usually, with the increase of tower height, self weight of towers increase significantly. However, the seismic forces do not increase in the same magnitude and hence the trends that can be observed in Figure 3.4 (b) occurs.

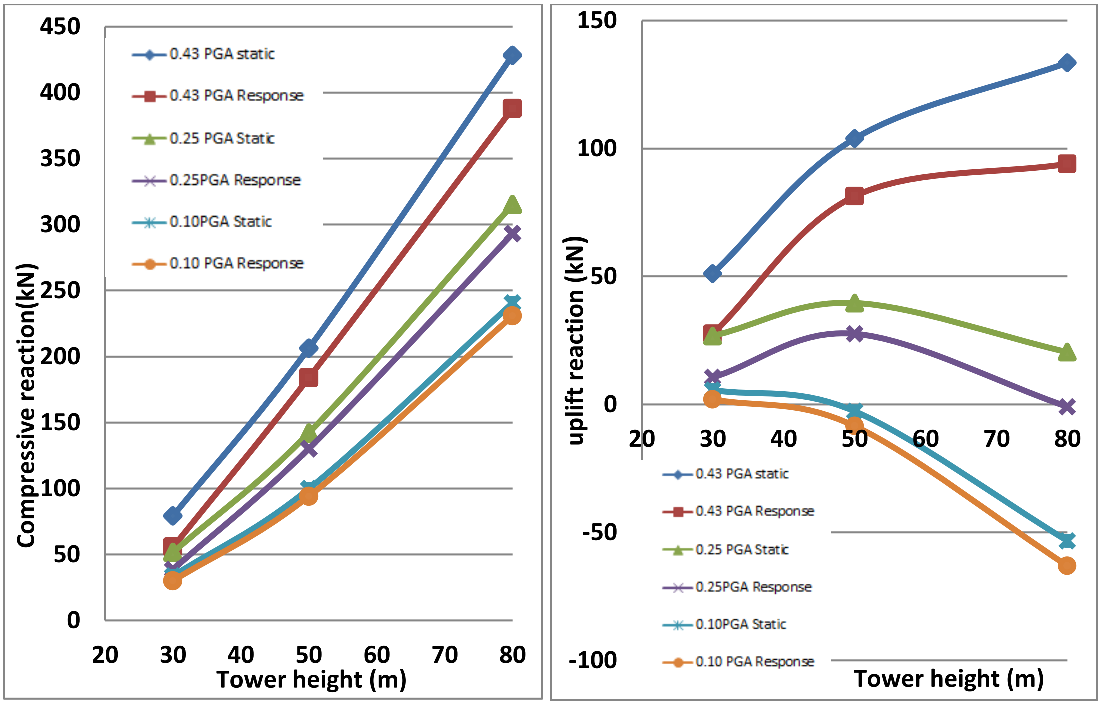
Fig. 3.5 shows the same results including wind analysis results. Dominancy of forces under wind loads over seismic loads can be clearly observed. Even under lowest wind speed of 33m/s case, forces under wind are dominating by clear margins over forces under seismic loads even under highest seismicity level consider in analyses (0.43 g PGA). The gap between wind and seismic analysis results are widening with the increase of height of towers and the main reason seems to be the rapid increase of wind loads with height of towers.



a)Compressive force *b)Tensile force*

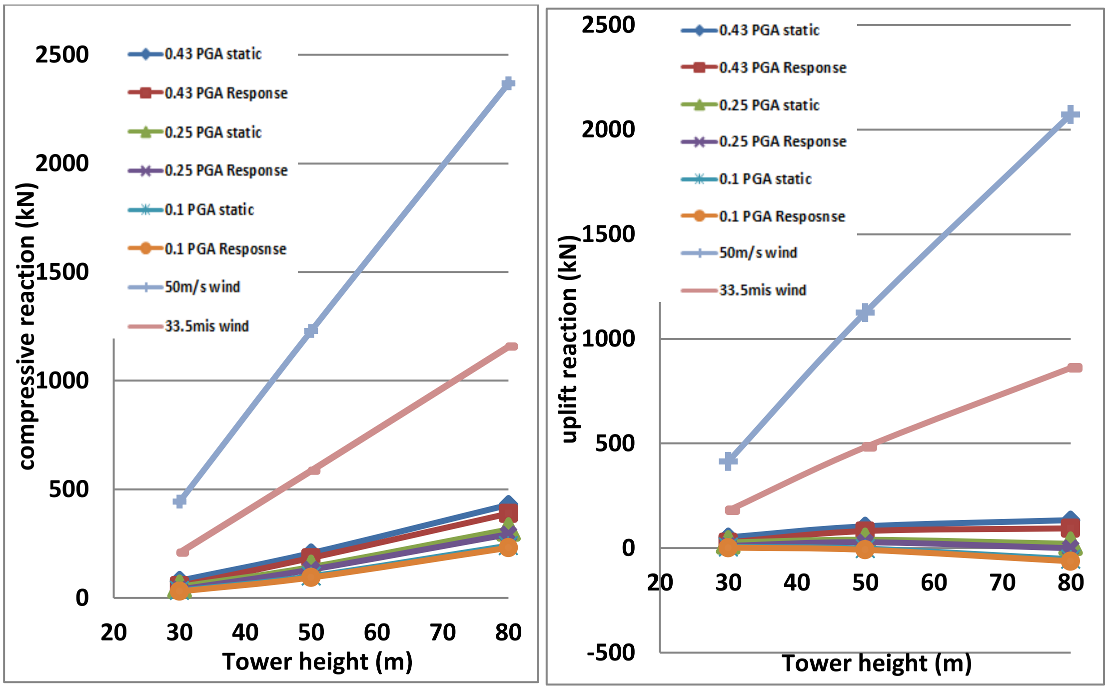
Fig. 3.5 – Compressive and Tensile force in leg members under wind loading & seismic loading with soil class C in four leg towers

Fig. 3.6 and Fig. 3.7 show the results for support reactions. In Fig, 3.6, results only under seismic load cases have been plotted and in Fig. 3.7, wind analysis results are also added for comparison. Trends in support reactions are almost identical to the leg member forces results.



a) Compressive Reaction *b) Uplift Reaction*

Fig. 3.6 – Compressive and Uplift reactions under seismic loading with soil class C in four leg towers

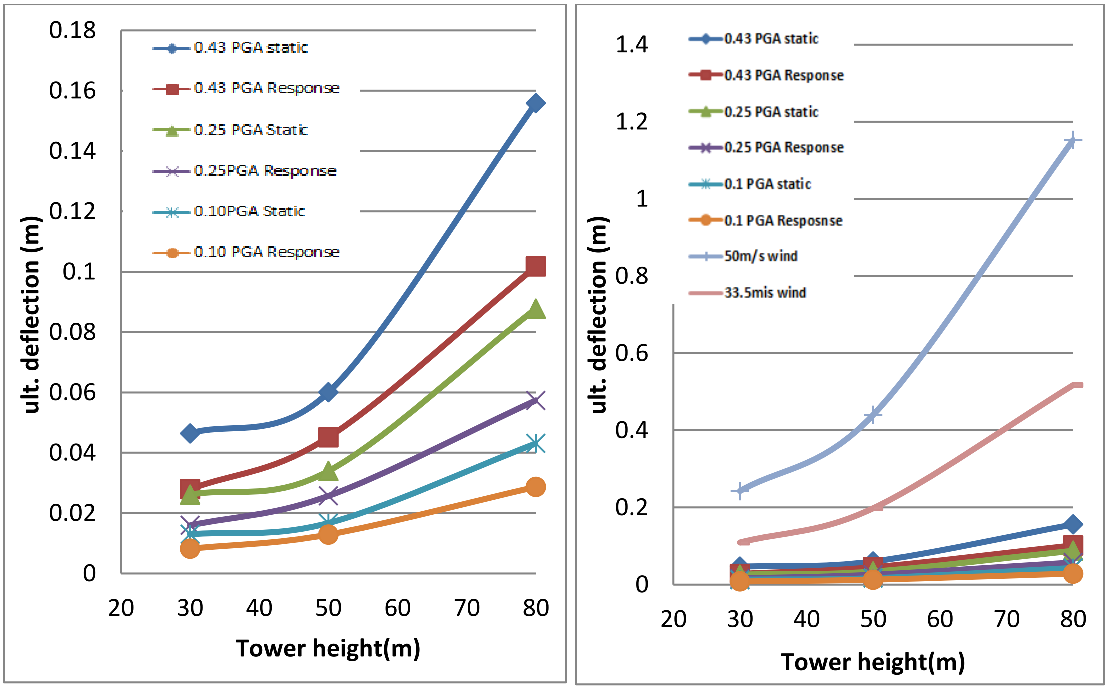


a) Compressive Reaction *b) Uplift Reaction*

Fig. 3.7 – Compressive and Uplift reactions under wind & seismic loading with soil class C in four leg towers

In Fig. 3.8 (a) and Fig.3.8 (b), variation of ultimate deflections of towers under seismic load cases and both seismic and wind load cases are shown respectively. However, ANSI/TIA-222-G does not specify to check deflection under serviceability limit state probably as an earthquake is a specific event that would happen once in a while. Hence, violation of serviceability limits under such event would not be an issue if a tower can survive without a structural failure.

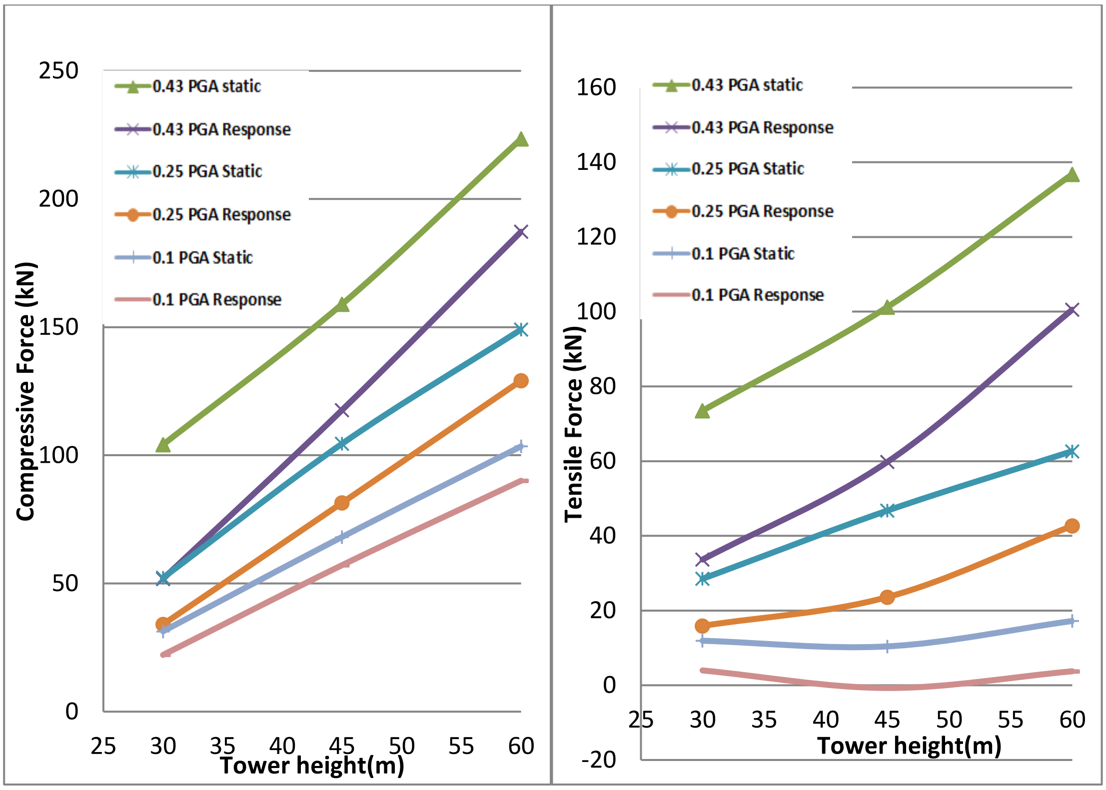
Nevertheless, ultimate deflections under wind and earthquake cases were plotted for a comparison. Significant increase of deflection under each seismicity and wind load case is visible with increase of towers height. Static Equivalent results have dominated over Response Spectrum results as usual and conservative nature of Static Equivalent analysis is evident here as well. As can be seen, deflections under wind loads are much higher than seismic loads.



a) Only with seimic loadngs *b) Including wind loads*

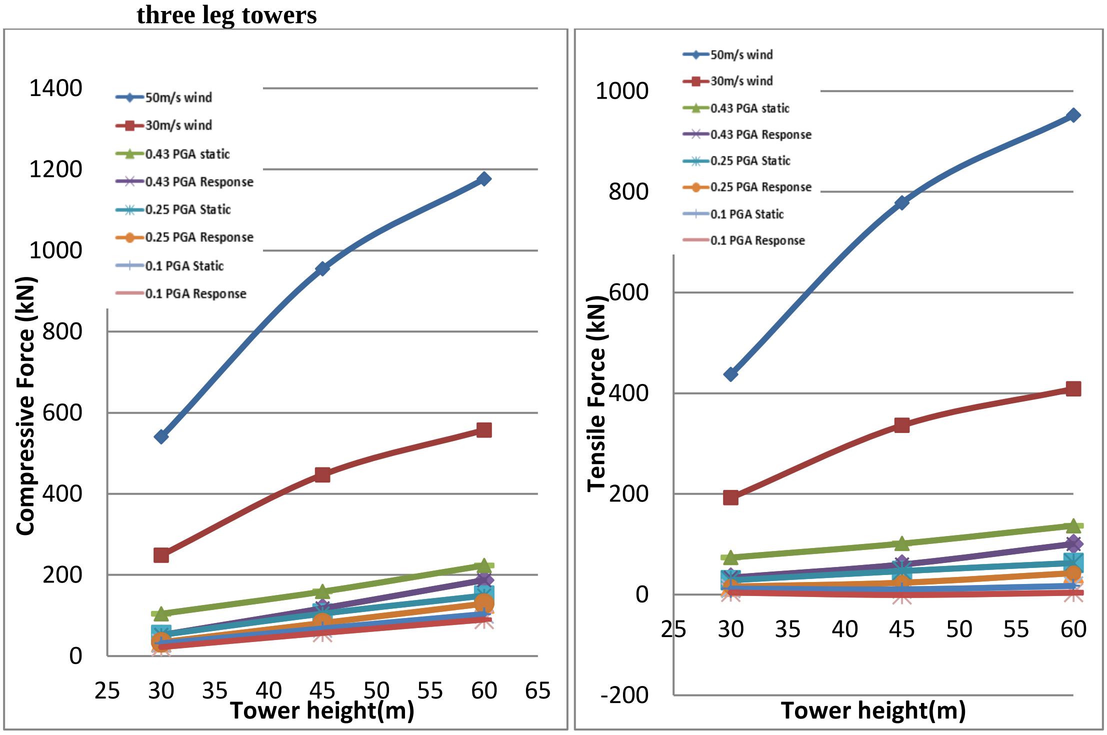
Fig. 3.8 – Ultimate deflections under wind & seismic loading with soil class C in four leg towers

Fig. 3.9 and Fig. 3.10 show maximum compression and tensile leg member forces of three leg towers under seismic load cases. Static Equivalent analysis results dominate over Response spectrum analysis results in these cases as well. However, trends in curves with respect to tower heights are somewhat different to patterns observed in four leg towers. As an example, negative tensile forces are not observed in taller towers as in the case of Four leg towers. The probable reason for this is the effect of self-weights of towers (including antenna and accessories) considered under Four leg and Three leg tower cases. In Fig 3.11, the variation of self-weight of towers considered under Four leg and Three leg cases have been plotted.



a) *Compressive force* b) *Tensile force*

Fig. 3.9 – Compressive and Tensile force in leg members under seismic loading with soil class C in three leg towers



a) *Compressive force* b) *Tensile force*

Fig. 3.10 – Compressive and Tensile force in leg members under wind and seismic loading with soil class C in three leg towers

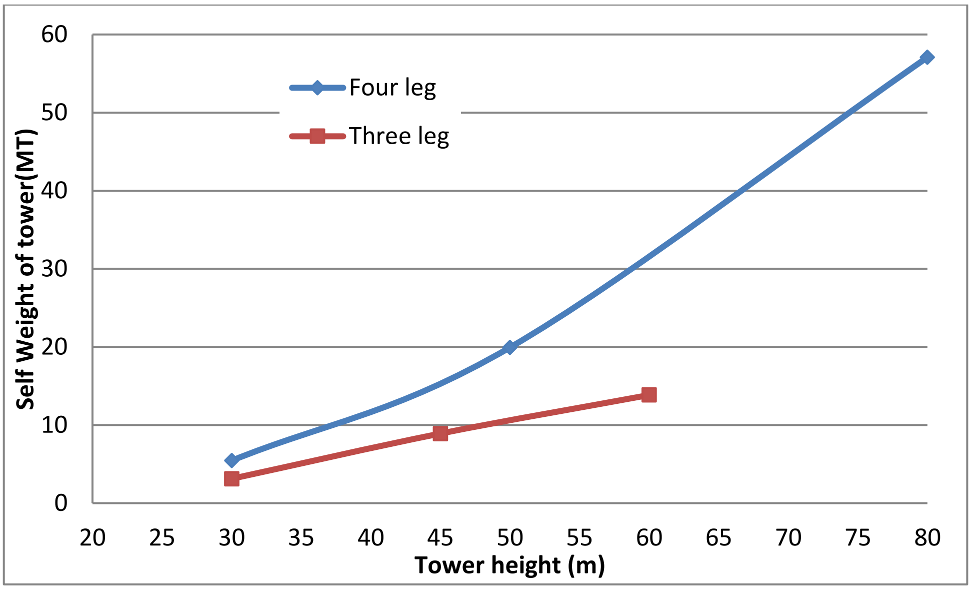
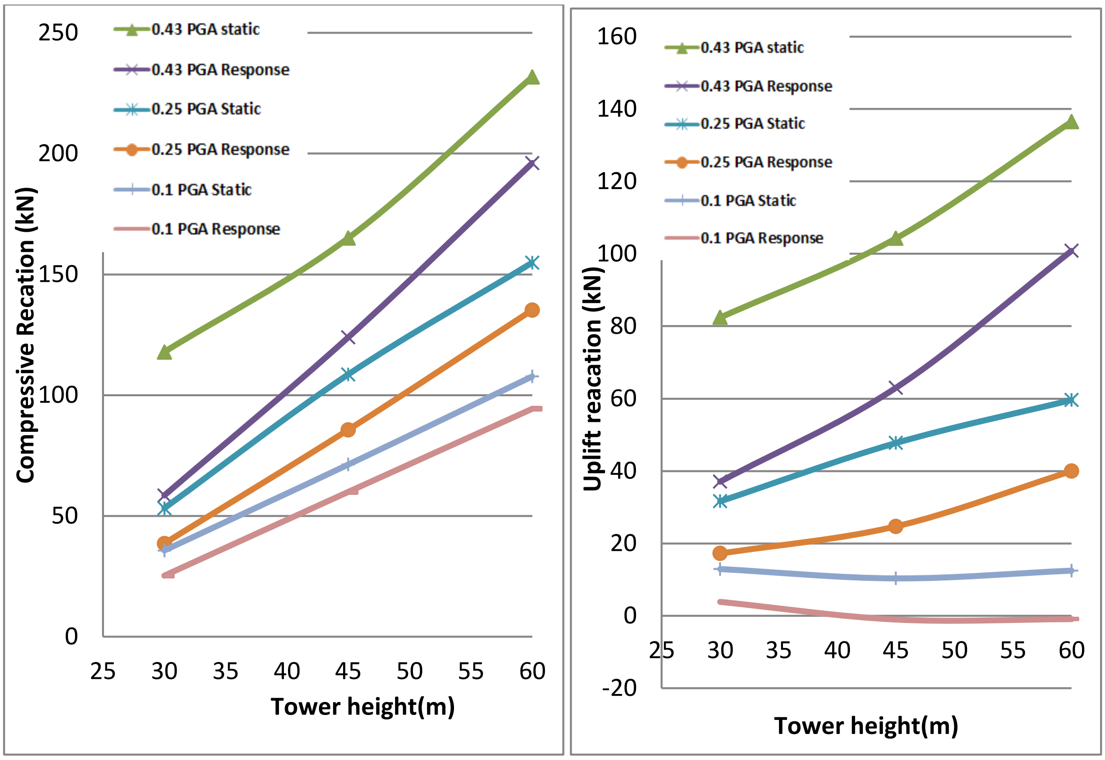


Fig. 3.11 – Variation of self-weight of towers

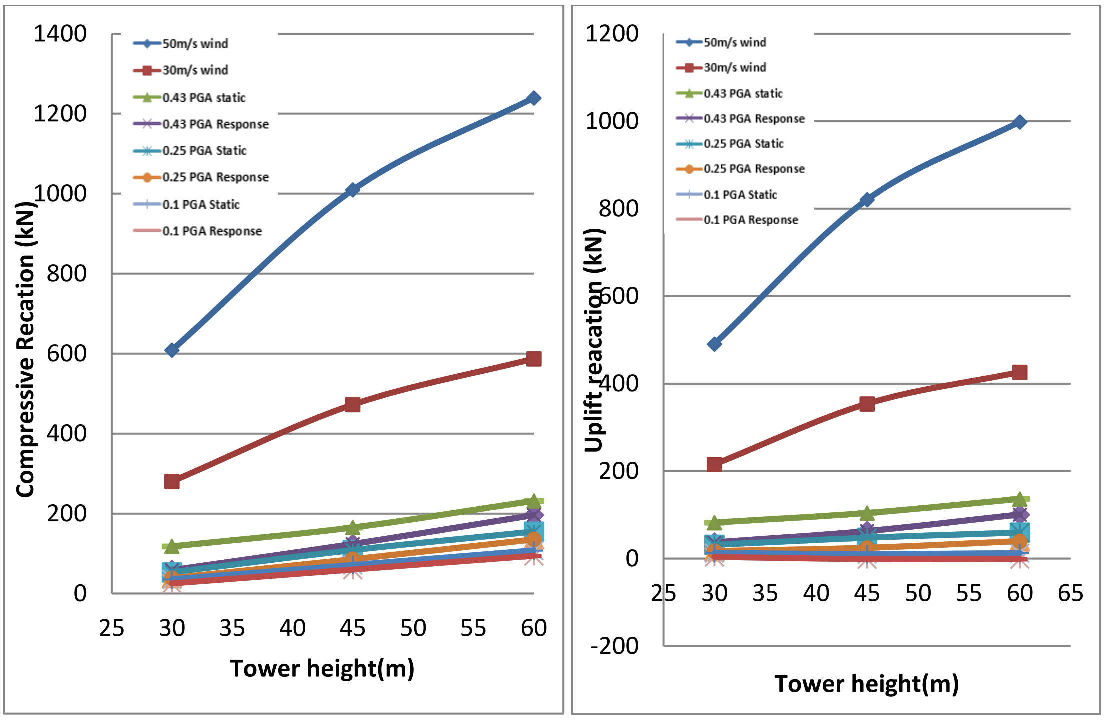
According to Fig. 3.11, self-weight, which can significantly affect to the seismic load, has notably increased with the height of the towers in Four leg tower case. However, the increase of self-weight with the tower height is not that steep in Three leg towers. Self-weight is an inherent characteristic of a tower and it may depend on factors like design wind shield area, tower architecture, etc. Due to comparatively small design windshield capacities of selected three leg towers (see Table 3.3), this variation of self-weight have been experienced. Nevertheless, the member forces under wind load cases dominate over seismic load cases as same as in Four leg tower case (see Fig. 3.10).

Fig. 3.12 and Fig. 3.13 show support reaction results of Three leg towers. Understandably, trends of these curves are very similar to the member force trends of these towers presented in Fig. 3.9 and Fig. 3.10. As explained previously, since actual self-weights of these Three leg tower are low (compared to Four leg towers) uplift reactions are generated even under lowest seismicity level (0.1 g PGA), in contrast to results in Four leg tower case. However, these uplift reactions under seismic loadings are far below of same under wind loading. Hence, dominancy of wind over seismic load is noticeable here as well (See Fig. 3.13).



a) Compressive Reaction *b) Uplift Reaction*

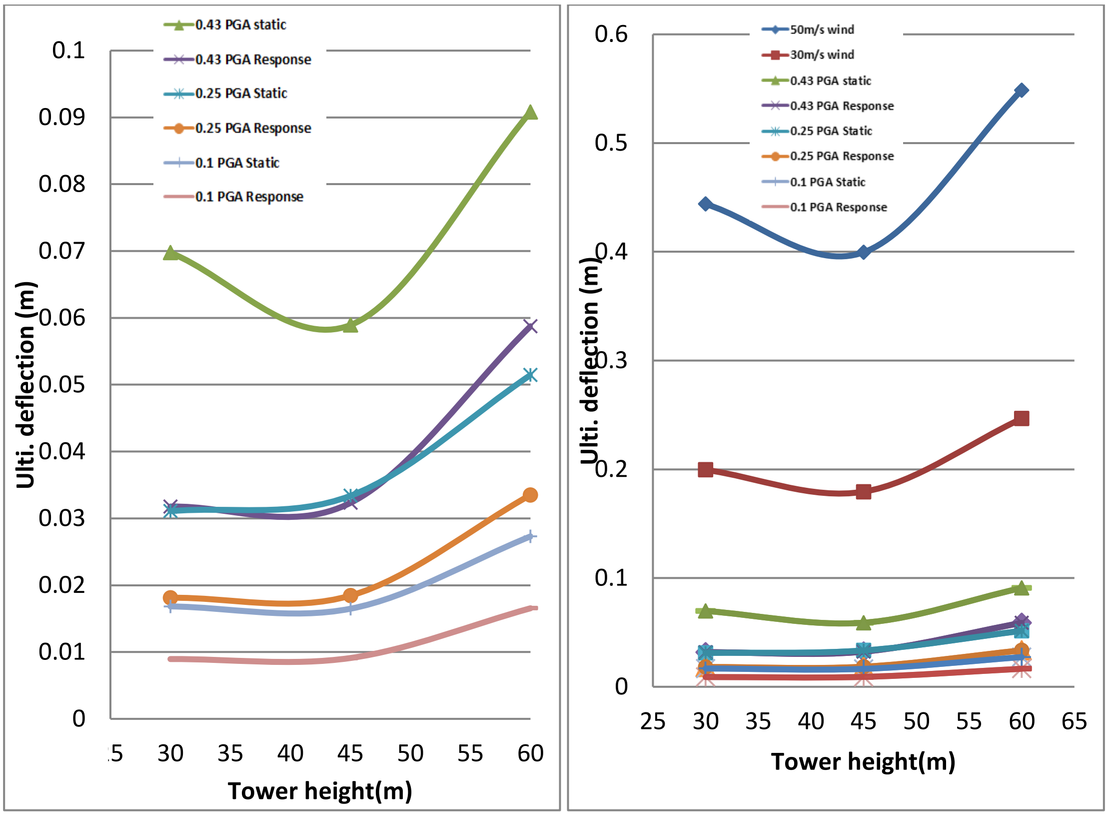
Fig. 3.12 – Compressive and Uplift reactions under seismic loading with soil class C in three leg towers



b) Compressive Reaction *b) Uplift Reaction*

Fig. 3.13– Compressive and Uplift reactions under wind & seismic loading with soil class C in three leg towers

Fig. 3.14 shows the variation of deflections of the three leg towers considered . Trends are compatible with the expected results. However, due to same effect of self-weights of selected towers, pattern of curves are different to Four leg towers.



a) Only with seimic loadngs *b) Includimg wind loads*

Fig. 3.14– Ultimate deflections under wind & seismic loading with soil class C in three leg towers

Therefore, it is clearly evident that self-supporting lattice telecommunication/ broadcasting towers may not face a considerable structural concern under seismic loading, even under highest seismic hazard levels applicable to the region, if such towers are properly designed and constructed according to the applicable regional wind data. This observation is applicable for both three leg and four leg towers. In this study, Soil Class “C”, which is relevant to hard soil stratum, was considered when developing the Response spectrum and equivalent static loading. As there is a tendency to amplify the seismic forces on towers with weaknesses of the sub soil stratum, assessing seismic performances of tower under such soil properties would be required to arrive at the final conclusion in this respect.

In addition to this, forces in main diagonal members were also checked and forces under wind load cases were comparatively high with respect to the seismic load cases in all the cases considered.

3.3 Seismic analysis of Four leg and Three leg towers under different sub soil conditions

3.3.1 Analytical approach

As mentioned in Chapter 2, a tendency of amplifying seismic forces with weak sub soil stratums had been reported. Amplification of seismic waves due to thick soft sediments has been reported as a reason for massive destruction in Katmandu, the capital city of Nepal, during catastrophic earthquake in 2015 (Goda et al., 2015). The presence of soft sediments especially in urbanized cities in South Asia are common as most of cities are located close to rivers or flood plans of rivers. In Sri Lanka, soft soil deposits are common, especially in Colombo and its suburban areas in the Western Province as it is located close to Kelani River. However, this amplification may not affect all structures as vibration characteristics of a structure also have a role in this regard.

Hence, assessing seismic performance of selected Three leg and Four leg towers under different sub soil stratum was identified as an important aspect before reaching a final conclusion on seismic performances of lattice towers under regional conditions. For this assessment, Soil classes “C”, “D” and “E” specified in ANSI/TIA-222-G were selected and Response Spectrum curves were developed under all three (03) seismicity levels considered in Section 3.2.1. Fig. 3.15 shows Response spectrum curves that were developed based on above parameters.

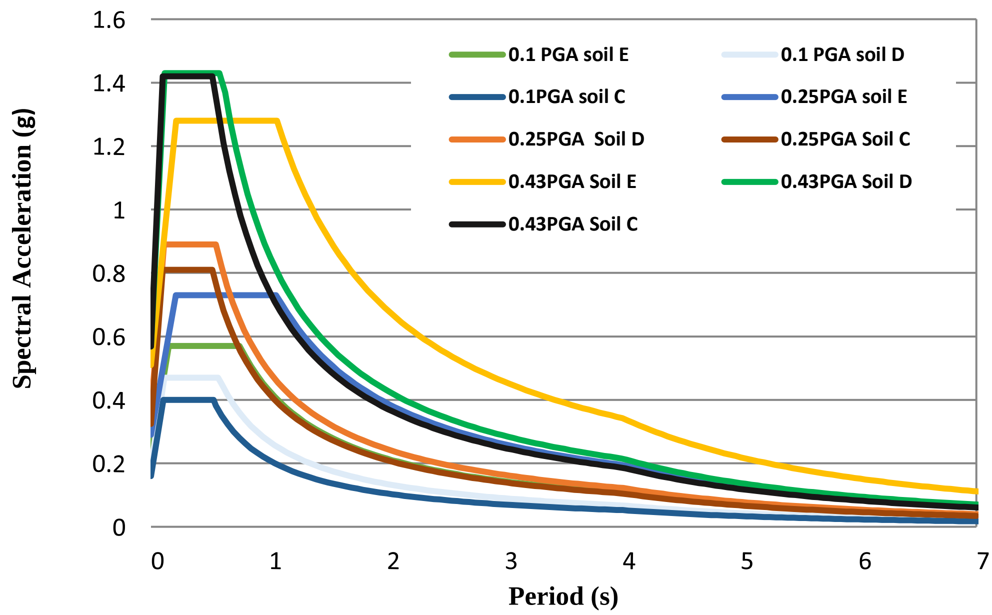


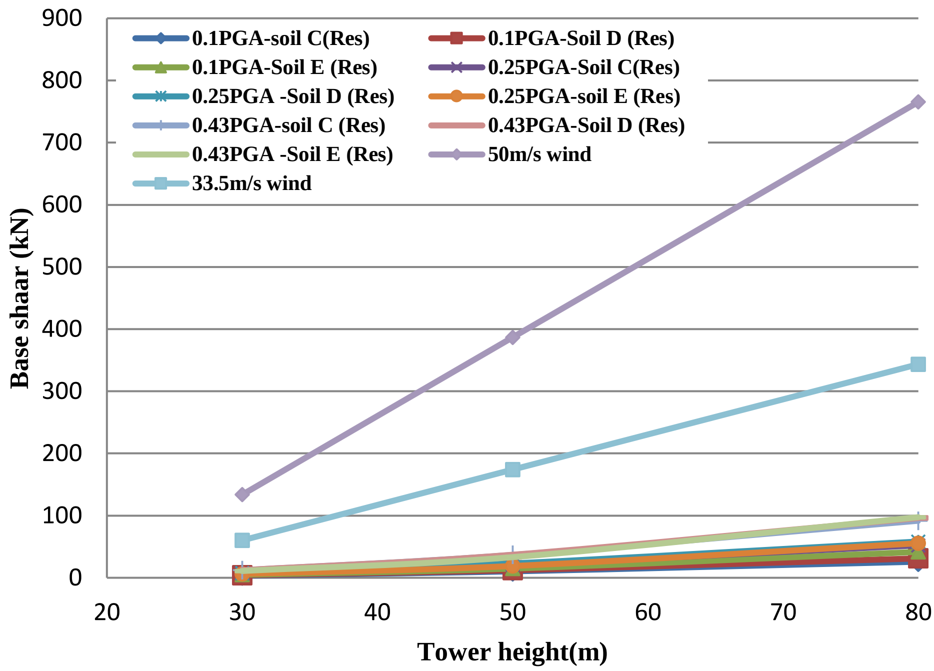
Fig. 3.15- Response spectrum curves under different sub soil conditions

Analyses were carried out using same SAP2000 models developed in previous case and static equivalent analyses were also carried out for the purpose of comparison. Two wind speeds described in Chapter 2 were used to compare these results. According to above response spectrum curves, maximum peak spectral accelerations are dependent on both the soil class and the PGA values considered. Hence, Soil Class “E” did not give the highest values under all considered PGA values. However, in case of the region with respect to longer periods (in the decreasing part of response spectrum curves), spectral accelerations under soil class “E” are always high compared with same of other soil classes under all three considered PGA values.

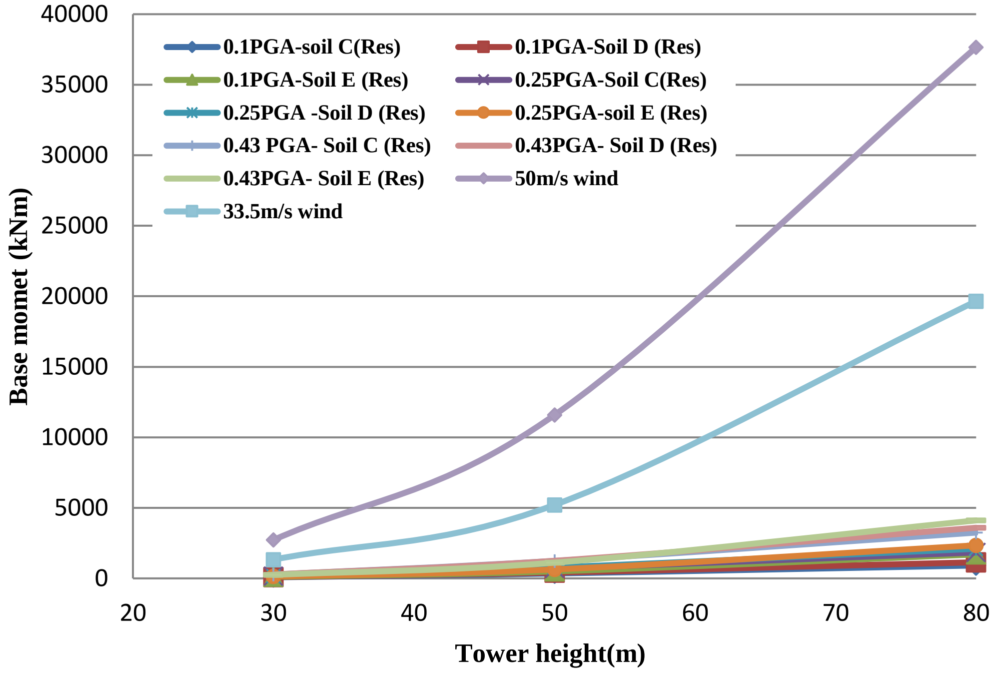
3.3.2 Results and discussion

As the main purpose of this analysis was observing fluctuations of structural actions due to seismic loading with sub soil characteristics, base shear and base moment developed in towers were used to compare the result here. Hence, base shear and base moment of each seismic and wind load cases were compared directly (load combinations were not considered).

Fig. 3.16 and Fig. 3.17 show base shear and base moment values under nine (09) seismicity cases and two (02) wind load cases of Four leg and Three leg towers, respectively. According to these results, dominancy of wind loads over seismic loads is clearly visible. Considerable gap between wind and seismic load cases can be observed in each graph as seen in Section 3.2.2.

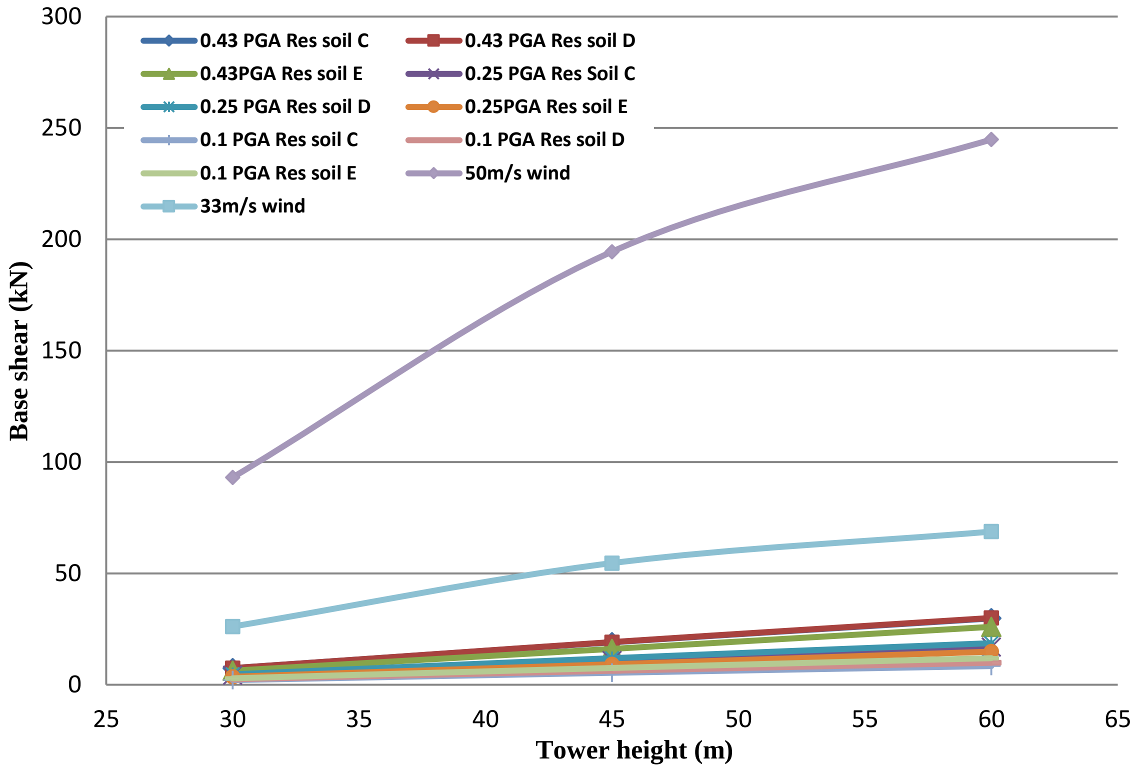


a) Base shear

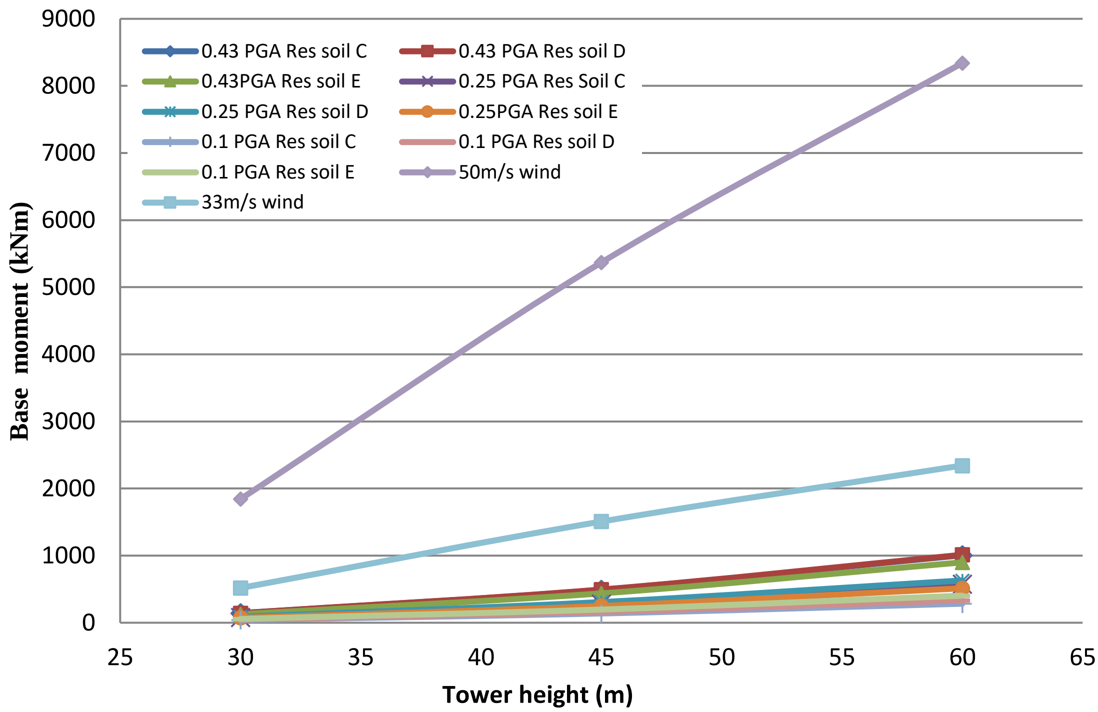


b) Base moment

Fig. 3.16- Base shear and base moment under wind and seismic loading in four leg towers



a) Base shear

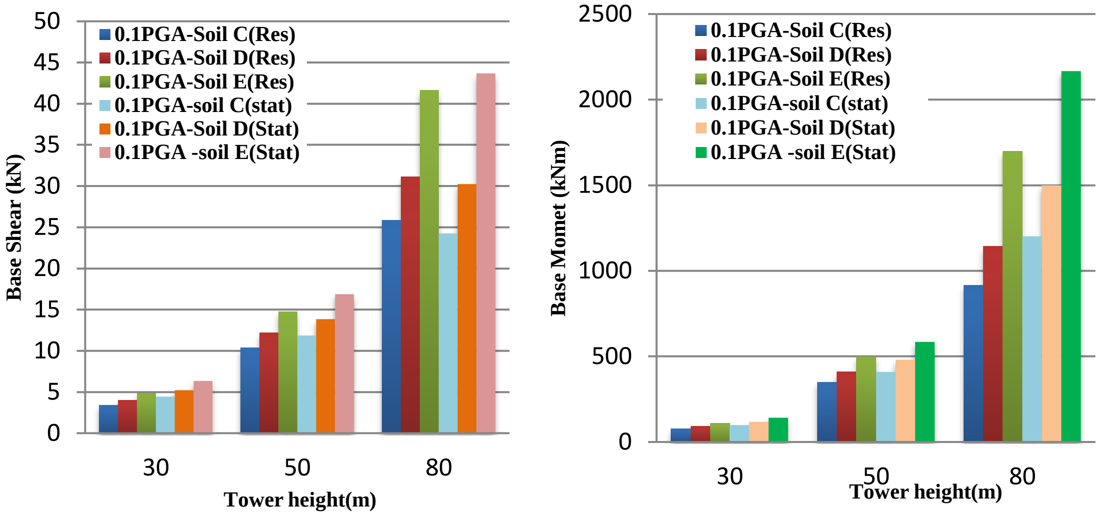


b) Base moment

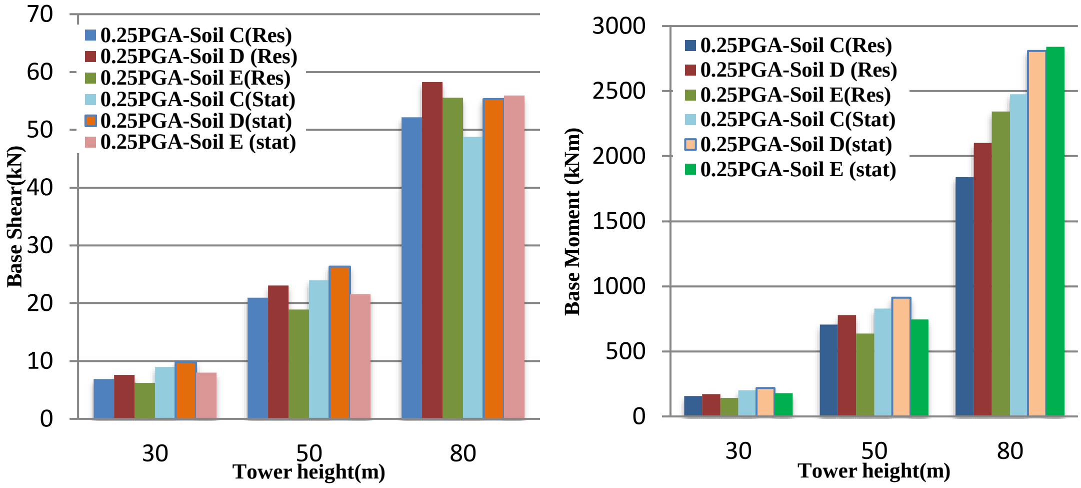
Fig. 3.17- Base shear and base moment under wind and seismic loading in three leg towers

Considerable variation of base shears and base moments of towers have been observed among cases of different subsoil properties. However, these variations are not really visible if wind analysis results are presented with seismic analysis results in same graphs. Hence, for the detailed discussion of effect on sub soil properties, results of Base shear and Base moments under different PGA values with different subsoil properties (without wind analysis results) of Four leg and Three leg towers are presented in bars charts form in Fig. 3.18 and Fig. 3.19, respectively.

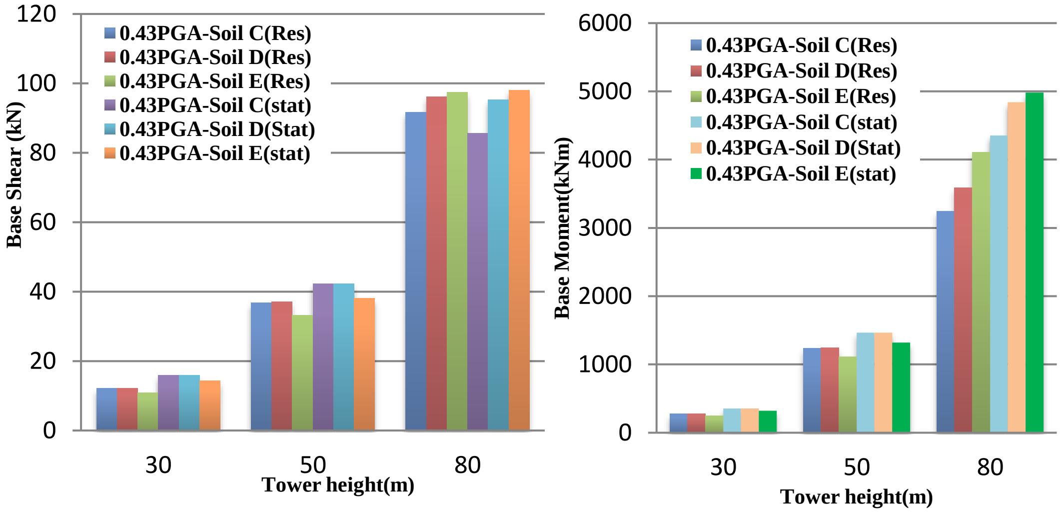
According to results, base shear values of Response spectrum analysis and Static Equivalent analysis of respective cases are very close to each other in all towers under all PGA values and soil conditions. However, base moment results of Static Equivalent analysis considerably deviate from the results of Response Spectrum analysis (see Fig.3.18 and 3.19). This is quite understandable as higher vibration modes would also contribute for seismic response of these types of tall light weight structures under response spectrum analysis. Lump masses of tower models behave as per vibration patterns of modes by causing deviation of Base moment (see Fig. 3.20). The deviation between results of Static Equivalent analysis and Response Spectrum analysis increases with increase of tower heights understandably as more and more modes have to be considered in such taller towers to achieve required mass participation. As an example, in case of 80 m four leg tower, 30 modes were required to achieve required mass participation.



a) Under 0.1 PGA

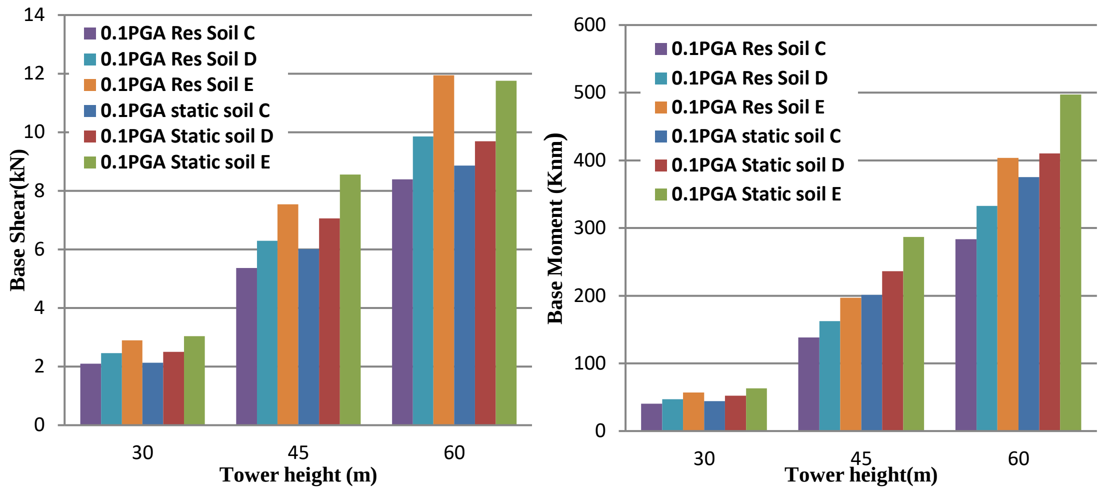


b) Under 0.25PGA

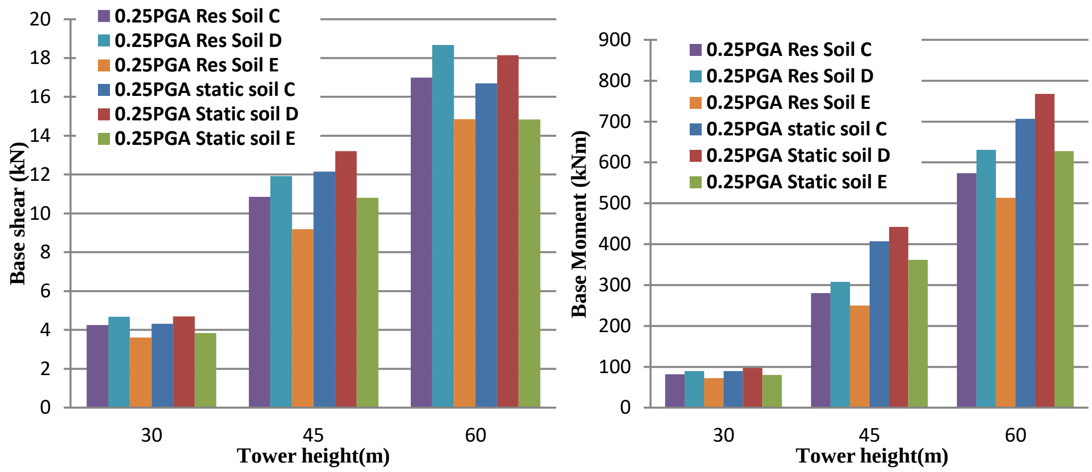


c) Under 0.43 PGA

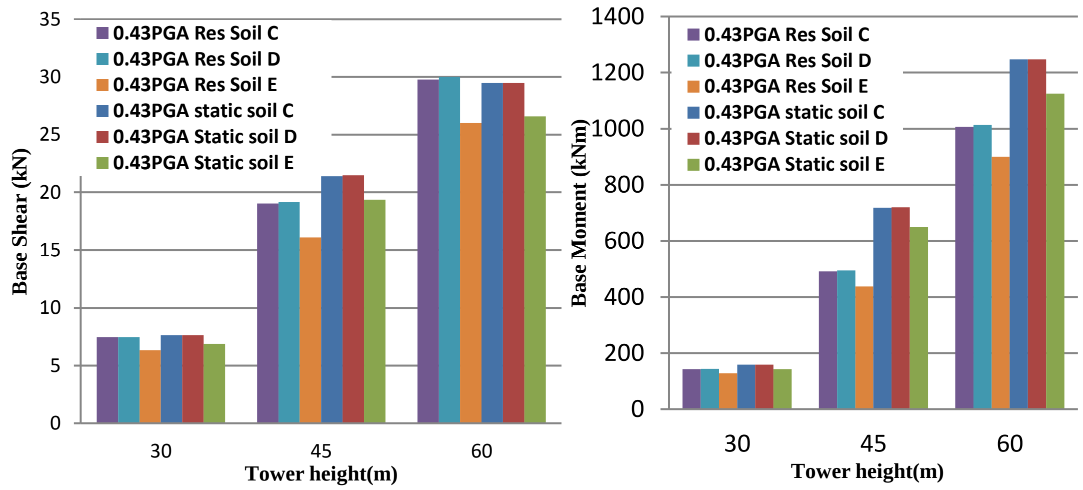
Fig. 3.18- Base shear and base moment variations under different sub soil conditions of four leg towers



a) Under 0.1PGA

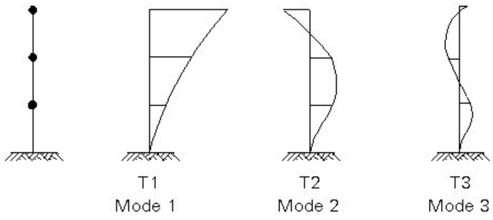


b) Under 0.25PGA

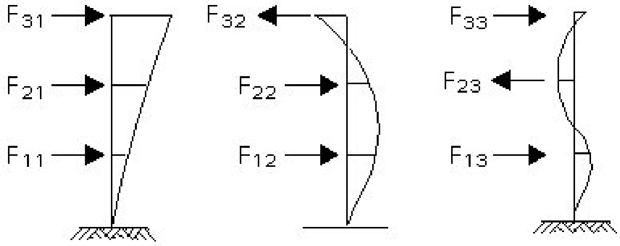


c) Under 0.43PGA

Fig. 3.19- Base shear and base moment variations under different sub soil conditions of three leg towers



First few modes shape of a typical structure



Responses of few modal shapes

The combine effect was obtained by a square root of the sum of squares (SRSS) of the various modal responses

Fig. 3.20- Typical Responses under different mode shapes

However, capturing all such higher modes effects through a simple static equation may not be quite practical. Hence, Static equivalent equation given in ANSI/TIA-222-G (2005), which seems to be based on first few modes of vibration have proven its conservativeness. Amiri et al., 2007 had also reported a similar observation regarding the static equivalent analysis of telecommunication towers using Iranian Seismic code (ISC). As per their study, the static equivalent analysis method given in ISC is based on fundamental variation modes of structures and therefore, results given by it are quite over-conservative compared to Response spectrum analyses. The gap between two results in their study had increased with increase of tower heights as observed in this study too.

According to above graphs, seismic actions (especially base moment) on towers do not increase with the weakness of sub soil properties always. As an example, if the results of four leg towers are considered, trends of base shear and moment of three towers (30 m, 50 m and 80 m) are quite different to each other under different soils classes and PGA values. In case of 80 m tower, base shear and moment have increased when sub soil classes change as C, D and E (when soil properties weaken) under all seismic conditions (see Fig. 3.18). However, as per results of 30 m and 50 m towers, same pattern was not observed.

In those towers, base shear and moment values have increased when soil classes change as C, D and E only under 0.1 g PGA value (see Figure 3.18 (a)). In contrast, under higher PGA values (0.25 & 0.43), base shear and moment increase when sub soil changes from C to D and values have decreased when sub soil class changes from D to E (see Fig 3.18 (b) and 3.18 (c)). Therefore, it can be stated that amplification of seismic forces on towers do not solely depend on changes of sub soil classes. It can be observed that Natural Period of vibration of First Modes greatly influences the seismic performances of towers under different sub soil stratums. Table 3.6 shows the natural period of vibration of first mode of towers considered in this analysis.

Table 3.6 – Natural Period of Vibration of 1st modes of towers

Tower height	Tower type	Natural period of vibration of 1st mode (s)
30 m	4 Leg	0.39
50 m	4 Leg	0.43
80 m	4 Leg	0.83
30 m	3 Leg	0.40
45 m	3 Leg	0.39
60 m	3 Leg	0.51

It is possible to observe that generally the natural period of vibration of towers increases with higher height of towers as stiffness of a tower decrease with the height of a tower. However, in case of three leg towers, Natural period vibration of 1st mode has slightly dropped, when it goes from 30 m tower to 45 m tower. Probable reasons may be the geometrical differences, member sizes, etc. of selected 30 m and 45 m towers.

Generally, as per results, spectral acceleration values of respective response spectrum curves at time value equal to Natural period of vibration of 1st mode of respective towers influence base moment variations significantly. As an example, Natural period of vibration of 1st mode of 80 m four leg tower is 0.83 s. Spectral accelerations under 0.25 g PGA of soil C, Soil D and Soil E at 0.83 s are 0.50 g, 0.58 g and 0.73 g (which are in decreasing part of response spectrum curves), respectively (see Fig. 3.21). Base moments calculated using Response Spectrum Analysis of 80 m tower under 0.25 g

PGA reduces following same pattern with changing soil types (see Fig. 3.21). However, Static Equivalent Analyses have not perfectly captured this variation may be due to inherit deficiencies of the method explained earlier. When 60 m Three leg tower under 0.43 g PGA is considered, Spectral accelerations of soil C, Soil D and Soil E at 0.51 s are 1.42 g, 1.43 g and 1.28 g, respectively. When base moments under soil condition C and D are compared, it has almost equal base moments as it can be expected since spectral accelerations at 0.51 s are 1.42 g and 1.43 g respectively. In case of sub soil E, a relatively less base moment value is observed as it has a spectral acceleration of 1.28 g at 0.51 s. Therefore, results are fully compatible with argument developed regarding influence of spectral acceleration with respect to Natural period vibration of first mode. Similar observations can be made regarding other cases as well.

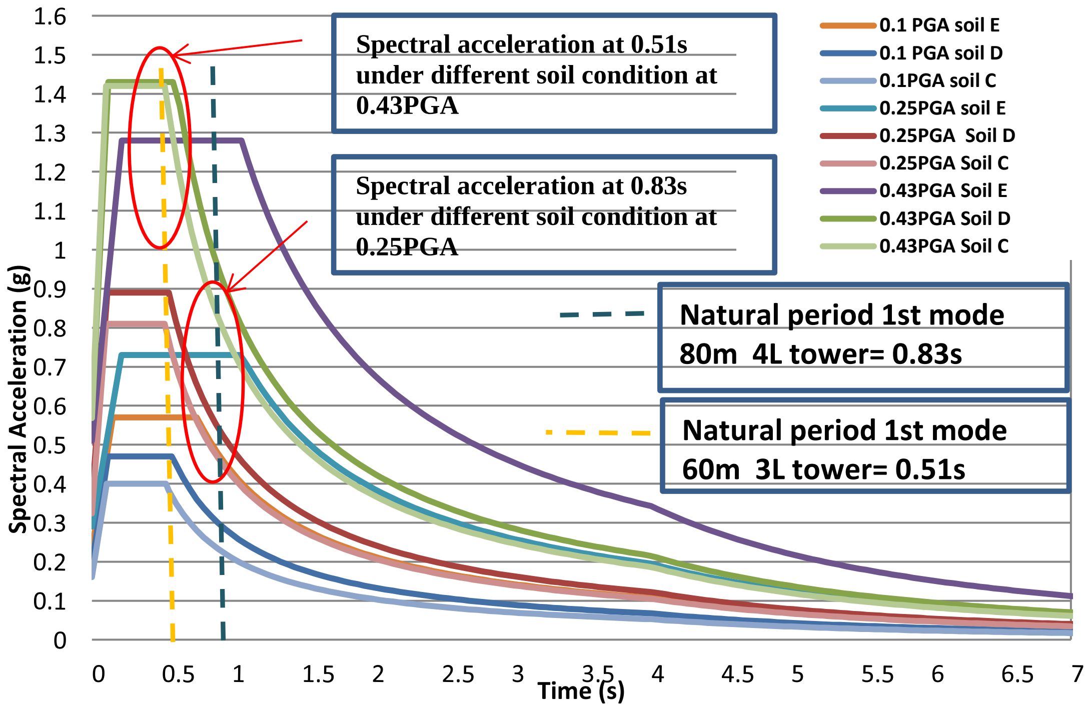


Fig. 3.21 - Response Spectrum curves with natural period of vibration of first mode of selected cases

Accordingly, the highest influence of 1st mode of structure on seismic performance has been proved even though other modes may also make considerable influences especially in taller towers. Therefore, the behaviour of Response Spectrum curve under a specific sub soil stratum close to the natural period of vibration of first mode of the particular structure can significantly influence the seismic performance of that particular structure. Hence, consideration of sub soil properties and vibration characteristics are found to be important for accurate seismic analysis of self-supporting telecommunication/broadcasting towers. However, in general, the structural performance of steel lattice telecommunication towers in the region would not get affected when an earthquake occurs if such towers have been properly designed considering recommended local design wind speeds.

3.4 Summary

Detailed evaluation of seismic performance of selected three and four leg towers under regional seismic parameters and probable sub soil conditions was focused in this analysis. Hence, seismic parameters were selected to cover whole range of seismic hazard levels in the region from minor to very severe conditions. Sub soil parameters were also selected in same way varying from normal hard soil condition to weak sub soil stratum to observe probable effects of soil properties on seismic loads.

According to results of the analyses, stresses and deformations of all selected towers under all seismicity levels and sub soil conditions are less than stresses and deformations under wind load cases, since it usually governs the design of towers. Hence, in general, lattice towers shall perform well under seismic effects once designed for the wind loads recommended.

However, amplification of seismic stresses and deformations (even though those were still less than under wind load cases) under certain sub soil conditions were observed in some towers. It was found that natural vibration characteristic (especially the first mode of natural vibration) of towers would also contribute partly to these amplifications. Once the shape of Response spectrum changes with sub soil properties, occurrence of amplification would be dominated by the Spectral acceleration of the Response spectrum curve with respect to the natural period of vibration of first mode of the particular structure.

Stresses and deformations observed under Equivalent static analyses (as per method given in ANSI/TIA-222-G-2005) were found to be considerably higher than the same of Response spectrum analyses in all respective cases. Therefore, conservative approach of Equivalent static analysis theory was again proved as it would not consider effects higher modes.

Chapter 4 -

Investigation on behaviour of lattice tower baseplates under uplift forces

4.1 Structural behaviour of lattice tower baseplates

Importance of a baseplate of a tower can be clearly understood as it acts as the interface between main structure and its foundation. Regarding baseplates of columns in buildings, design theories are well developed and well documented in Text books and design standards. Downward/compressive forces govern the design of column baseplates of steel buildings in almost all cases unless lightweight single storey structures (such as warehouses with portal frames) located in high wind areas.

However, in case of lattice telecommunication/broadcasting towers, the scenario is totally different as one or two legs would predominantly subject to uplift forces based on the direction of the wind as discussed in Section 2.4.1. At the same time, leg or legs in the other side will be subjected to compressive or downward force and behaviour of baseplate of that leg or legs is similar to a baseplate of a typical column baseplate.

In case of a baseplate of a column or tower which is subjected to downward forces, stress transfer from baseplate to stem of column would take place through both anchor bolts and concrete or grout layer immediately below the baseplate. Therefore, design theory of baseplate of such cases had been developed based on plate bending/deformation theory under such condition. Well defined theories and formulae are available in this regard in General Steel Design codes and Text books as discussed in Sections 2.4.3.

However, the above indicated case would not be critical in case of tower baseplates as baseplate/baseplates subject to uplift in opposite direction of the tower. It is more susceptible than compressive condition due to different stress transferring pattern under uplift condition. Uplift force has to be entirely resisted by anchor bolts connected to the baseplate and structural grouting or concrete below baseplate would not make any contribution. Therefore, baseplate subjected to uplift shall have adequate structural capacity under this condition and design check of baseplate under this critical condition is of utmost importance to ensure the safety of the tower. However, as discussed in

Section 2.4.4, direct guideline for structural checking of lattice tower baseplates that match with actual orientations of tower legs, anchor bolts and stiffener plates is not available.

4.2 Yield line theory for tower baseplate subjected to uplift

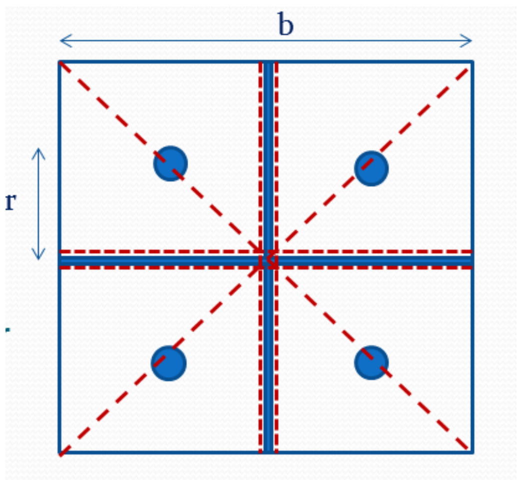
Due to the complex deformation pattern of baseplate, two dimensional yield line analysis theory was selected to develop deign guideline/formula for this case. Fig. 4.1 shows the selected baseplate pattern for this analysis, which is the most common type of baseplate that can be observed in lattice towers.

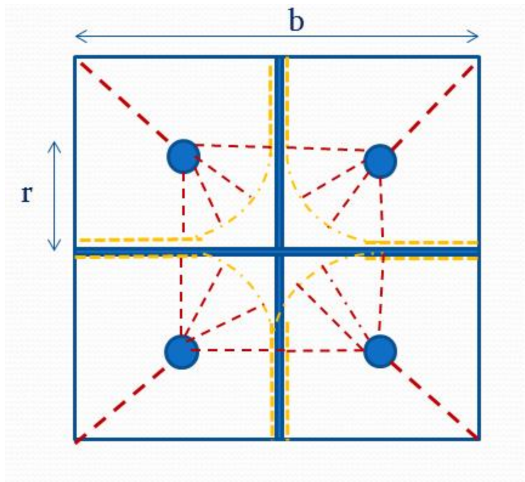
Fig 4.1- Typical baseplate of a lattice tower

As in Fig. 4.1, usually crucified stiffener plates are provided for the "L" angle leg member of the tower at baseplate level to improve the stress distribution. Even though the predominant force on the baseplate under this scenario is uplift force, it may also be subjected to certain amount lateral forces due to non-verticality of leg members in actual towers. Usually, this inclination is less than 6^0 and therefore, lateral forces are much lower compared to predominant uplift force.

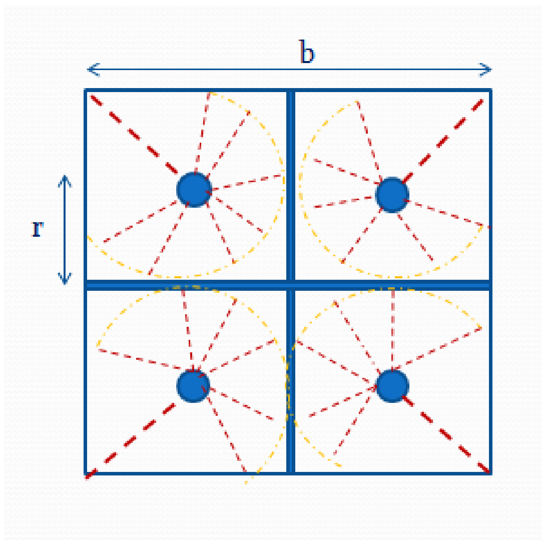
Even though the baseplate given in Fig. 4.1 appears as an axisymmetric system, the quadrant in which the "L" angle section is located would be subjected to higher stresses compared to other quadrant in reality.

However, for the application of fundamental yield line theory for the analysis, these factors can be neglected and corrections can be introduced for these once the fundamental theory is developed. Accordingly, this system was considered as an axisymmetric system with uplift forces and two dimensional yield line failure patterns for the system were identified. There are quite a few yield line patterns that can be identified. However, based on the preliminary checks made (consideration of symmetricity and other fundamental theories relevant to formations of yield lines), it was able to limit number of patterns to three (03) (as given in Fig. 4.2) for detail analysis.





Pattern 1 **Pattern 2**



Pattern 3

Fig 4.2 Three (03) selected yield line patterns for detail analysis

According to the general theory of yield line analysis, the work done by external loads is equal to the work absorbed by the yield lines.

Work done by external loads = Work absorbed by the yield lines

$$\sum w \times \delta = \sum m \times l \times \theta \tag{4.1}$$

Where,

w	=	Yield load
δ	=	Perpendicular deflection at which yield load moves
m	=	Moment capacity of the section per unit length
l	=	Length of yield line
θ	=	Rotation of the segment satisfying the compatibility of deflection

By applying the above theory to the three selected yield line patterns, yield analyses were carried out. For convenience, only one quadrant of the baseplate was considered in each case considering assumed symmetricity. Derivations of yield line equations for three (03) patterns are as follows:

Analysis of Yield Line Pattern 1

Considering one quadrant of the baseplate,

Work done by external load $= \dfrac{\sqrt{2}\ r \times \frac{F}{4}}{\sqrt{2} \times \frac{b}{2}}$ (4.2a)

$= \dfrac{Fr}{2b}$ (4.2b)

Work absorbed by the yield lines (W_{ab}) $= 2m_p\left(\sqrt{2} \times \frac{1}{\sqrt{2}}\right) + 2m_p$

$$\mathbf{W_{ab}} = 4m_p \tag{4.3}$$

Based on fundamentals of yield line theory,

$$m_p = f_y \times \frac{t}{2} \times \frac{t}{2} = \frac{f_y t^2}{4} \tag{4.4}$$

Where f_y – characteristic yield strength of the baseplate

Hence, according to fundamentals of yield line theory as per Eq. 4.1

$$F = \frac{2bf_y t^2}{r} \tag{4.5}$$

Analysis of Yield Line Pattern 2

Considering one quadrant of the baseplate,

Work done by external load $= \dfrac{r\times \frac{F}{4}}{\sqrt{2}\left(\frac{b}{2}-r\right)+r}$ (4.6)

Work absorbed by the yield lines (W_{ab})

$$= m_p \left[\frac{4\left(\frac{b}{2}-r\right)}{b} + 2\times\frac{2}{\sqrt{2}b}\times\sqrt{2}\left(\frac{b}{2}-r\right) + \frac{\pi r}{2\left(\sqrt{2}\left(\frac{b}{2}-r\right)+r\right)} \right] \quad (4.7)$$

$$= m_p \left[\frac{8\left(\frac{b}{2}-r\right)}{b} + \frac{\pi r}{2\left(\sqrt{2}\left(\frac{b}{2}-r\right)+r\right)} \right] \quad (4.8)$$

Based on fundamentals of yield line theory,

$$m_p = f_y \times \frac{t}{2} \times \frac{t}{2} = \frac{f_y\, t^2}{4} \quad (4.9)$$

Where f_y – characteristic yield strength of the baseplate

Hence, according to fundamentals of yield line theory as per Eq. 4.1

$$F = \frac{f_y t^2 \left\{ \left[\sqrt{2}\left(\frac{b}{2}-r\right)+r\right] \times \left[\frac{8\left(\frac{b}{2}-r\right)}{b} + \frac{\pi r}{2\left(\sqrt{2}\left(\frac{b}{2}-r\right)+r\right)} \right] \right\}}{r} \quad (4.10)$$

Analysis of Yield Line Pattern 3

Considering one quadrant of the baseplate, Work done by external load $= \dfrac{r \times \frac{F}{4}}{\sqrt{2}\left(\frac{b}{2} - r\right) + r}$ (4.11)

Work absorbed by the yield lines (W_{ab})

$$= m_p \left[\frac{4\left(\frac{b}{2} - r\right)}{b} + \frac{r\left(\frac{\pi}{2} + 2\sin^{-1}\left[\frac{\frac{b}{2} - r}{r}\right]\right)}{\sqrt{2}\left(\frac{b}{2} = r\right) + r} \right] \tag{4.12}$$

Based on fundamentals of yield line theory,

$$m_p = f_y \times \frac{t}{2} \times \frac{t}{2} = \frac{f_y t^2}{4} \tag{4.13}$$

Where f_y – characteristic yield strength of the baseplate

Hence, according to fundamentals of yield line theory as per Eq. 4.1

$$F = \frac{f_y t^2 \left[\sqrt{2}\left(\frac{b}{2} - r\right) + r\right] X \left[\frac{4\left(\frac{b}{2} - r\right)}{b} + \frac{r\left(\frac{\pi}{2} + 2\sin^{-1}\left[\frac{\frac{b}{2} - r}{r}\right]\right)}{\sqrt{2}\left(\frac{b}{2} - r\right) + r} \right]}{r} \tag{4.14}$$

A rational value for Equation 4.14 can only be obtained when r/b > = 0.25 as Sin $^{-1}$[(b/2-r)/r] would not have a realistic value when r/b < 0.25.

According to above analysis, it is possible to obtain three (03) separate governing equations (Eq. 4.5, Eq. 4.10 and Eq. 4.14) under three (03) shortlisted yield line patterns. Due to the complex nature of equations, identification of the governing failure criteria out of three (03) cases through a mathematical technique is not practically possible. Therefore, it was decided to carry out a parametric study using real particle baseplates in this regard to identify the governing yield line pattern. Yield strength each baseplate was considered as 275 N/mm^2 as usually S275 steel are used as baseplates. According to yield line theory, the case where it gives lowest failure forces is the critical case as the failure will take place under the lowest energy. Table 4.1 shows the details of the parametric study.

According to results of the parametric study, it can clearly identify that yield line pattern 3 would become governing pattern where r/b > 0.25 and pattern 2 would become governing when r/b =< 0.25. In any case, pattern 1 would not become critical as failure forces are high compared to other two patterns. Therefore, as per theory, a tower baseplate shall fail either under yield line pattern 2 or 3 depending on the r/b ratio.

Table 4.1 – Details of parametric analysis

Baseplate thickness mm	b (mm)	r (mm)	r/b	f_y (N/mm^2)	Yield Force from yield line Pattern 1 (kN)	Yield Force from yield line Pattern 2 (kN)	Yield Force from yield line Pattern 3 (kN)	Governing Yield force (kN)	Governing yield line Pattern
3.4	250	50	0.20	275	31.79	28.80	Not valid	28.80	Pattern 2
3.4	250	75	0.30	275	21.19	14.87	14.57	14.57	Pattern 3
3.4	250	87.5	0.35	275	18.17	11.12	10.87	10.87	Pattern 3
3.4	250	100	0.40	275	15.90	8.43	8.32	8.32	Pattern 3
15	450	100	0.22	275	556.88	477.68	Not valid	477.68	Pattern 2
15	450	125	0.28	275	445.50	331.58	329.11	329.11	Pattern 3
15	450	150	0.33	275	371.25	237.97	232.35	232.35	Pattern 3
15	450	175	0.39	275	318.21	174.36	171.61	171.61	Pattern 3
22	500	125	0.25	275	1,064.80	851.57	948.42	851.57	Pattern 2
22	500	150	0.30	275	887.33	622.68	610.08	610.08	Pattern 3
22	500	175	0.35	275	760.57	465.48	455.13	455.13	Pattern 3
22	500	200	0.40	275	665.50	353.09	348.29	348.29	Pattern 3
28	550	100	0.18	275	2,371.60	2245.30	Not valid	2,245.30	Pattern 2
28	550	150	0.27	275	1,581.07	1192.40	1,190.23	1,190.23	Pattern 3
28	550	200	0.36	275	1,185.80	698.41	684.20	684.20	Pattern 3
28	550	225	0.41	275	1,054.04	544.56	538.16	538.16	Pattern 3

4.3 Experimental Investigation

4.3.1 Scope of investigation and test set up

The equations developed using yield line analysis would need modifications for the secondary effects that were neglected during yield line analysis. It was planned to execute it through a parametric study using Finite Element Modelling (FEM) of selected set of baseplates. However, it is always advisable to carry out limited experimental investigation to verify FEM results before using it for parametric study. Further, experimental results would verify the outcome of yield line analysis as well. Therefore, a limited experimental investigation was under taken.

Three (03) samples as given in Fig. 4.3 were developed and actual dimensions of samples are given in Table 4.2, where “t” is the thickness of the baseplate.

Fig. 4.3 Typical test sample

Table 4.2 – Actual dimensions of samples

Sample name	b x b (mm)	r (mm)	t (mm)
Sample 1	251.1 x 252.3	75.4	3.4
Sample 2	249. 7 x 251.1	76.1	3.4
Sample 3	251.4 x 252.1	75.2	3.4

Two (02) 50x50x5 “L” angle sections in back to back orientation were welded to the mild steel baseplate as in Fig. 4.4. Even though this arrangement was not perfectly symmetrical due to practical limitations, still point of application of loading was maintained at the centre point to maintain the acceptable accuracy. Four (04) numbers of 12 mm bolts holes located having same radial distance (106 mm, where r = 75 mm) from centre of baseplate while locating one (01) bolt hole in every quadrant. Geometries and other parameters of fabricated samples were maintained as similar as possible to the system used for yield line analysis. Stiffener plates having a height 100 mm, extended up to edges of the baseplate were also provided for the same reason. A ring was provided at the top of the crucified "L" angles to apply uplift force for the system. All steel material used to fabricate samples were compiled strength S275 of BS5950:2000 and it was verified with tensile testing discussed in Section 4.3.3.

The application of tensile loading was carried out through a 5 MT chain block connected to the steel ring fabricated at top of samples during testing as experiment was carried out as a quasi-static tensile load test in force control approach. The applied load was measured through POT -10 Maruto Proving ring having a load capacity of 100 kN.

To measure the transverse strains of a specified locations (as marked in Fig. 4.4) of the baseplate samples, a 60 mm Gauge length Tokyo Sokki Kenkyujo Co. Ltd., Japan strain gauge was fixed on the top surface of baseplate at a specific location. The reading of the strain gauge was obtained through Data Logger TDS -530. A standard transducer was connected to the same Data Logger TDS -530 with necessary clamping to measure the vertical deformation of the baseplate at a specified point.

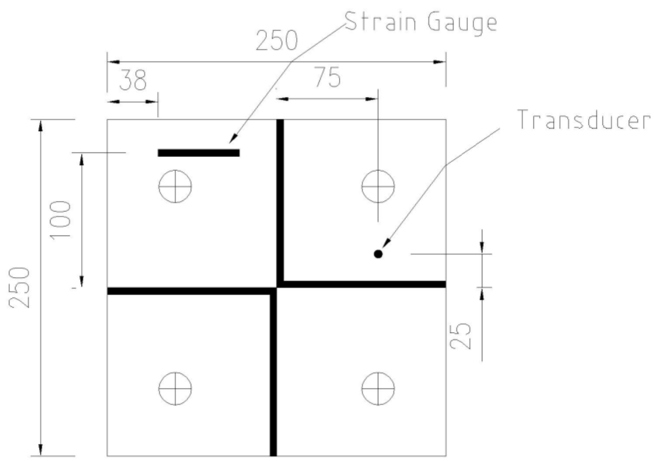


Fig. 4.4- Locations of transducer and strain gauge on test samples
(All dimensions are in mm)

The physical setup used for execution of testing is shown in Fig. 4.5.

Fig.4.5 - Physical setup with pulling mechanism and Data logger

The baseplate test samples were fixed to a 20 mm steel plate fixed to two steel "I" beams during testing. Deformation of the bottom plate was checked prior to the testing and that deformation was found to be negligible.

4.3.2 Results of testing

According to physical parameters of the samples used for the testing, the failure yield line pattern should be **yield line pattern 3** and yield force **equation should be Eq. 4.14** as r/b ratio of this sample is 0.30, which is higher than 0.25. Therefore, the calculated yield force for this sample was 14.62 kN as per the formula developed (The yield strength of the material was considered as 276 N/mm^2 based on average value of the tensile testing carried out using samples taken from the same material used for the baseplate – refer Section 4.3.3).

Deflection data and stain data of specified locations were collected through the transducer and the strain gauge connected to the Data logger TDS -530. Load vs Deflection curves of three (03) samples with respective to data collected through transducer (after necessary adjustments for initial stiffness corrections) is given in Fig. 4.6.

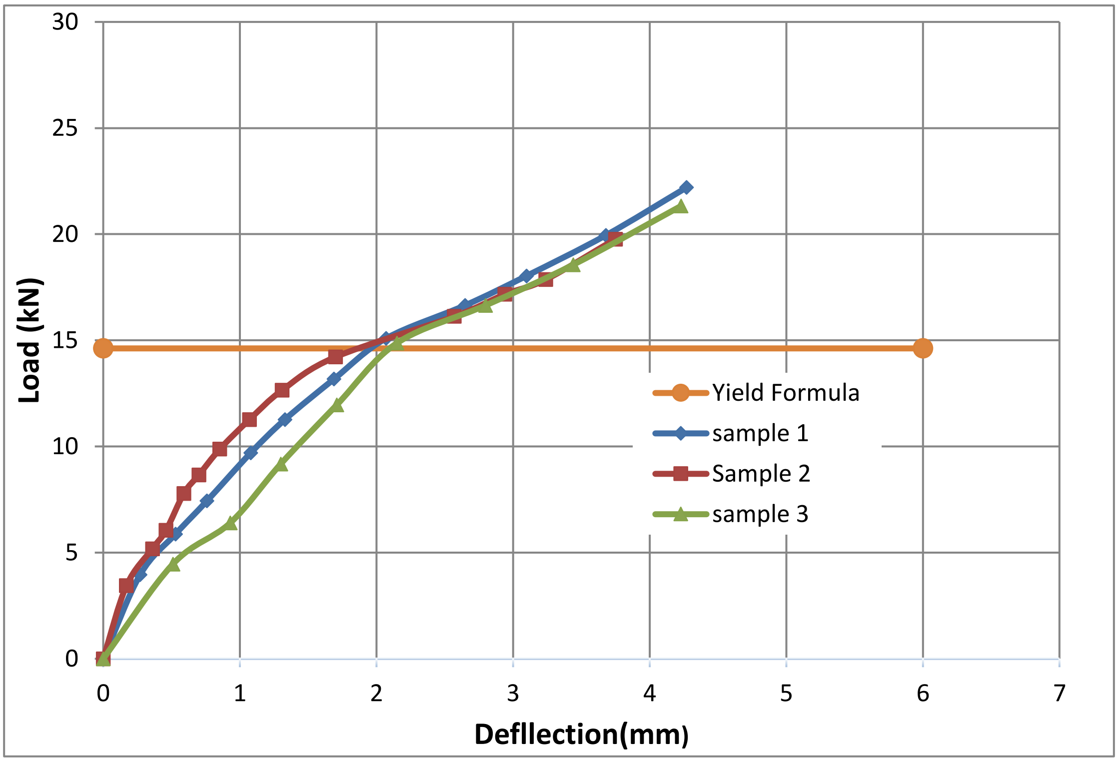


Fig. 4.6 - Deflection curves as per data measured by transducer

The load calculated using the yield line formula developed is also marked in graphs for the comparison. According to deflection curves of physical models, observed yield loads of two samples (in sample 1 and sample 3) are higher than the yield load calculated using yield formula proposed. In case of sample 2, yield load was found to be 14.27 kN and it is almost equal (it about 98% of the yield load calculated by the formula) to the yield load given by the formula.

Similarly, results obtained through the strain gauge (after necessary adjustments initial stiffness corrections) provided for each sample are given in Fig. 4.7. An observation similar to deflection data could be found for the strain data as well.

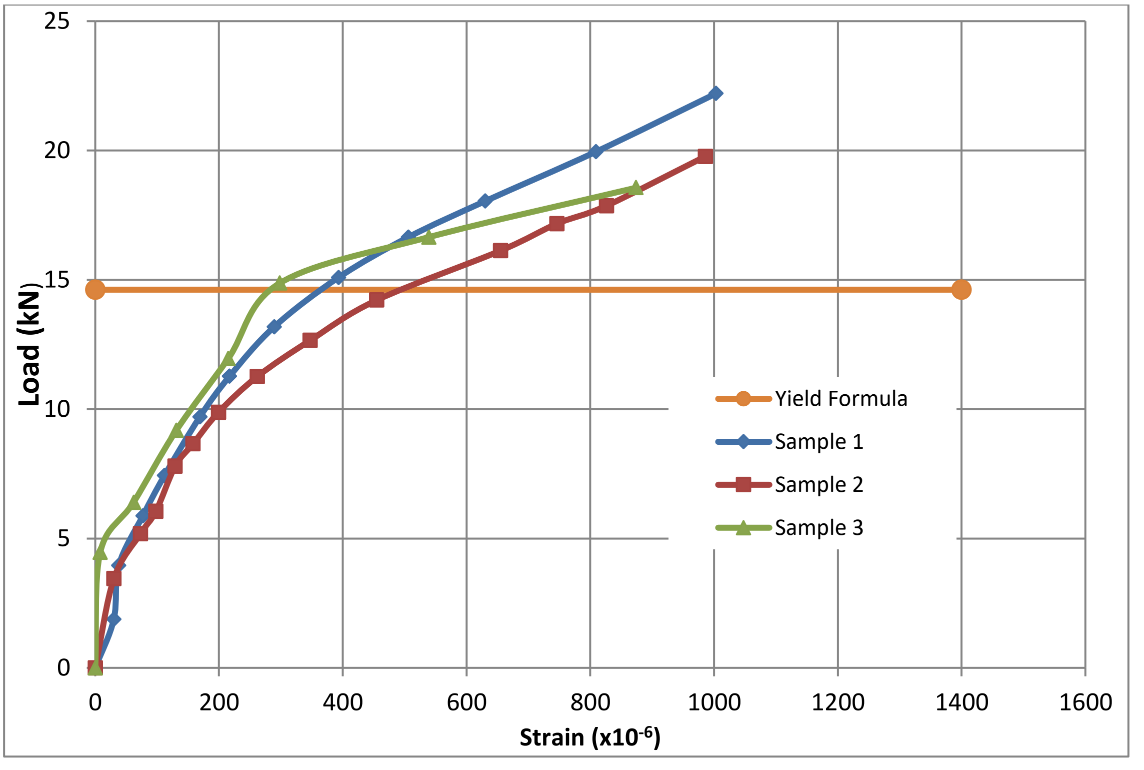


Fig. 4.7 - Curves developed based on data recorded from the strain gauge

Hence, according above results, acceptable matching can be observed between yield load calculated using the formula proposed and actual yield points of tested samples. Further, at the end of the testing, permanent deformations were observed in each test sample and it is a proof for exceedance of yield point during tests. The photographs of samples at the end of tests are given in Fig. 4.8.

Sample 1 **Sample 2** **Sample 3**

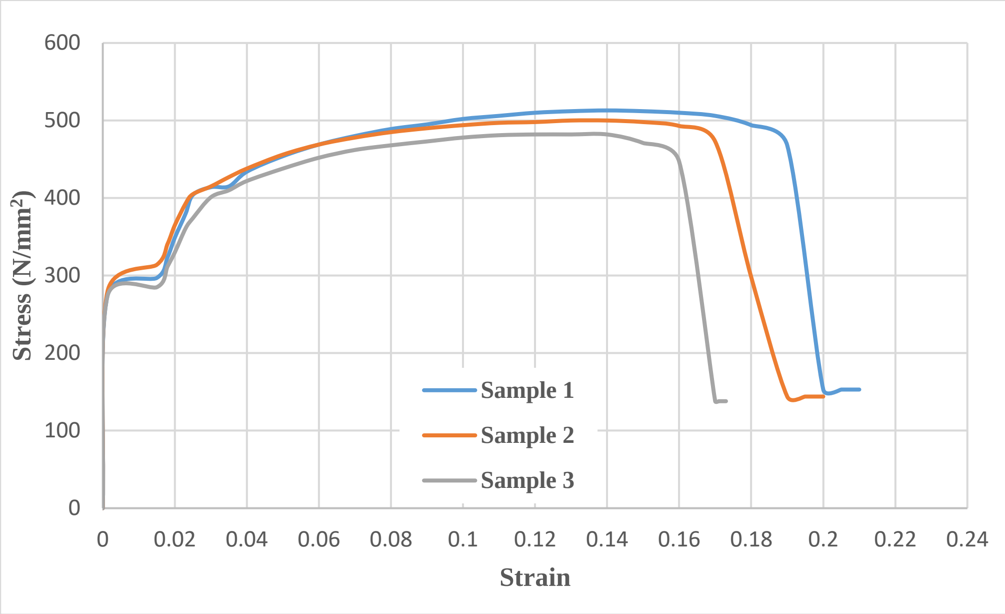
Fig. 4.8 – Deformed samples at the end of test

4.3.3 Tensile testing of baseplate samples

For the purpose of obtaining essential material properties of baseplates used for the experimental investigation, standard tensile testing was carried out using three (03) 100 mm samples of the same material. Testing was carried out using a Universal testing machine as per the standard procedure until failure.

The stress- curves of three (03) samples are given in Fig. 4.9 (a). Based on tensile testing results, average yield strength and Modules of elasticity were found as 276 N/mm^2 and 210 kN/mm^2, respectively.

Fig. 4.9 (b) shows the stress-strain curve developed based on experimental data of tensile testing to use for Finite element modelling.



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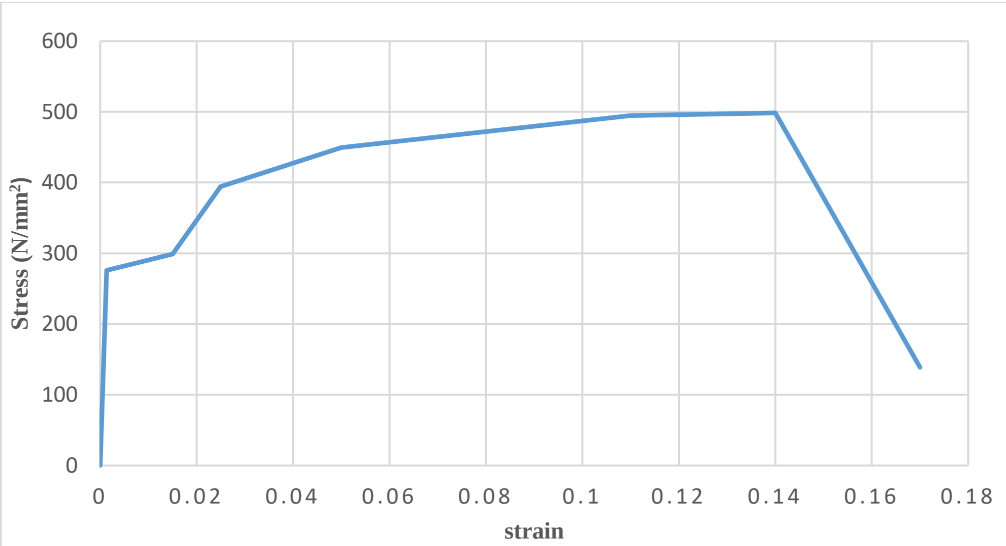
(a) Stress- strain curves of tensile testing



(b) Stress- strain curve developed for FEM modelling based on tensile testing data

Fig. 4.9 – stress – strain curve of tensile testing

4.4 Development of Finite element models for baseplates

4.4.1 Introduction

As discussed in Section 4.3.1, one of the main purposes of carrying out limited experimental investigation was the validation of Finite element models. Once numerical models are validated, parametric analyses required to fine-tune the already developed formula based on Yield line theory can be carried out using validated numerical models as conducting such parametric analysis using physical test models is an expensive and tedious exercise.

There are a lot of Finite element programs available to carry out Non liner static analysis for this type of work. SAP2000 (version 14.0.0 (2009) - User's Manual,Computer and Structures, Inc., Berkeley, CA) and ABAQUS (Simulia, Abaqus 6.14 (2014) –Dassault Systèmes Simulia Corp., Providence, RI, USA) were selected for the numerical modelling for this work.

4.4.2 Characteristics of the finite element models

SAP2000 model

The accuracy of numerical analysis greatly depends on the factors like element types, geometrical and material non-linearity, boundary conditions, etc. Accordingly, following parameters were carefully selected in SAP2000 commercial software to simulate the test model.

- Full 3D model was prepared using Shell elements compatible with test samples
- The material properties (for both elastic and plastic ranges) were assigned as per the stress- stain curve presented in Fig. 4.9 (b)
- A 10 mm x 10 mm liner grid was used to create Shell elements for entire model
- The Shell elements were defined as Shell-layered/Nonlinear elements with nonlinearity in all S11, S22 and S12
- The anchor bolts (4 Nos of anchor bolts) were modelled as pin supports after initially trails with different alternatives. (In initial trails, bolts were modelled as spring supports with actual flexibility characteristics of bolts. Nevertheless, results were almost same as in the pin support condition)
- Non liner analyses were carried out at selected load steps (few load steps beyond the yield load were also considered)

Typical three dimensional model developed in SAP2000 is given in Fig. 4.10.

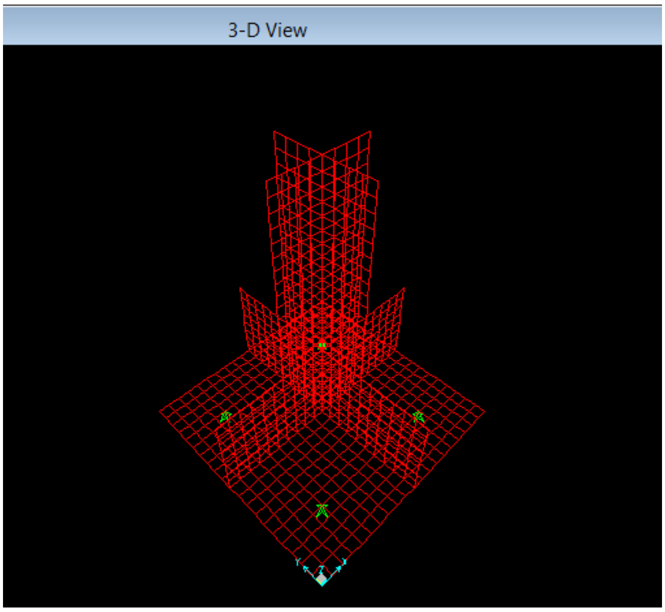


Fig. 4.10 - Three dimensional view of the numerical model developed in SAP2000

ABAQUS model

In case of ABAQUS also, the most appropriate characteristic to simulate the test samples were selected. The salient features of the ABAQUS model as follows:

- The whole test model was developed as a 3D solid homogenous development compatible with physical features of test model
- The analysis was carried out as a “Static, General” analysis in ABAQUS
- The material properties (for both elastic and plastic ranges) were assigned as per the stress- stain curve presented in Fig. 4.9 (b)
- The load was applied as a uniform pressure at the top surface of the model
- Boundary conditions of bolt holes are defined as pinned and all other boundaries were kept as free
- The element meshing was done using tetrahedron elements with 3D stress meshing option while associating boundary conditions to meshing. Acceptable finer meshing was attained at the critical regions accordingly
- Force controlled static Non liner analysis was carried out

Typical three dimensional model developed in ABAQUS without element meshing is given in Fig. 4.11. Same model with element meshing is given in Fig. 4.12.

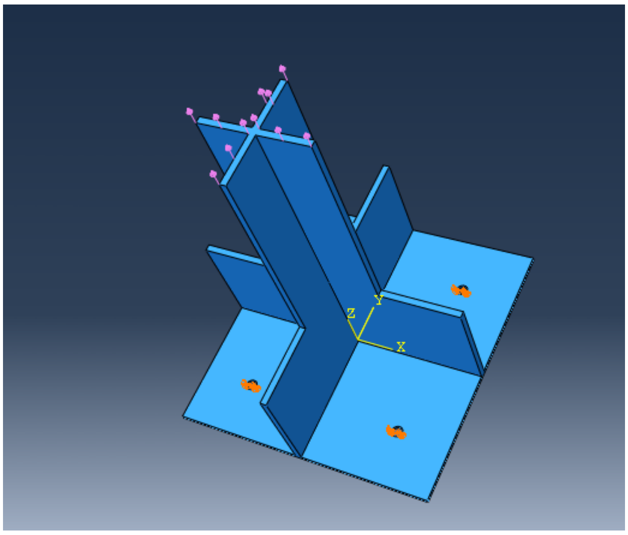


Fig. 4.11 - Three dimensional view of the numerical model developed in ABAQUS

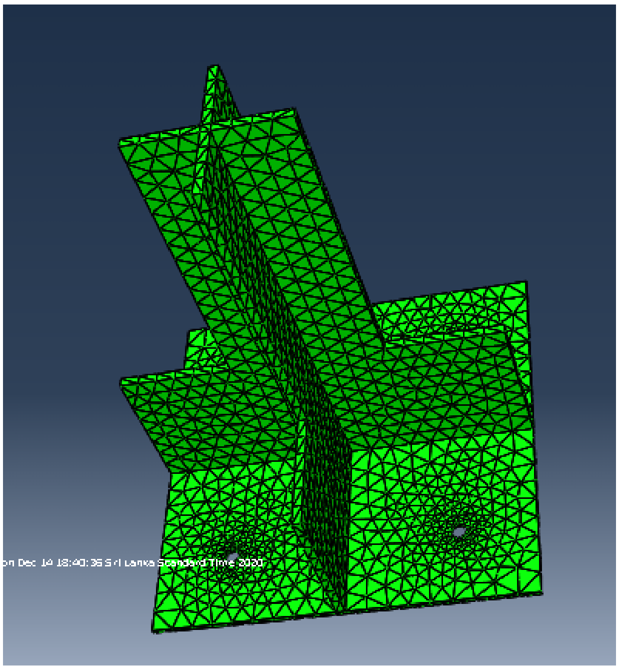
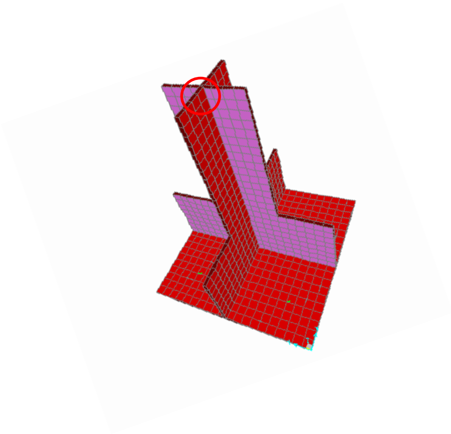


Fig. 4.12 - Three dimensional view of the numerical model developed in ABAQUS with element mesh

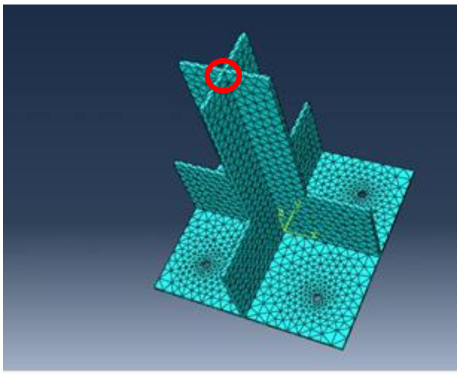
4.4.3 Results of Finite element modelling and discussion

4.4.3.1 Deflection data and deformation patterns

Nodal displacements of top most points of SAP2000 and ABAQUS (as marked in Fig. 4.13) analyses are plotted in Fig. 4.14 as it would give clear indication of the yield load of FEM models.



(a) SAP2000 model



(b) ABAQUS model

Fig. 4.13 Top most point of SAP2000 and ABAQUS models (The top point is indicated in red circle)

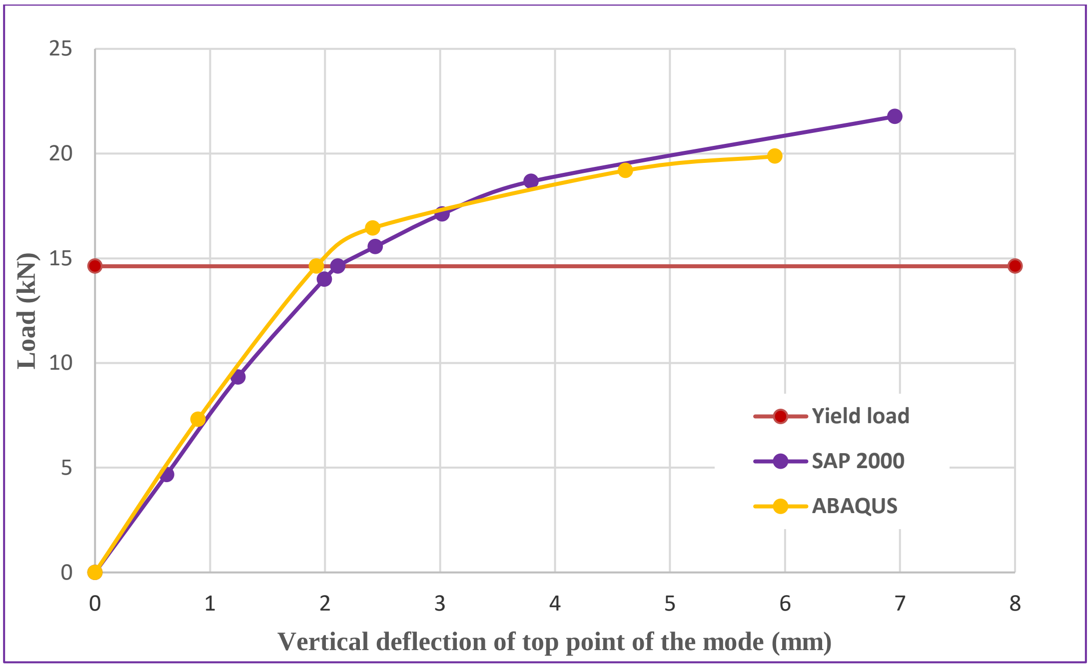


Fig. 4.14 Nodal displacements of top most points of SAP2000 and ABAQUS models

A reasonable match between two curves can be observed. Also, Yield loads indicated by SAP2000 and ABAQUS models are quite close to each other and it is reasonably match with the yield load predicted by the Yield line analysis as well.

For comparison with the results of experimental investigation, nodal displacements data of both SAP2000 and ABAQUS models with respect to the point where transducer was installed in actual physical test models were extracted. Based on extracted data of both SAP2000 and ABAQUS, Load Vs. Deflections curves were plotted in the same graph, where deflection data obtained through transducer in physical testing had been plotted. That common graph is shown in Fig. 4.15.

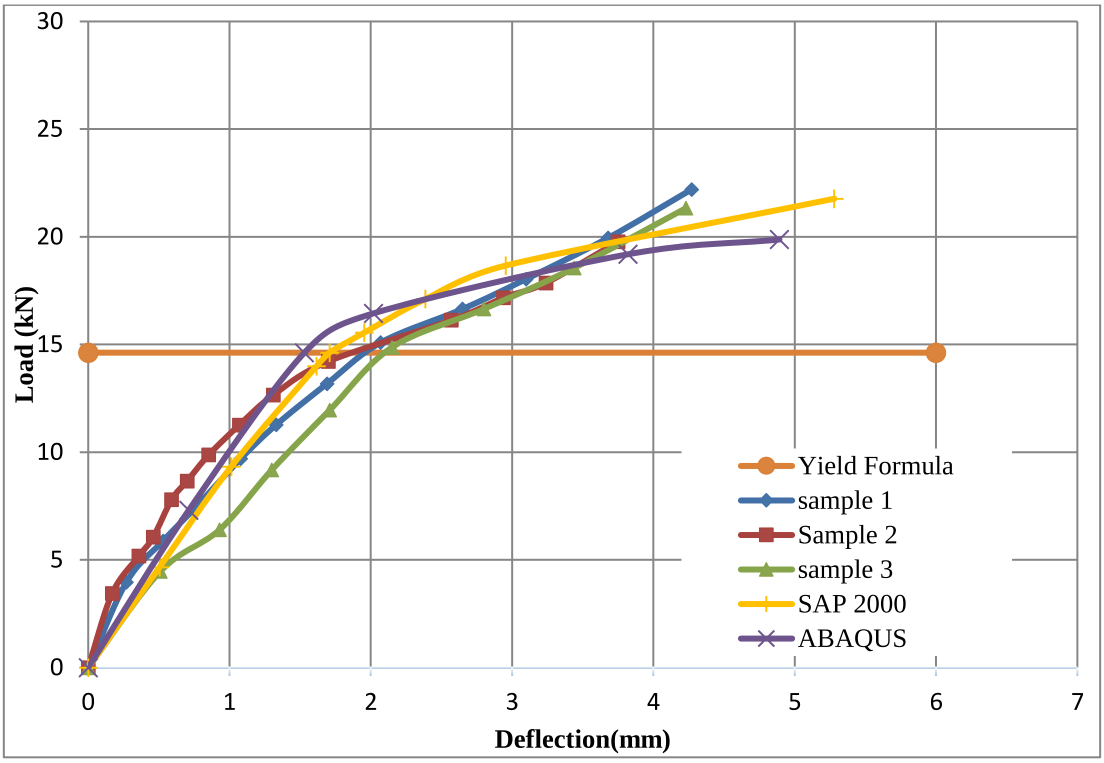
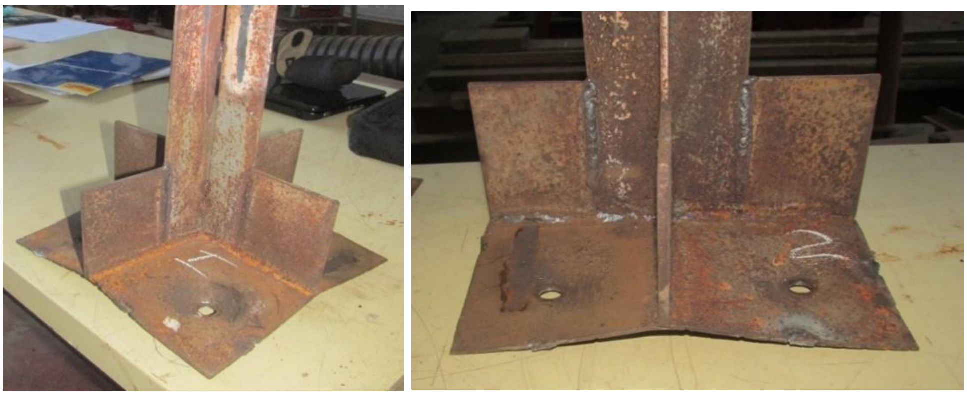


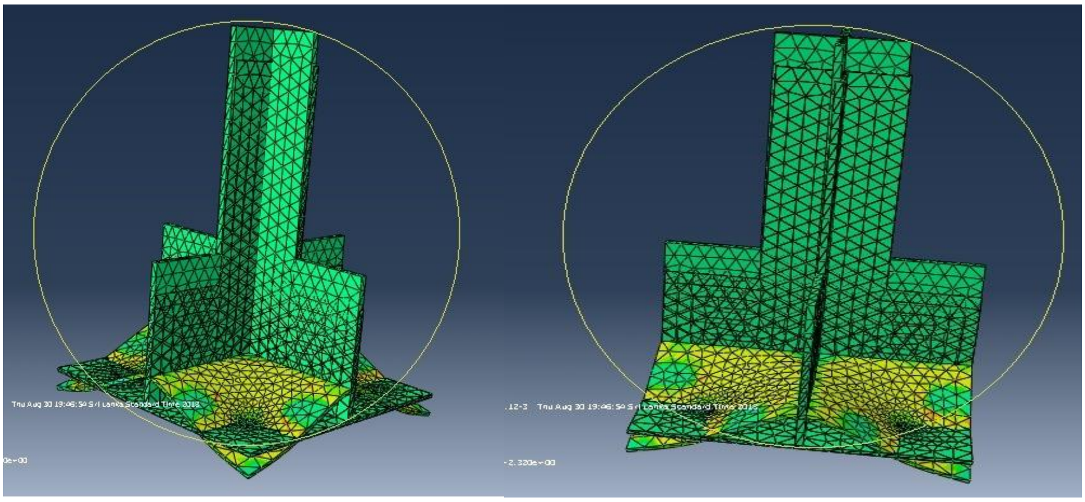
Fig.4.15 - Deflection results of physical tests and numerical analyses

As can be seen, the yielding predicted by the mathematical models and the actual testing indicate a reasonable match. The Yield load predicted by the Yield line formula is also compatible with above results.

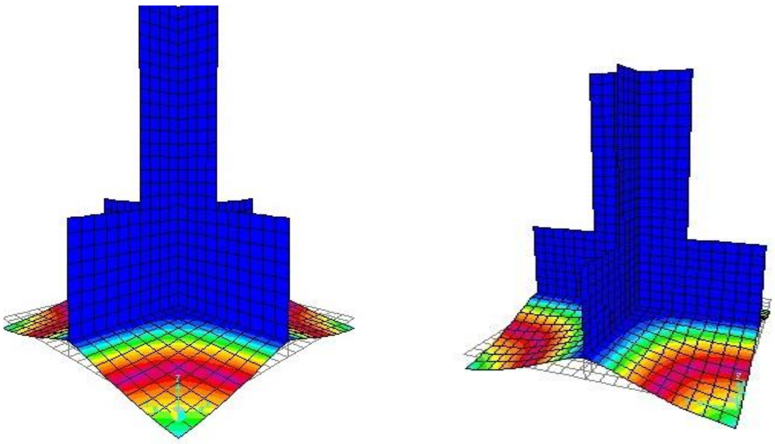
Fig. 4.16 shows the deformation shapes of physical test samples, ABAQUS model and SAP2000 model. Almost identical deformation patterns and characteristics can be observed between these cases and hence indicates that numerical models would be representative.



(a) Deformed pattern of physical models



(b) Deformation pattern of ABAQUS model



(c) Deformation pattern of SAP 2000 model

Fig.4.16 - Comparison of deformation results in graphical form

4.4.3.2 Strain data results

Fig. 4.17 shows strain data extracted from SAP2000 and ABAQUS models relevant to locations of the strain gauge of physical test samples. For the purpose of comparison, the strain data from numerical analysis and physical testing were plotted on the same graph.

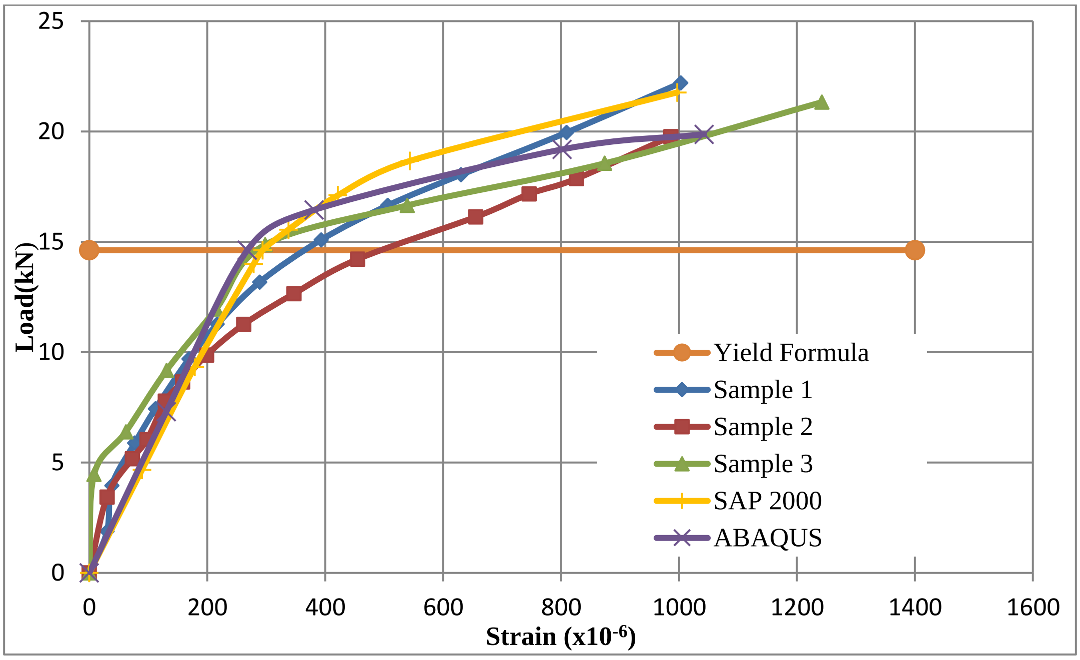


Fig.4.17 - Strain results of physical tests and numerical analyses relevant to the Strain gauge

According to both physical test and numerical analyses results, strain values at which the linearity of curves is lost, are less than the yield strain that may be calculated based on Hook's law. This is understandable as baseplate would yield under two dimensional deformation due to nature of stress patterns.

A closer matching between results of physical test models and numerical analyses results (results of both ABAQUS and SAP2000) can be observed in Fig. 4.17 except sample 2.

In ABAQUS, there is a feature called "AC Yield ", where it graphically displays the regions (in yellow colour in Fig. 4.18) having plastic strains developed (where Von Mises stresses in particular elements exceeds the yield stress criteria) with the contours. Fig. 4.18 shows graphical output extracted from the ABAQUS.

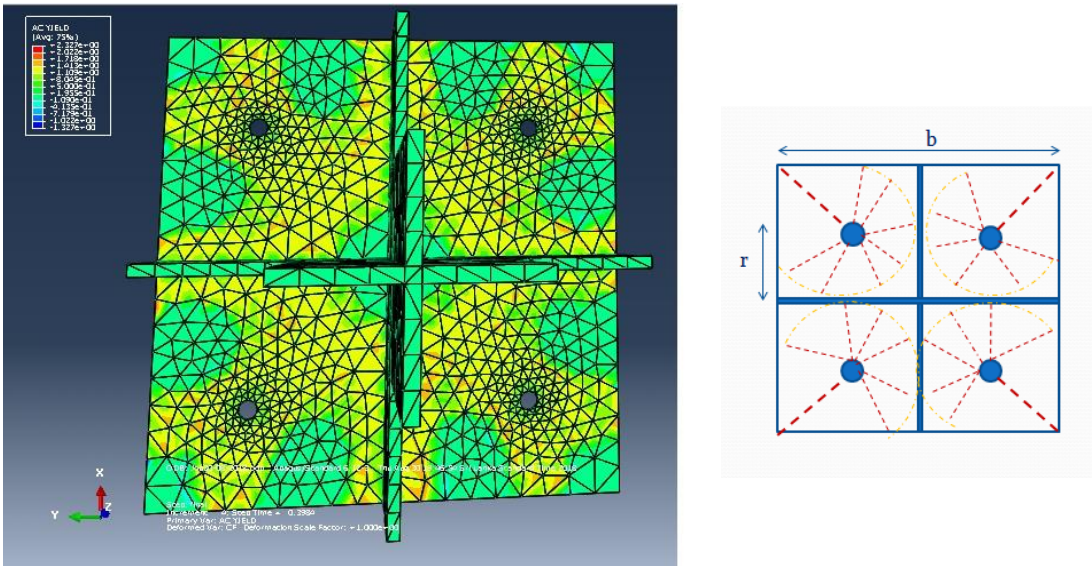


Fig.4.18 - Yield contours in ABAQUS and governing yield line pattern identified

In the case of SAP2000, identification of yielding regions through a direct graphical output is not possible. However, by plotting Von Mises stresses, yielding regions can be identified.

Under Von Mises failure criteria of a shell element, Von Mises stress (equivalent tensile stress) is defined as;

$$\sigma_v^2 = \sigma_{11}^2 - \sigma_{11}\times\sigma_{22} + \sigma_{22}^2 + 3\sigma_{12}^2 \qquad (4.15)$$

Where;

σ_{11} – Principal plane stress in X direction

σ_{22} – Principal plane stress in Y direction

σ_{12} – In-plane Shear stress

σ_v – Von Mises stress (equivalent tensile stress)

Accordingly, if the Von Mises stress in an element exceeds 276 N/mm^2 (yield strength), it can be considered as an element under yielding. Hence, in SAP2000 numerical model, Von Mises stress contours are plotted while defining Von Yield failure stress as the maximum limit. Fig. 4.19 shows the graphical output of SAP2000 developed based on this method.

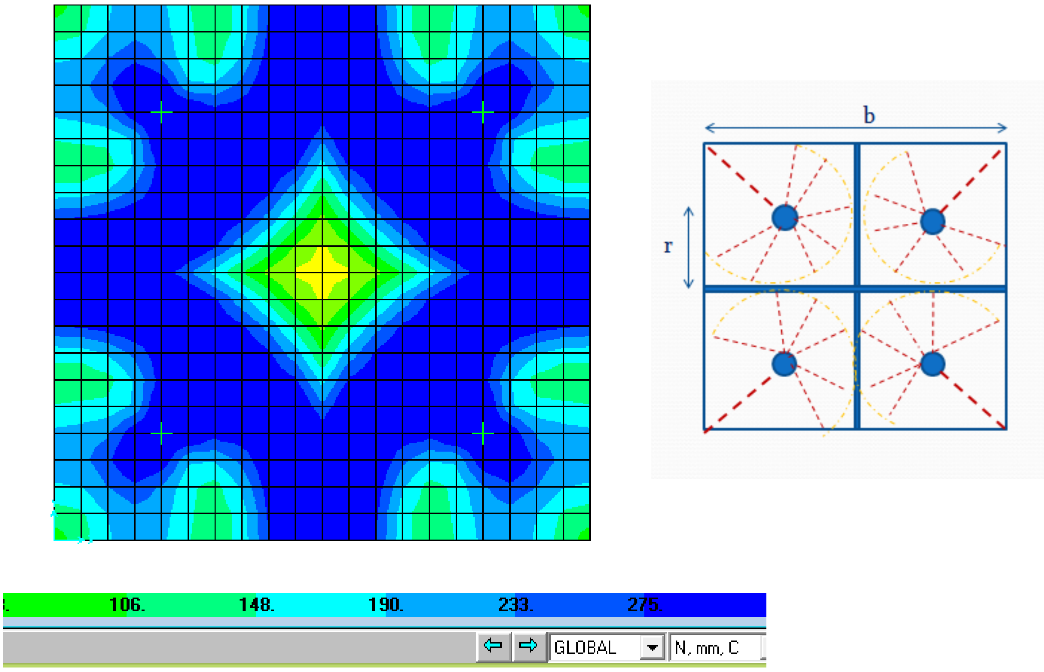


Fig. 4.19- Yield contours in SAP 2000 and governing yield line pattern identified

The dark blue area in Fig. 4.19 represented the area where Von Misses stresses exceeded the yield strength. Accordingly, this also can be considered as a graphical output representing yielding pattern, which can be compared with ABAQUS output given Fig. 4.18. In comparison, it can be identified that both graphical outputs are more or less identical. Further, both patterns are compatible with the Yield line pattern 3 that was used to calculate yield force under yield line analysis.

4.5 Summary

Development of design methodology for commonly used tower baseplates under governing uplift load case was focused here. Accordingly, Yield line theory was adopted for development of design formula to calculate yield force while neglecting secondary variations at this stage. This successfully ended up with the development of basic formulae to calculate yield load of the baseplate arrangement considered.

For the verification of the accuracy of formulae developed, an experimental investigation was carried out with physical test samples. Experimental test results confirmed the validity of formulae developed.

For further verification of formulae and validation of numerical models for further studies, numerical modelling of test samples was carried out. According to numerical modelling carried out using SAP2000 and ABAQUS softwares, comparable results were obtained as discussed in Sections 4.4.3.1 and 4.4.3.2. Further, all three results (physical testing, numerical analyse and Yield line analysis) are compatible with each other with as clearly indicated. The yield load predicted by the formula developed based on Yield line theory proved to be compatible when compared with results of both physical testing and Finite element numerical modelling.

Hence, both SAP2000 and ABAQUS models could be considered as validated models for required parametric analyses to fine tune the formulae developed for secondary variations that have been omitted (as discussed in Section 4.2) in development based on fundamental theory.

Chapter 5 - Development of design formulae for lattice tower baseplates

5.1 Characteristics of actual tower baseplate

Most of lattice telecommunication/broadcasting towers in present days are having single “L” angle section as tower legs as shown in Fig. 5.1 along with the stiffener plates. Further, legs usually have a slight inclination from the perfect verticality of about 0^0 to 6^0 due to structural configuration of lattice towers. In addition to that, there are shear forces that act on horizontal plane of baseplates due to structural actions of lattice tower systems.

Fig. 5.1 -Typical view of a tower baseplate

Due to these factors, stress distributions among quadrants of a baseplate are not exactly axis-symmetrical in an actual tower. Fig. 5.2 shows a graphical output of stress contours produced by SAP2000 numerical model of an actual baseplate where load transfer through a single “L” leg member with a slight inclination. It is possible to clearly identify the overstressing of one particular quadrant, in which “L” leg member is located, compared with other quadrants.

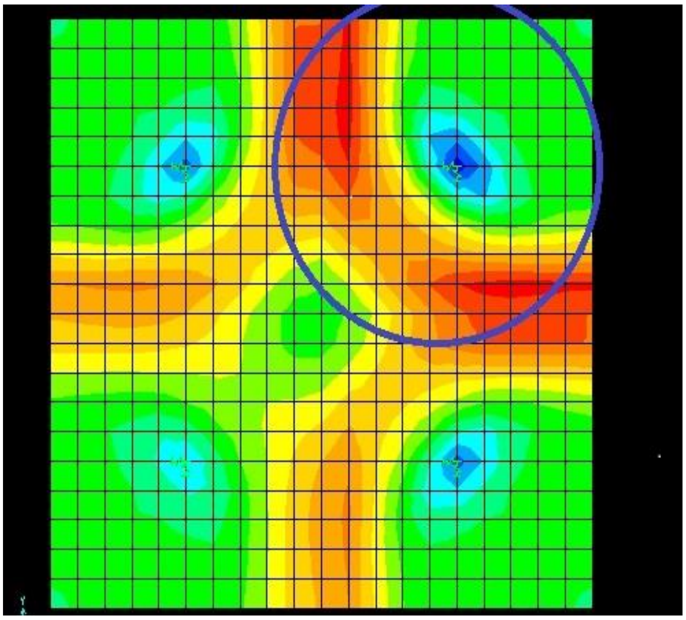


Fig. 5.2 - Stress contours of actual tower baseplate modelled in SAP2000

However, for initial formalization of design theory, these effects had to be neglected as it was done in Section 4.2 due to limitations of yield line theory to take these effects into account. Hence, appropriate modification factor/factors have to be introduced to the already developed formulae to apply it for real practical applications.

5.2 Parametric analysis

It was decided to address this using a parametric study as it would not be able to find a direct solution to determine the appropriate modification factor/factors. The same data set used in Section 4.2 was selected for this study as well. The basic geometrical details of selected baseplates are given in Table 5.1. Yield force of each of the above baseplates based on developed formulae were already known as those were calculated and tabulated in Table 4.1.

Next step was developing two finite element models for each case using SAP2000. In the first model, loading was applied symmetrically, which is identical to the case considered in yield line analyses. In the second model, loading was applied simulating practical scenario. Other than that all material, dimensional and analytical properties were kept as the same in both models. Material characteristics of all of these SAP2000 models were kept as per the already validated model discussed in Section 4.4.2 except overall dimensional properties and element grid for those analyses. Dimensional properties were defined as physical dimensions of selected baseplates and grids were changed as appropriate to suit the overall dimensions.

Table 5.1 Data set of actual baseplates selected for parametric study

No	Baseplate thickness (mm)	b (mm)	r (mm)	r/b	f_y (N/mm^2)
1	3.4	250	50	0.20	275
2	3.4	250	75	0.30	275
3	3.4	250	87.5	0.35	275
4	3.4	250	100	0.40	275
5	15	450	100	0.22	275
6	15	450	125	0.28	275
7	15	450	150	0.33	275
8	15	450	175	0.39	275
9	22	500	125	0.25	275
10	22	500	150	0.30	275
11	22	500	175	0.35	275
12	22	500	200	0.40	275
13	28	550	100	0.18	275
14	28	550	150	0.27	275
15	28	550	200	0.36	275
16	28	550	225	0.41	275

The analyses were carried out as Nonlinear analyses at selected load steps similar to the approach adopted in Section 4.4.2. In each case, yield loads were obtained by monitoring deflection data of top center point of each model (see Fig. 4.13). Load at which gradient of the deflection graph changed was considered as the yield load of the respective case. Deflection graphs of symmetrical and actual load cases of case no.05 of Table 5.1 are shown in Fig. 5.3 as an example. As per Fig. 5.3, yield load of symmetrical and actual load case are 506 kN and 380 kN, respectively for this particular case. Further, element stresses of SAP2000 models under above yield loads were checked to verify identified yield loads as above.

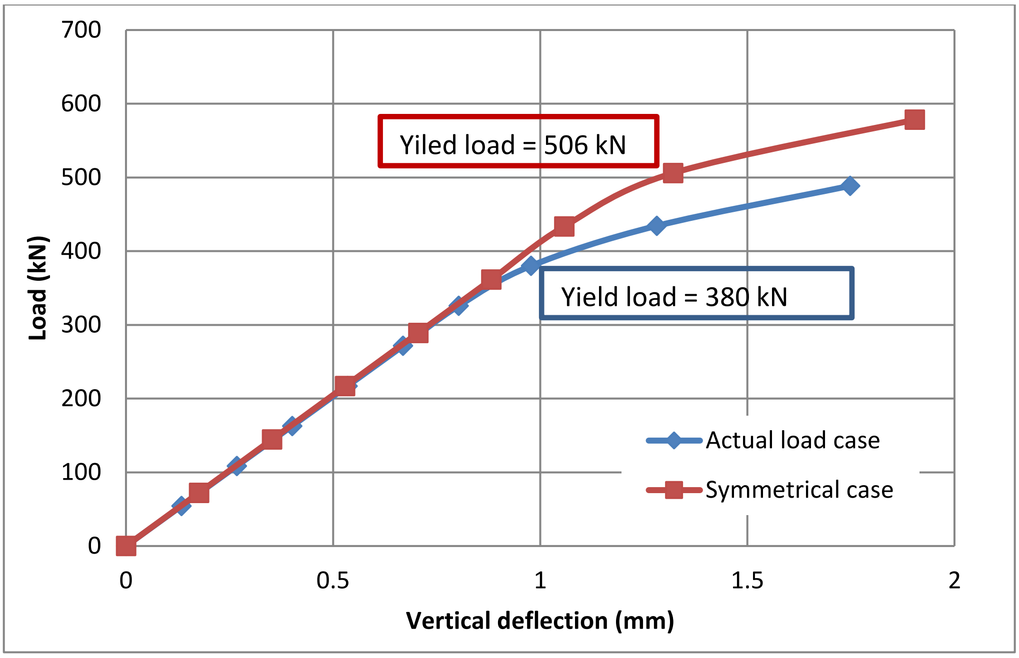


Fig 5.3 – Vertical deformation curves of case no.05 of Table 5.1

5.3 Results and discussion

The results of the parametric analysis are given in Table 5.2. Yield forces of symmetrical models and the models simulating actual practical loading scenarios are given as "YS" and "YA", respectively in Table 5.2. Two (02) dimensionless ratios YS/YF and YA/YF are defined as follows for a clear comparison.

$$\frac{\text{YS}}{\text{YF}} = \frac{\text{Yield force given by SAP models developed as per symmetrical loading condition}}{\text{Yield force given by the formula developed using the yield line theory}} \tag{5.1}$$

$$\frac{\text{YA}}{\text{YF}} = \frac{\text{Yield force given by SAP models developed simulating actual condition of a baseplate}}{\text{Yield force given by the formula developed using the yield line theory}} \tag{5.2}$$

According to results in Table 5.2, it is possible to note that the calculated Yield loads of symmetrical models are higher than the same of numerical models developed representing actual loading scenarios. This is valid for all the cases considered.

Development of higher stresses in the quadrant, in which leg member of the tower was located compared to remaining quadrant was observed in all numerical models developed representing actual loading. The yielding initiated in that particular quadrant in all cases. In other quadrants, the stresses were found to be lower. This could be the reason for observing relatively lower yield forces under actual loading condition in all the cases considered.

Table 5.2 Results of parametric analysis

Base plate thick ness (mm)	b (mm)	r (mm)	r/b	f_y (N/mm^2)	Govern. yieldline formula	Govern. Yield force (kN) from yield line analysis	Yield force given by Symm. SAP model (kN)	Yield force given by SAP model Under Actual loading (kN)	YS/ YF	YA/ YF
						YF	YS	YA		
3.4	250	50	0.20	275	Formula 2	28.82	32.60	22.42	1.13	0.78
3.4	250	75	0.30	275	Formula 3	14.57	15.50	11.21	1.06	0.77
3.4	250	87.5	0.35	275	Formula 3	10.87	12.44	9.34	1.14	0.86
3.4	250	100	0.40	275	Formula 3	8.32	9.33	7.48	1.12	0.90
15	450	100	0.22	275	Formula 2	477.65	506.00	380.00	1.06	0.80
15	450	125	0.28	275	Formula 3	329.11	380.00	272.00	1.15	0.83
15	450	150	0.33	275	Formula 3	232.35	253.00	181.00	1.09	0.78
15	450	175	0.39	275	Formula 3	171.61	181.00	145.00	1.05	0.84
22	500	125	0.25	275	Formula 2	851.57	912.00	642.00	1.07	0.75
22	500	150	0.30	275	Formula 3	610.08	685.00	514.00	1.12	0.84
22	500	175	0.35	275	Formula 3	455.13	513.00	356.00	1.13	0.78
22	500	200	0.4	275	Formula 3	348.29	392.00	285.00	1.13	0.82
28	550	100	0.18	275	Formula 2	2245.30	2320.00	1547.00	1.03	0.69
28	550	150	0.27	275	Formula 3	1190.23	1353.00	967.00	1.14	0.81
28	550	200	0.36	275	Formula 3	684.20	774.00	515.00	1.13	0.75
28	550	225	0.41	275	Formula 3	538.16	560.00	418.00	1.04	0.78

As per results, YS/YF ratios are higher than 1 in all considered cases. This is a good proof to reiterate the accuracy of the developed formulae for the symmetrical case.

In the case of YA/YF, the values are always below the unity and therefore, introduction of correction factors for the yield load calculation formulae is inevitable to use it for practical design applications. Fig. 5.4 shows the variation of YS/YF and YA/YF against r/b ratios of baseplates under whole set of data given in Table 5.1. According to the yield line analysis results, the governing yield line pattern changes from pattern 2 to pattern 3 when r/b ratio is greater than 0.25. Hence, introduction two different correction factors for two cases (r/b ratio is less than or equal to 0.25 and r/b ratio is greater than 0.25) is appropriate.

Therefore, in Fig. 5.5, variations of YS/YF and YA/YF were plotted only considering cases where r/b was greater than 0.25.

When r/b is greater than or equal to 0.25 (which is the common case in actual tower baseplates), the lowest and highest values of YA/YF are 0.75 and 0.90, respectively. Therefore, 0.75 can be selected as the suitable modification factor for this situation. Then, the modification factor for the calculation of the baseplate thickness would be 1.33 since 1/0.75 = 1.33.

When r/b is less than or equal to 0.25, a further drop in both YS/YF and YA/YF ratios can be observed. This may be due to the change in the governing yield line pattern. YA/YF has recorded a value as low as 0.69 (see Fig. 5.4). Hence, 0.69 would be the appropriate modification factor for calculating the yield force under this case. When it comes to the calculation of the thickness of the baseplate, it has to be 1.45, since 1/0.69 = 1.45. However, r/b ratio falling below 0.25 in practice is seldom due to practical implications such as lack of space for bolt tightening etc. under such low r/b ratios.

Hence, final design formulae for calculating the baseplate thickness can thus be presented as given in Eq. 5.3 and 5.4, when design uplift force on the baseplate (F_u) is known. This would be applicable for design of baseplates of towers where inclination of tower legs are less than 6^0 from the perfect verticality and r/b ratio is more than 0.18.

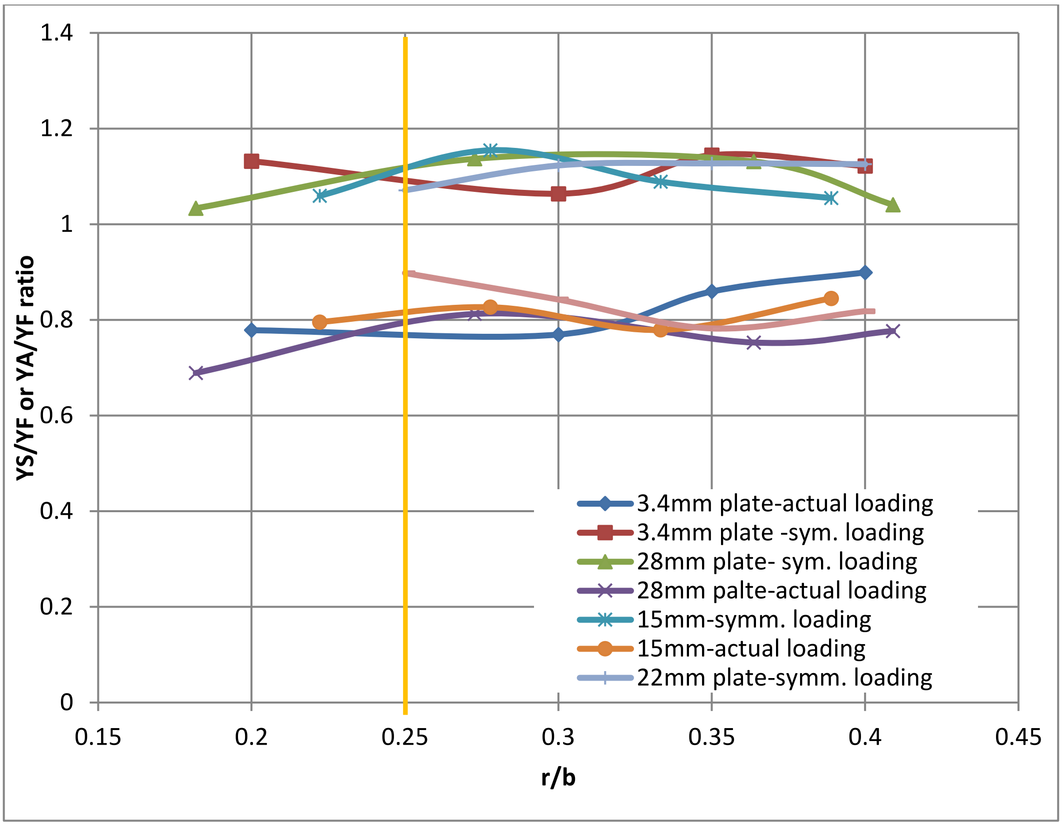


Fig 5.4 - Variation of YS/YF and YA/YF against r/b ratios of baseplates

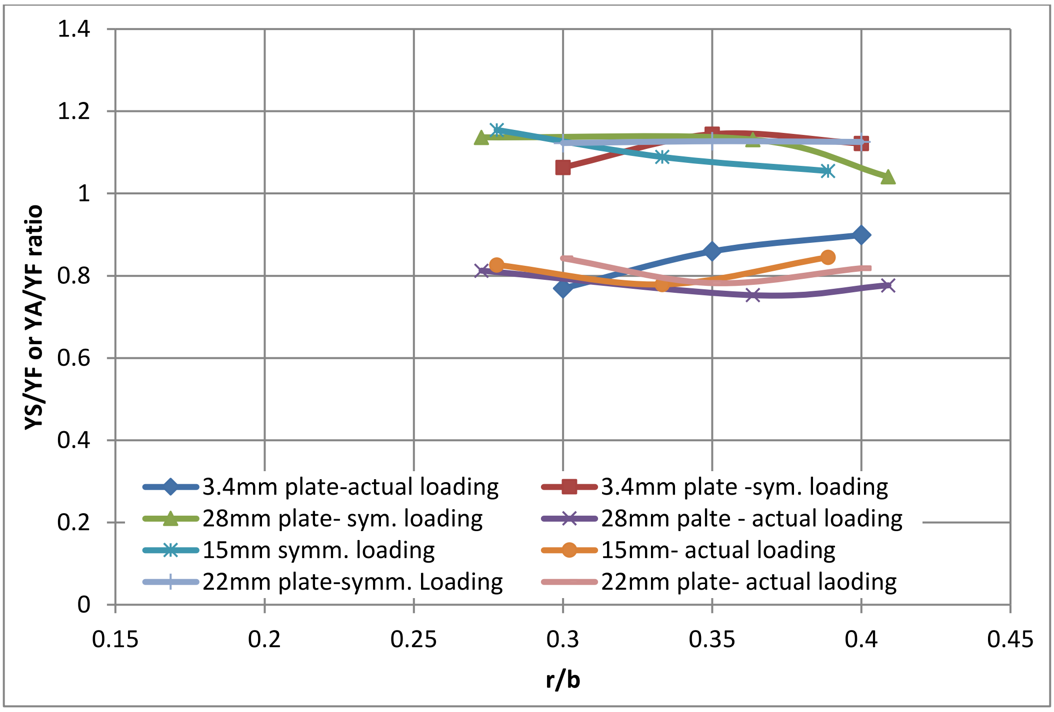


Fig 5.5 - Variation of YS/YF and YA/YF against r/b ratios of baseplates when r/b>0.25

$$t = \left[\frac{1.33F_u \times r}{f_y\left(\sqrt{2}\left(\frac{b}{2}-r\right)+r\right) \times \left(\frac{4\left(\frac{b}{2}-r\right)}{b} + \frac{r\left(\frac{\pi}{2}+2\sin^{-1}\left(\frac{\frac{b}{2}-r}{r}\right)\right)}{\sqrt{2}\left(\frac{b}{2}-r\right)+r} \right)} \right]^{0.5} \qquad (5.3)$$

When r/b > 0.25

$$t = \left[\frac{1.45F_u \times r}{f_y\left(\sqrt{2}\left(\frac{b}{2}-r\right)+r\right) \times \left(\frac{8\left(\frac{b}{2}-r\right)}{b} + \frac{\pi r}{2\left(\sqrt{2}\left(\frac{b}{2}-r\right)+r\right)} \right)} \right]^{0.5} \qquad (5.4)$$

When r/b =< 0.25

Where,

t	=	Thickness of baseplate (mm)
F_u	=	Yield uplift force on baseplate (N)
f_y	=	Characteristic yield strength of the material (N/mm^2)
r	=	Distance to the bolts from the center (mm)
b	=	Width of baseplate (mm)

.

5.4 Simplification of developed formula for design applications

Although it is possible to use eq. 5.3 and 5.4 for design of tower baseplates, those equations are somewhat inconvenient to use in design office practice as certain trigonometric terms and complicated algebraic operations are available in such equations.

Therefore, a Microsoft excel spread sheet was developed for above equations and a screenshot of the spread sheet is given in Fig. 5.6. The required thickness for the baseplate for a particular case can be effortlessly found using the developed spread sheet by entering basic data.

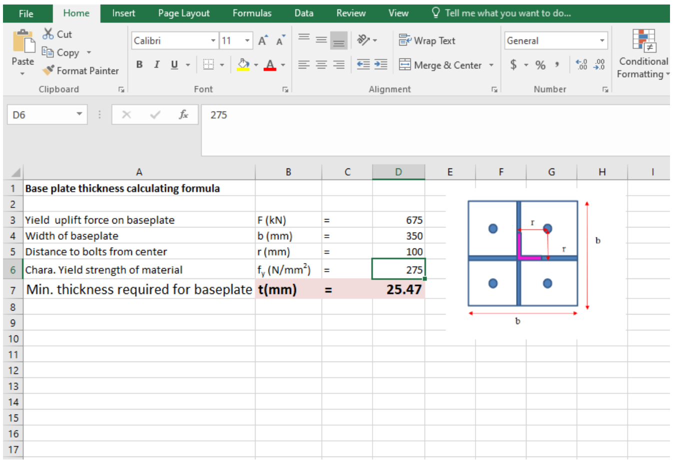


Fig 5.6 – Screenshot of developed Microsoft excel spread sheet to calculate baseplate thickness

Further, development of simplified alternative equations for Eq. 5.3 and 5.4 with acceptable accuracy was also attempted. It was observed liner relationships of $(F_u/f_y)^{0.5}$ terms with "t" in both Eq. 5.3 and 5.4. Therefore, Eq. 5.3 and 5.4 could be rearranged as follows:

$$t = \left(\frac{1.33F_u}{f_y}\right)^{0.5}\left[\frac{r}{\left(\sqrt{2}\left(\frac{b}{2}-r\right)+r\right)\times\left(\frac{4\left(\frac{b}{2}-r\right)}{b}+\frac{r\left(\frac{\pi}{2}+2\sin^{-1}\left(\frac{\frac{b}{2}-r}{r}\right)\right)}{\sqrt{2}\left(\frac{b}{2}-r\right)+r}\right)}\right]^{0.5} \qquad (5.5)$$

$$t = \left(\frac{1.45F_u}{f_y}\right)^{0.5} \left[\frac{r}{\left(\sqrt{2}\,\left(\frac{b}{2} - r\right) + r\right) \times \left(\frac{8\left(\frac{b}{2} - r\right)}{b} + \frac{\pi r}{2\left(\sqrt{2}\left(\frac{b}{2} - r\right) + r\right)}\right)}\right]^{0.5} \tag{5.6}$$

Hence, remaining variable parts of Eq. 5.5 and 5.6 were defined $v1$ and $v2$ respectively, as follows:

$$v1 = \left[\frac{\mathrm{r}}{\left(\sqrt{2}\,\left(\frac{b}{2} - r\right) + r\right) \times \left(\frac{4\left(\frac{\mathrm{b}}{2} - \mathrm{r}\right)}{\mathrm{b}} + \frac{\mathrm{r}\left(\frac{\pi}{2} + 2\sin^{-1}\left(\frac{\frac{\mathrm{b}}{2} - \mathrm{r}}{\mathrm{r}}\right)\right)}{\sqrt{2}\left(\frac{\mathrm{b}}{2} - \mathrm{r}\right) + \mathrm{r}}\right)}\right]^{0.5} \tag{5.7}$$

$$v2 = \left[\frac{\mathrm{r}}{\left(\sqrt{2}\,\left(\frac{b}{2} - r\right) + r\right) \times \left(\frac{8\left(\frac{b}{2} - r\right)}{b} + \frac{\pi r}{2\left(\sqrt{2}\left(\frac{b}{2} - r\right) + r\right)}\right)}\right]^{0.5} \tag{5.8}$$

The variation of $v1$ was plotted with the variation of r/b ratio for the case r/b > 0.25. It is given in Table 5.3. Based on the values of $v1$, it was possible to fit a liner expression for $v1$ with reasonable accuracy and the proposed liner expression is given in Eq. 5.9.

$$v1 \approx 1.595\left({}^{r}/_{b}\right) \tag{5.9}$$

The variation of $v1$ given by Eq. 5.7 and Eq. 5.9 are indicated in Table 5.3. Accordingly, for all r/b ratios considered, the proposed liner expression provided slightly higher values (however, very close to exact values of $v1$) for each case. The values of Table 5.3 are presented in graphical form in Fig. 5.7. As simplified liner equation (given as Eq. 5.9) provided conservative values (will result in slightly higher baseplate thickness) with reasonable accuracy for $v1$, the semi-empirical formula given as Eq. 5.10.b was developed to calculate the baseplate thickness in case of r/b is greater than 0.25.

Table 5.3- Variation of $v1$ of developed formula and linear equation with r/b

r/b	Value of $v1$ from developed formula (Eq. 5.7)	Value of $v1$ from approximate linear equation (Eq. 5.9)
0.252	0.386	0.402
0.260	0.404	0.415
0.275	0.429	0.439
0.300	0.467	0.479
0.325	0.504	0.518
0.350	0.541	0.558
0.375	0.579	0.598
0.400	0.618	0.638
0.425	0.659	0.678
0.450	0.703	0.718
0.475	0.749	0.758
0.500	0.798	0.798

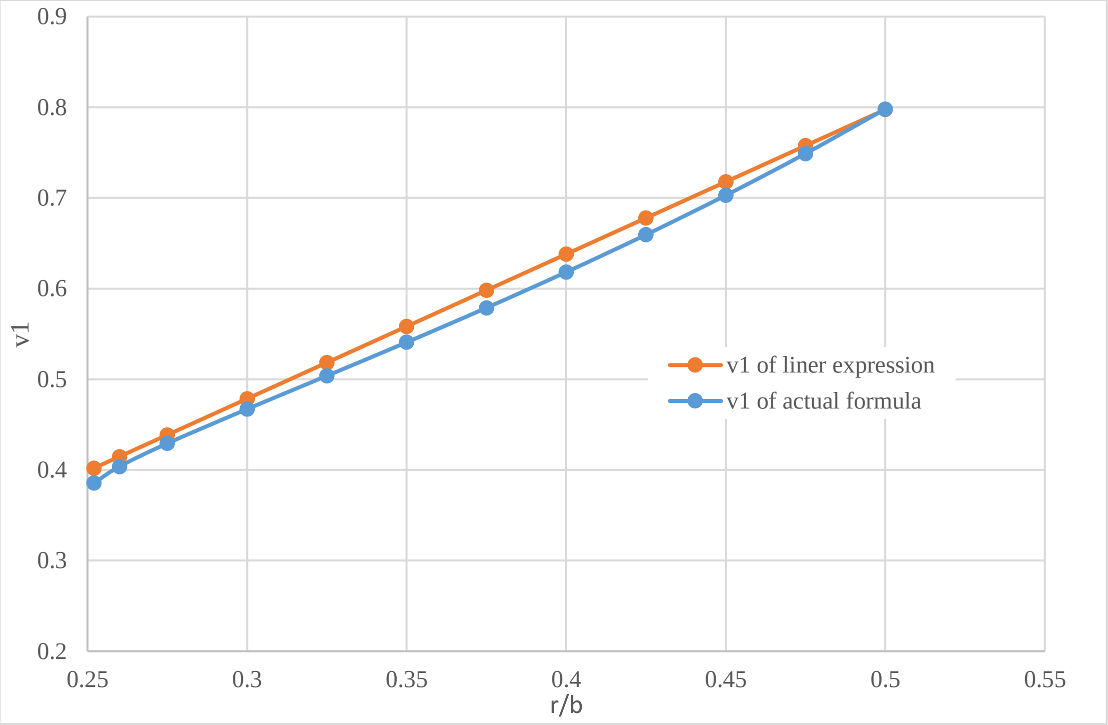


Fig. 5.7- Variation of $v1$ of developed formula and linear equation with r/b in graphical form

$$t = \left(\frac{1.33F_u}{f_y}\right)^{0.5}\left(1.595\left(\frac{r}{b}\right)\right) \qquad (5.10.a)$$

$$t = \left[1.839\left(\frac{r}{b}\right)\right]\left[\frac{F_u}{f_y}\right]^{0.5} \qquad (5.10.b)$$

This simplified semi-empirical formula (Eq. 5.10.b) can be used for calculation of required baseplate thickness for case where 0.25 < r/b with an acceptable accuracy.

The same procedure was followed for $v2$ as well for the case where r/b is less than or equal to 0.25. Variation of $v2$ in considered range is given in Table 5.4 and graphical representation of same is given in Fig. 5.8. Based on those values, it was possible to fit a different liner expression for $v2$ with a reasonable accuracy and the proposed liner expression is given in Eq. 5.11.

$$v2 \approx 1.236\left({}^{r}/_{b}\right) + 0.09 \qquad (5.11)$$

Table 5.4 - Variation of $v2$ of developed formula and linear equation with r/b

r/b	Value of v2 from developed formula (Eq. 5.8)	Value of v2 from approximate linear equation (Eq. 5.11)
0.100	0.252	0.253
0.125	0.290	0.290
0.150	0.326	0.328
0.175	0.363	0.365
0.200	0.400	0.402
0.225	0.438	0.439
0.250	0.476	0.477

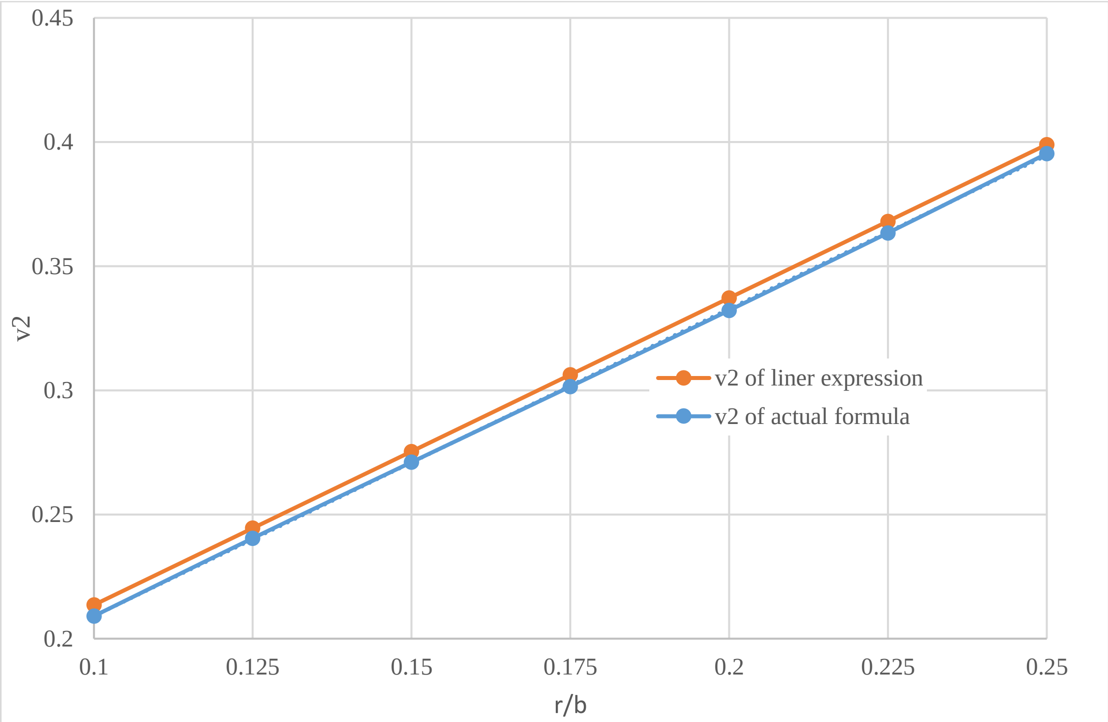


Fig. 5.8- Variation of $v2$ of developed formula and linear equation with r/b in graphical form

Therefore, the semi-empirical formula proposed for the case where r/b is less than or equal to 0.25 would be the following:

$$t = \left(\frac{1.45F_u}{f_y}\right)^{0.5}\left(1.236\left(\frac{r}{b}\right) + 0.09\right) \qquad (5.12.a)$$

$$t = \left[1.488\left(\frac{r}{b}\right) + 0.108\right]\left[\frac{F_u}{f_y}\right]^{0.5} \qquad (5.12.b)$$

This simplified semi-empirical formula (Eq. 5.12.b) can be used for calculation of the baseplate thickness required for cases where r/b ratio is less than or equal to 0.25 with acceptable accuracy.

Therefore, Eq. 5.10.b and 5.12.b can be used to calculate the minimum baseplate thickness for tower baseplates in case of r/b > 0.25 and r/b =< 0.25 respectively, instead of exact complicated theoretical expressions given as Eq. 5.3 and 5.4.

5.5 Summary

Modification of the basic design methodology developed based on Yield line theory for practical design applications of tower baseplates was focused here. Since yield line analysis did not include some deviations that would be present in actual practice, it was necessary to effect the necessary modifications to capture such peculiarities to ensure that the derived formula could be used for structural design purposes.

Accordingly, a comprehensive parametric analysis was carried out using set of data for actual baseplates. Each baseplate was modelled under a symmetrical system (similar to the condition assumed in Yield line analysis) and under the actual condition present in real scenario in the field by developing two separate numerical models for nonlinear analyses using SAP2000. Based on results of this parametric analysis, necessary modification factors that have to be assigned for the fundamental design formulae were found.

Further, simplified semi-empirical formulae were also developed for the design office usage based on accurate design formulae.

Chapter 6 - Conclusion and Recommendations

6.1 Conclusion

This dissertation has focused on two main aspects. One aspect is assessment of structural performance of the telecommunication/broadcasting towers under seismic loading. The other is the development of an effective design guideline for typical tower baseplates commonly used in telecommunication towers.

A comprehensive study on structural performance of lattice telecommunication/ broadcasting towers under seismic loads was carried out using selected Three leg and Four leg self-supporting towers. These towers were carefully selected to cover a whole range of heights. In case of seismic hazard levels, PGA values from 0.1 g to 0.43 g were considered to cover the whole range of regional seismic intensities of South Asian region. Sub soil stratums, which influence the intensity of seismic forces on structures, were also addressed in the study by selecting three different sub soil classes specified in ANSI/TIA-222-G in addition to the towers located over firm rock.

The structural assessment carried out using Response Spectrum analysis indicated that for the towers designed for the expected wind loading can perform well under probable magnitudes of earthquakes when located over firm soil conditions. One of the key observations for seismic effects is the influence of subsoil properties. The subsoil stratums can influence the shape of the response spectrum curves. These changes indicated considerable effects on stress and deformation of towers. Vibration characteristics (especially the Natural period of vibration of first mode) of respective towers also played a role on this. It is found that the influence of sub soil stratums on seismic performance of a particular tower cannot be directly predicted based on changes of Response spectrum curves. Significant increase of stresses and deformations in certain towers were observed especially under weaker subsoil stratum. However, those stresses and deformations did not exceed the same under wind loading. Nevertheless, it is always advisable to check seismic performance of important towers constructed especially on weak subsoil stratums. For the whole range of seismic hazard levels, the Static equivalent approach specified in ANSI/TIA-222-G gave conservative results.

Development of an effective design guideline for typical tower baseplates was the other main focused area of this study as presently, a direct design methodology for design of tower baseplates is not available in design codes or standard text books. Uplift forces that would act on tower baseplates of self supporting lattice towers always govern the design of the baseplate. Hence, design methodologies used for deign of baseplates of buildings would not be applicable here as those were not developed based on uplift forces. Accordingly, for development of a design guideline for tower baseplates with four anchor bolts with "L" angle leg members (along with crucified stiffener plates) was undertaken. This is the most commonly used baseplate arrangement of this type of towers, Yield line theory was used to develop basic formulae. Idealised symmetrical system was used initially for the yield line analyses, even though it was not a perfectly symmetrical system in reality. The deviation from perfect symmetricity and other secondary effects were later addressed by necessary modification factors. In the analysis, one key parameter related was the distance to anchor bolts from symmetrical axis, "r". The other is the width of the baseplate, "b". Three different Yield line patterns showed the potential for governing the failure. Out of these, two patterns were found to be governing. When r/b =< 0.25, Yield line pattern 2 governed. Otherwise, Yield line pattern 3 governed. Hence, two design formulae have been proposed.

The accuracy of the developed yield line formulae were first checked through sufficient physical model testing with monitoring of strains and deflections at control points. For further verification, Nonliner Finite element modelling of the system was carried out through two Finite elements software programmes (SAP2000 and ABAQUS). Outputs of Finite element models were compared with the strain and deflection data of control points of physical models. An acceptable matching was observed. Deformation patterns and yield loads of physical test models and finite element models were compared with yield line analysis. An acceptable compatibility was noticed with each other. Therefore, accuracy of developed formulae was well justified.

Nevertheless, it was necessary to modify the formula developed to suit for the practical applications as idealised symmetrical system used in Yield line analyses will not exactly simulate the practical condition of typical tower baseplate as explained previously. Therefore, fine tuning of the fundamental formula for these effects was carried out. This was executed through a study based on finite element modelling of selected baseplates of actual towers. During the parametric analysis, Non linear Finite element analyses of each selected baseplates (16 Nos. of baseplates) were carried out and results of the analyses were compared with the Yield forces given by the formulae developed. Based on result, two (02) modification factors were successfully identified to modify formulae for practical design applications. The final design formulae to find appropriate thickness for baseplates are as follows;

$$t = \left[\frac{1.33\mathrm{F_u} \times \mathrm{r}}{f_y\left(\sqrt{2}\ \left(\frac{b}{2} - r\right) + r\right) \times \left(\frac{4\left(\frac{\mathrm{b}}{2} - \mathrm{r}\right)}{\mathrm{b}} + \frac{\mathrm{r}\left(\frac{\pi}{2} + 2\sin^{-1}\left(\frac{\frac{\mathrm{b}}{2} - \mathrm{r}}{\mathrm{r}}\right)\right)}{\sqrt{2}\left(\frac{\mathrm{b}}{2} - \mathrm{r}\right) + \mathrm{r}}\right)}\right]^{0.5}$$

When r/b > 0.25

$$t = \left[\frac{1.45F_u \times r}{f_y\left(\sqrt{2}\ \left(\frac{b}{2} - r\right) + r\right) \times \left(\frac{8\left(\frac{b}{2} - r\right)}{b} + \frac{\pi r}{2\left(\sqrt{2}\left(\frac{b}{2} - r\right) + r\right)}\right)}\right]^{0.5}$$

When r/b =< 0.25

However, for design office applications, following two semi-empirical formulae can also be used to find appropriate baseplate thickness with an acceptable accuracy:

When r/b > 0.25

$$t = \left[1.839\left(\frac{r}{b}\right)\right]\left[\frac{F_u}{f_y}\right]^{0.5}$$

When r/b =< 0.25

$$t = \left[1.488\left(\frac{r}{b}\right) + 0.108\right]\left[\frac{F_u}{f_y}\right]^{0.5}$$

6.2 Recommendations

Based on the findings of this study, satisfactory seismic performances of self-supporting lattice towers were identified when designed for the recommended wind loading. However, studying of seismic performances of other structural forms such as Monopoles, Guy masts, etc. would be an interesting area for future studies as especially a monopole would behave as an inverted pendulum in such case.

The outcome of this study relevant to baseplate of towers had ended up with an appropriate design guideline for tower baseplates with four bolts and crucified stiffeners by fulfilling an essential industrial need. However, few other types of baseplate configurations are also used for towers. Developing same type of design guidelines for such configurations would be another recommended area to focus in future studies.

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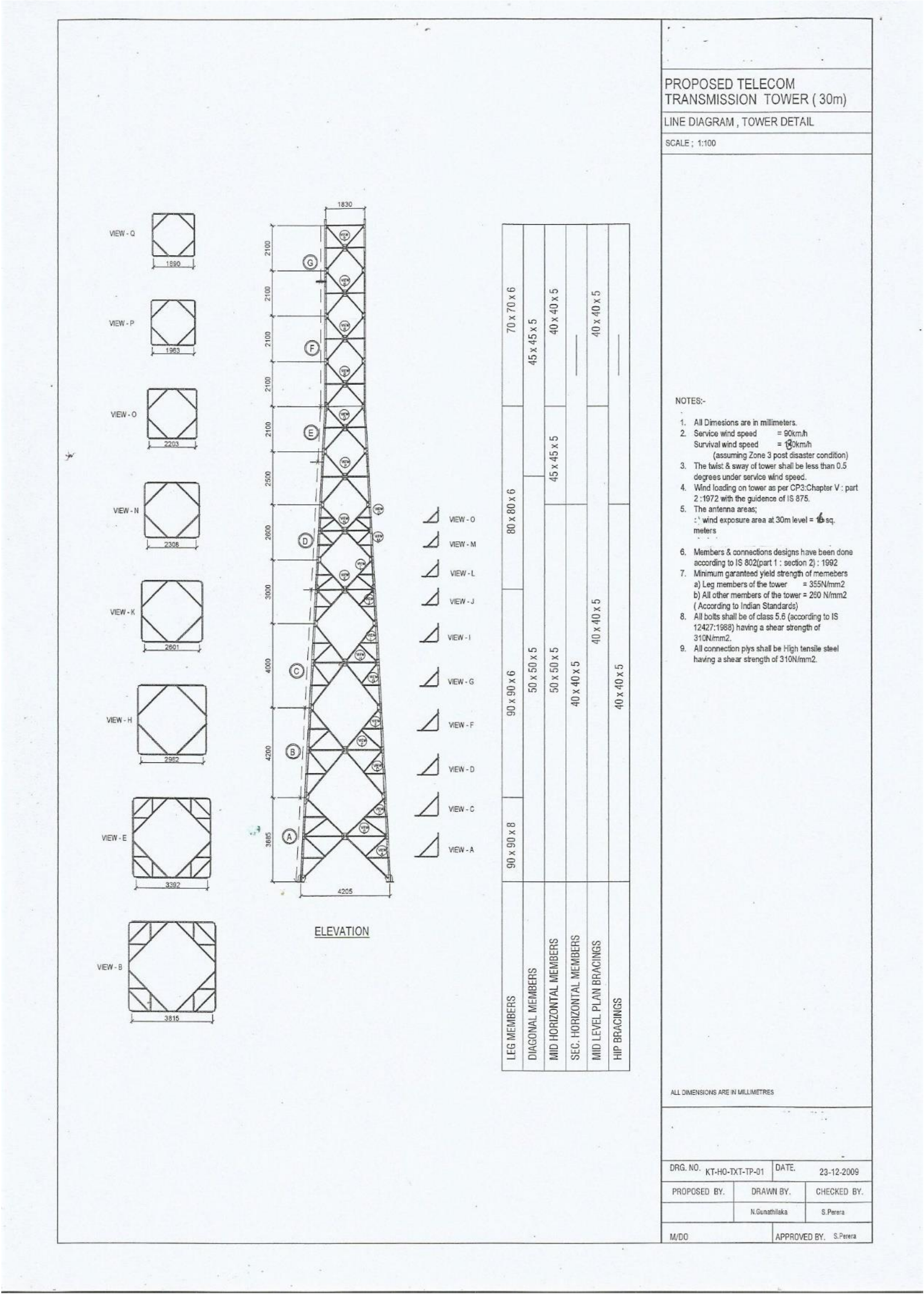
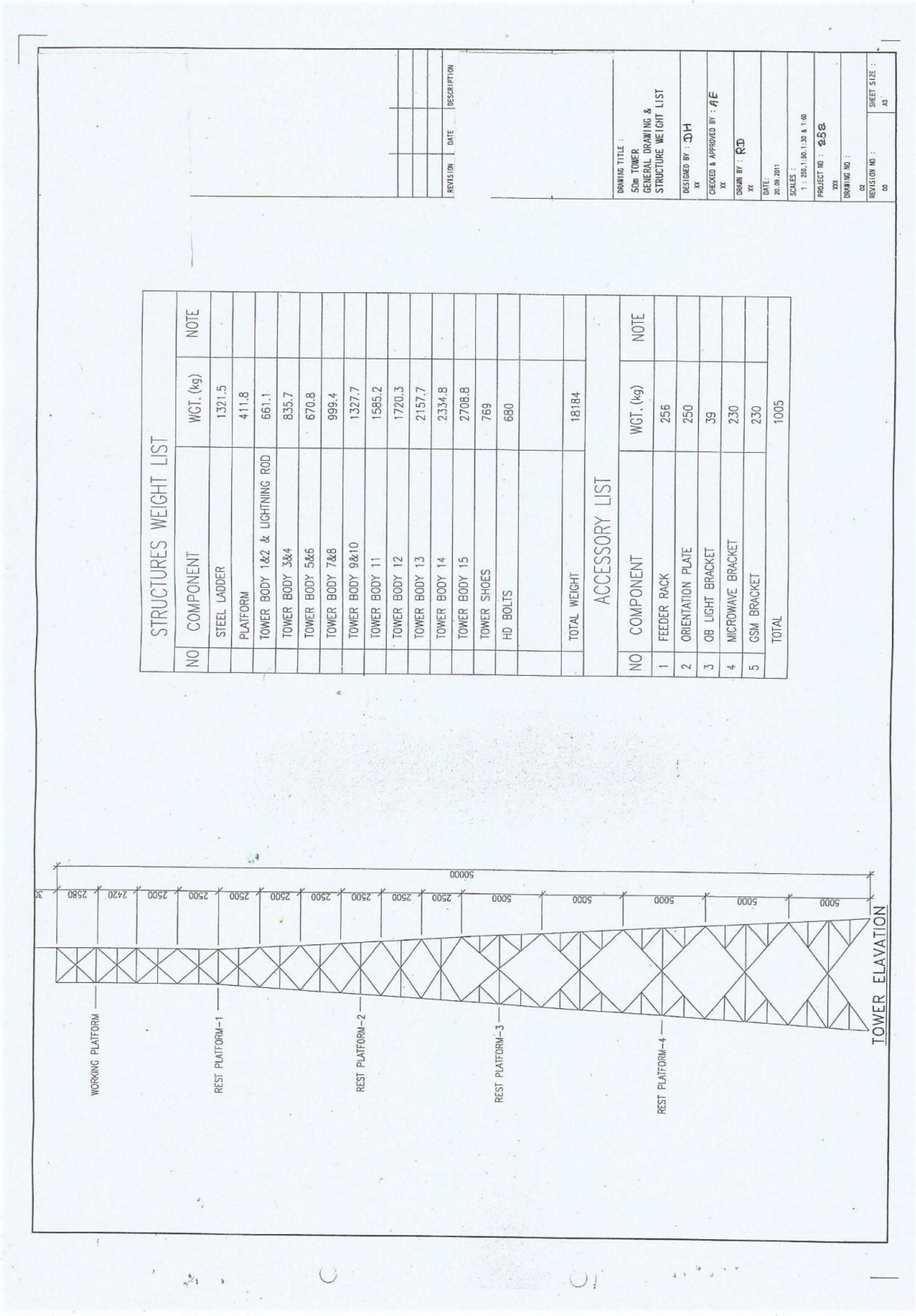
Annexure 1 - Drawings of towers

1. 30m Four leg tower drawing
2. 50m Four leg tower drawings
3. 80m Four leg tower drawings
4. 30m Three leg tower drawing
5. 45m Three leg tower drawing
6. 60m Three leg tower drawing

1. 30m Four leg tower drawing



2. 50m four leg tower drawings

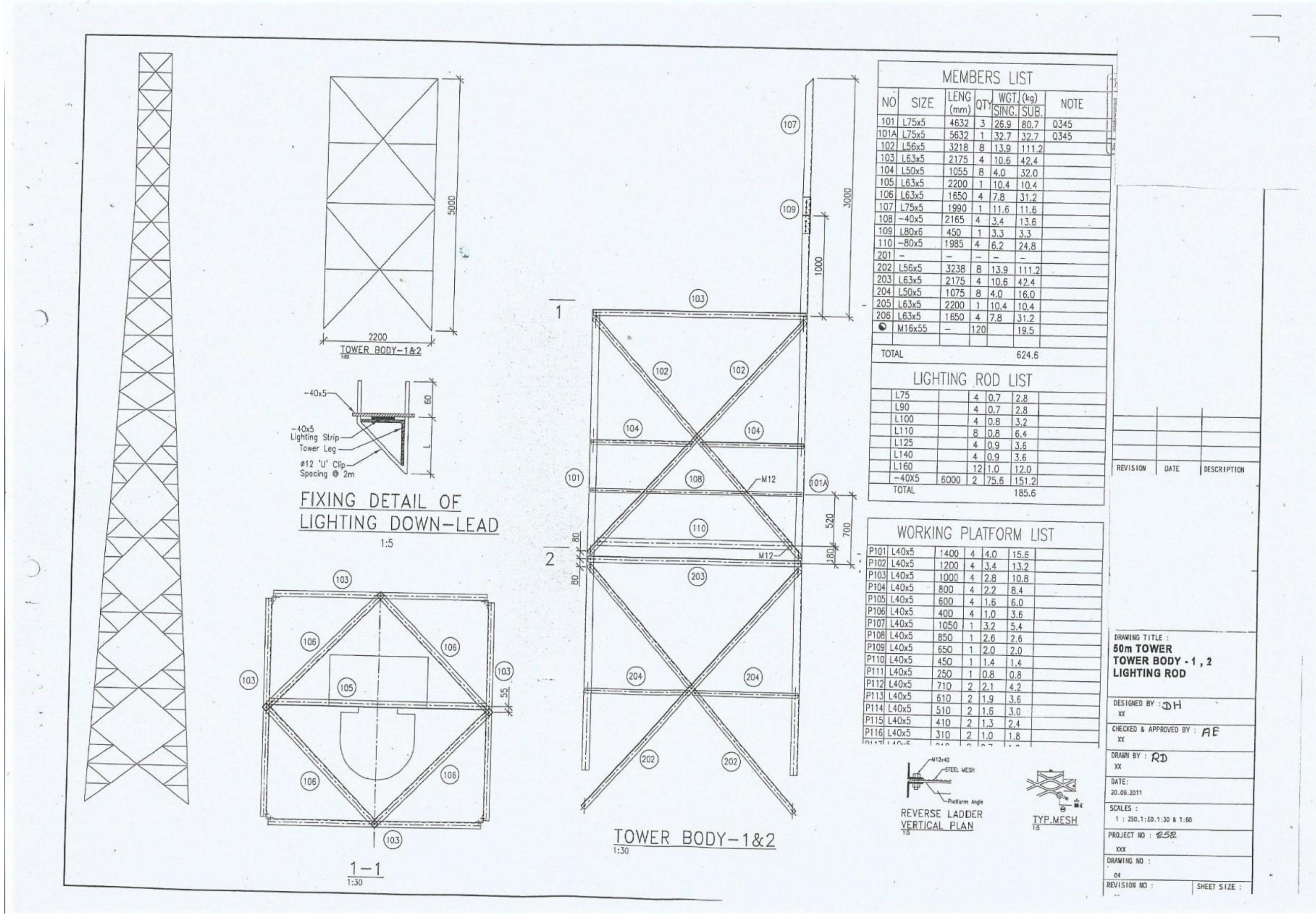


STRUCTURES WEIGHT LIST

NO	COMPONENT	WGT. (kg)	NOTE
	STEEL LADDER	1321.5	
	PLATFORM	411.8	
	TOWER BODY 1&2 & LIGHTNING ROD	661.1	
	TOWER BODY 3&4	835.7	
	TOWER BODY 5&6	670.8	
	TOWER BODY 7&8	999.4	
	TOWER BODY 9&10	1327.7	
	TOWER BODY 11	1585.2	
	TOWER BODY 12	1720.3	
	TOWER BODY 13	2157.7	
	TOWER BODY 14	2334.8	
	TOWER BODY 15	2708.8	
	TOWER SHOES	769	
	HD BOLTS	680	
	TOTAL WEIGHT	18184	

ACCESSORY LIST

NO	COMPONENT	WGT. (kg)	NOTE
1	FEEDER RACK	256	
2	ORIENTATION PLATE	250	
3	OB LIGHT BRACKET	39	
4	MICROWAVE BRACKET	230	
5	GSM BRACKET	230	
	TOTAL	1005	



MEMBERS LIST

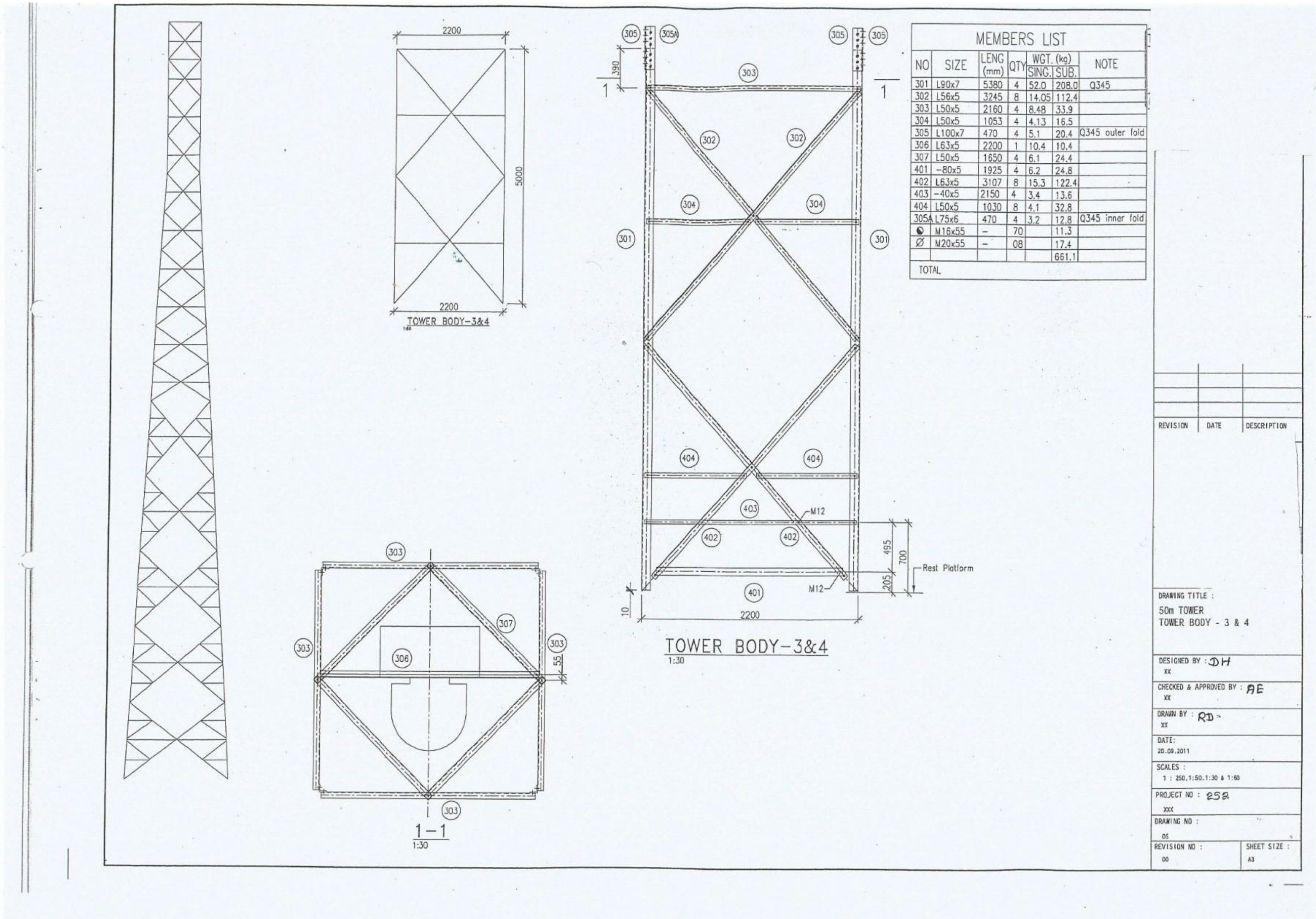
NO	SIZE	LENG (mm)	QTY	WGT (kg) SING.	WGT (kg) SUB.	NOTE
101	L75x5	4632	3	26.9	80.7	Q345
101A	L75x5	5632	1	32.7	32.7	Q345
102	L56x5	3218	8	13.9	111.2	
103	L63x5	2175	4	10.6	42.4	
104	L50x5	1055	8	4.0	32.0	
105	L63x5	2200	1	10.4	10.4	
106	L63x5	1650	4	7.8	31.2	
107	L75x5	1990	1	11.6	11.6	
108	-40x5	2165	4	3.4	13.6	
109	L80x6	450	1	3.3	3.3	
110	-80x5	1985	4	6.2	24.8	
201	-	-	-	-	-	
202	L56x5	3238	8	13.9	111.2	
203	L63x5	2175	4	10.6	42.4	
204	L50x5	1075	8	4.0	16.0	
205	L63x5	2200	1	10.4	10.4	
206	L63x5	1650	4	7.8	31.2	
●	M16x55	-	120		19.5	
TOTAL					624.6	

LIGHTING ROD LIST

L75		4	0.7	2.8	
L90		4	0.7	2.8	
L100		4	0.8	3.2	
L110		8	0.8	6.4	
L125		4	0.9	3.6	
L140		4	0.9	3.6	
L160		12	1.0	12.0	
-40X5	6000	2	75.6	151.2	
TOTAL				185.6	

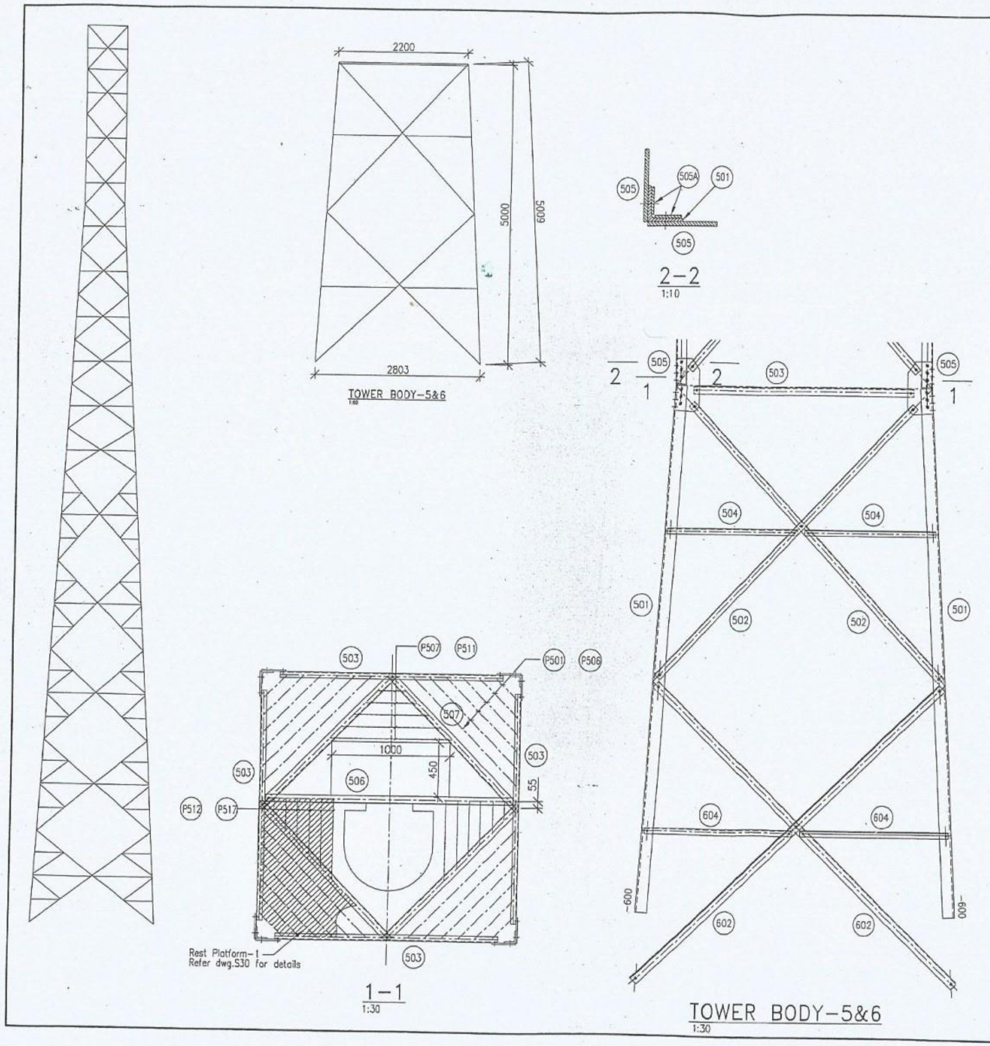
WORKING PLATFORM LIST

P101	L40x5	1400	4	4.0	15.6
P102	L40x5	1200	4	3.4	13.2
P103	L40x5	1000	4	2.8	10.8
P104	L40x5	800	4	2.2	8.4
P105	L40x5	600	4	1.6	6.0
P106	L40x5	400	4	1.0	3.6
P107	L40x5	1050	1	3.2	5.4
P108	L40x5	850	1	2.6	2.6
P109	L40x5	650	1	2.0	2.0
P110	L40x5	450	1	1.4	1.4
P111	L40x5	250	1	0.8	0.8
P112	L40x5	710	2	2.1	4.2
P113	L40x5	610	2	1.9	3.6
P114	L40x5	510	2	1.6	3.0
P115	L40x5	410	2	1.3	2.4
P116	L40x5	310	2	1.0	1.8
P117	L40x5	[illegible]	[illegible]	[illegible]	[illegible]



MEMBERS LIST

NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	WGT. (kg) SUB.	NOTE
301	L90x7	5380	4	52.0	208.0	Q345
302	L56x5	3245	8	14.05	112.4	
303	L50x5	2160	4	8.48	33.9	
304	L50x5	1053	4	4.13	16.5	
305	L100x7	470	4	5.1	20.4	Q345 outer fold
306	L63x5	2200	1	10.4	10.4	
307	L50x5	1650	4	6.1	24.4	
401	–80x5	1925	4	6.2	24.8	
402	L63x5	3107	8	15.3	122.4	
403	–40x5	2150	4	3.4	13.6	
404	L50x5	1030	8	4.1	32.8	
305A	L75x6	470	4	3.2	12.8	Q345 inner fold
●	M16x55	–	70		11.3	
Ø	M20x55	–	08		17.4	
TOTAL					661.1	



MEMBERS LIST

NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	WGT. (kg) SUB.	NOTE
501	L100x10	4418	4	66.8	267.2	Q345
502	L63x5	3170	4	15.2	60.8	
503	L63x5	1890	4	9.1	36.4	
504	L50x5	1120	4	4.2	16.8	
505	-12	440x220	8	9.1	72.8	Q345 outer fold
506	L63x5	2090	4	10.1	40.4	
507	L63x5	1570	4	7.6	30.4	
601	-	-	-	-	-	
602	L70x5	3565	4	17.2	68.8	
603	-	-	-	-	-	
604	L50x5	1267	4	4.8	19.2	
505A	-8x75	440	8	3.1	24.8	Q345 inner fold
◐	M16x55	-	52		84	
Ø	M20x55	-	88		24.8	
TOTAL					746.4	

REST PLATFORM 4 LIST

P501	L40x5	1310	4	3.9	15.6	
P502	L40x5	1110	4	3.3	13.2	
P503	L40x5	910	4	2.7	10.8	
P504	L40x5	710	4	2.1	8.4	
P505	L40x5	510	4	1.5	6.0	
P506	L40x5	310	4	0.9	3.6	
P507	L40x5	1080	1	5.4	5.4	
P508	L40x5	880	1	2.6	2.6	
P509	L40x5	680	1	2.0	2.0	
P510	L40x5	480	1	1.4	1.4	
P511	L40x5	280	1	0.8	0.8	
P512	L40x5	695	2	2.1	4.2	
P513	L40x5	595	2	1.8	3.6	
P514	L40x5	495	2	1.5	3.0	
P515	L40x5	395	2	1.2	2.4	
P516	L40x5	295	2	0.9	1.8	
P517	L40x5	195	2	0.6	1.2	
●	M12x45	-	84		3.3	
TOTAL					89.3	

REVISION	DATE	DESCRIPTION

DRAWING TITLE :
50m TOWER
TOWER BODY - 5 & 6

DESIGNED BY : DH
XX

CHECKED & APPROVED BY : AE
XX

DRAWN BY : RD
XX

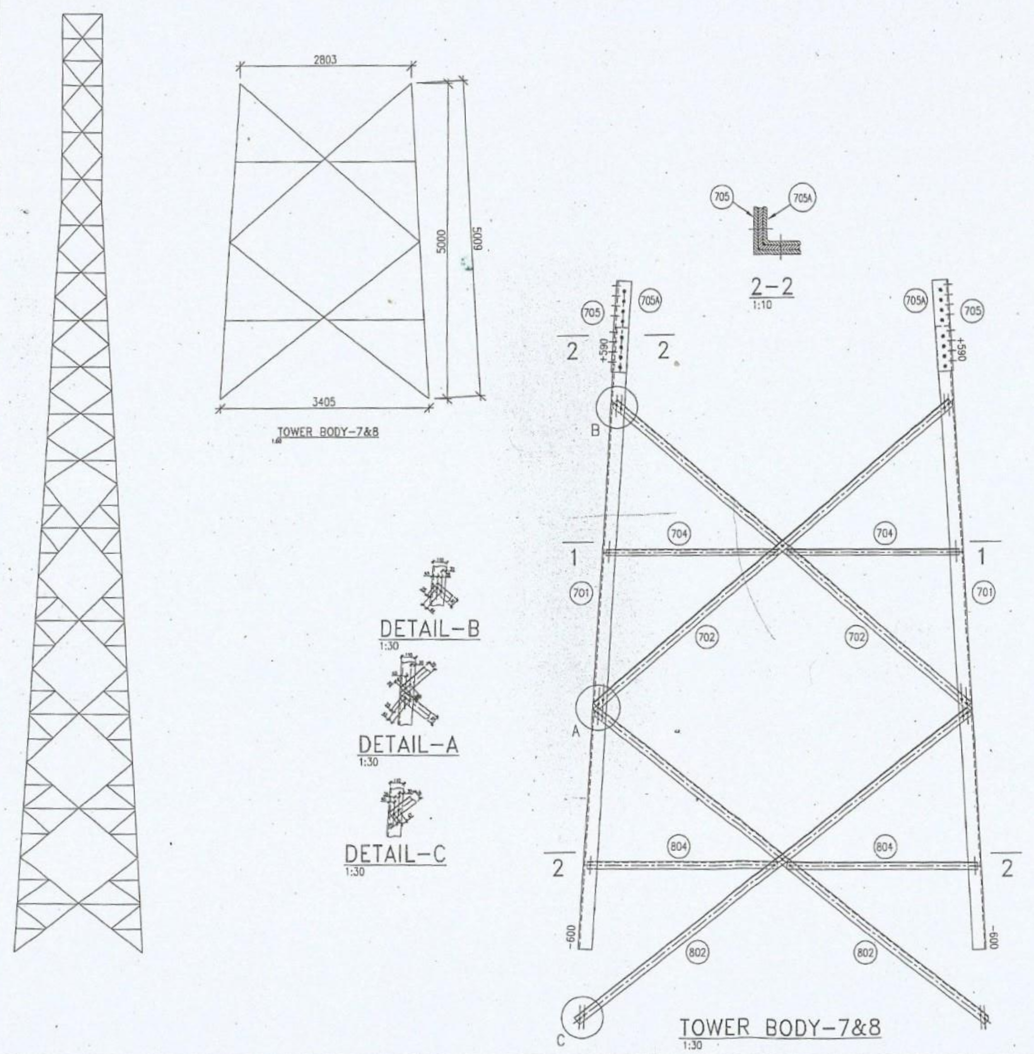
DATE:
20.09.2011

SCALES :
1 : 250,1:50,1:30 & 1:60

PROJECT NO : 252
XXX

DRAWING NO :
06

REVISION NO : 00
SHEET SIZE : A3



MEMBERS LIST

NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	WGT. (kg) SUB.	NOTE
701	L110x10	5009	4	83.6	334.4	Q345
702	L70x5	3797	8	20.5	164.0	
703	–	–	–	–	–	
704	L50x5	1410	8	5.3	42.4	
705	L125x10	590	4	11.3	45.2	Q345 outer fold
706	L63x5	1925	1	9.3	9.3	
801	–	–	–	–	–	
802	L70x5	4040	8	21.8	174.4	
803	–	–	–	–	–	
804	L56x5	1560	8	6.6	52.8	
805	L63x5	1825	2	8.8	17.6	
806	L63x5	3088	1	14.9	14.9	
807	L70x5	2136	4	11.5	46.0	
808	L50x5	1800	1	6.8	6.8	
705A	L100x10	590	4	9.8	39.2	Q345 inner fold
◐	M16x55	–	194		31.4	
Ø	M20x70	–	64		20.9	
TOTAL					999.3	

REVISION	DATE	DESCRIPTION

DRAWING TITLE :
50m TOWER
TOWER BODY - 7 &8

DESIGNED BY : DH
XX

CHECKED & APPROVED BY : AE
XX

DRAWN BY : RD
XX

DATE:
20.09.2011

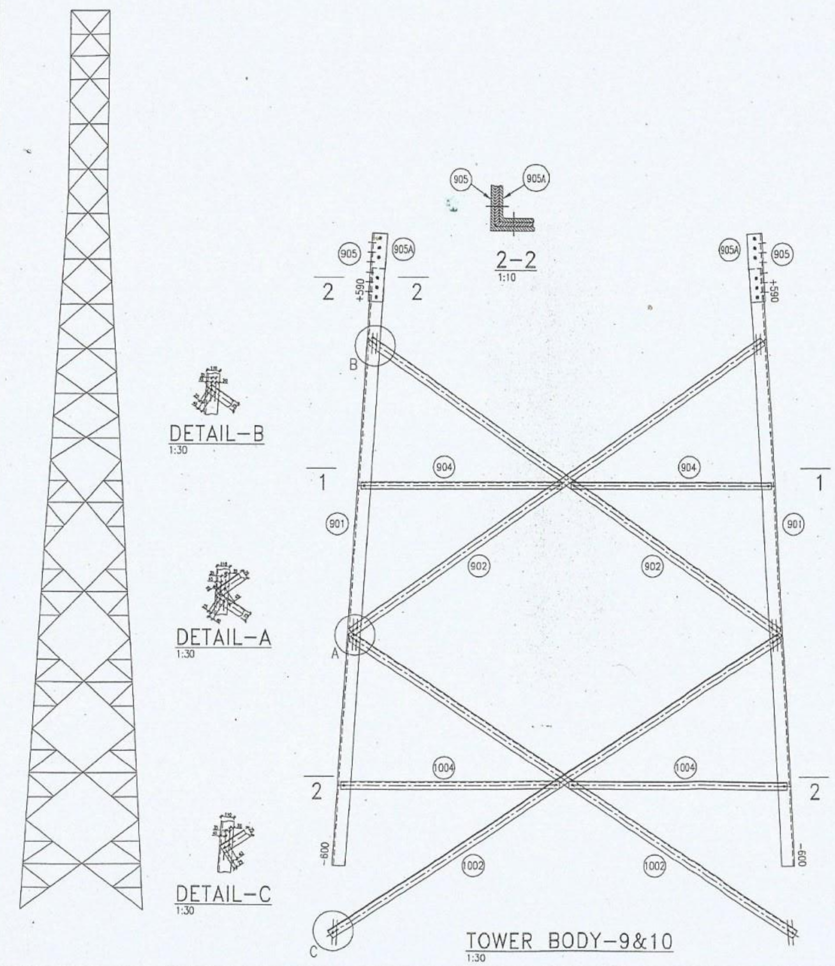
SCALES :
1 : 250,1:50,1:30 & 1:80

PROJECT NO : 252
XXX

DRAWING NO :
07

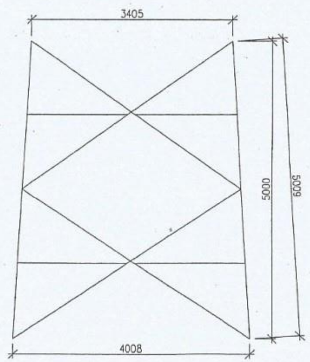
REVISION NO :
00

SHEET SIZE :
A3



MEMBERS LIST

NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	SUB.	NOTE
901	L110x12	5009	4	99.2	396.8	Q345
902	L75x5	4275	8	25.0	200.0	
903	–	–	–	–	–	
904	L56x5	1707	8	7.3	58.4	
905	L125x12	570	4	12.9	51.6	Q345 outer fold
906	L75x5	2350	4	13.7	54.8	
907	L50x5	2126	2	8.0	16.0	
908	–	–	–	–	–	
909	L50x5	3390	1	12.8	12.8	
1002	L75x6	4527	8	31.2	249.6	
1004	L63x5	1856	8	8.9	71.2	
1005	L63x5	2562	4	12.3	49.2	
1006	L50x5	2426	2	9.1	18.2	
1007	L63x5	3690	1	17.8	17.8	
1008	L50x5	950	4	3.6	14.4	
1009	L50x5	1306	4	4.9	19.6	
1010	L50x5	922	4	3.5	14.0	
905A	L100x10	570	4	8.6	34.4	Q345 inner fold
◐	M16x55	–	138		22.4	
Ø	M24x75	–	48		26.5	
TOTAL					1327.7	



TOWER BODY-9&10
1:60

REVISION	DATE	DESCRIPTION

DRAWING TITLE :
50m TOWER
TOWER BODY - 9 & 10

DESIGNED BY : DH
XX

CHECKED & APPROVED BY : AE
XX

DRAWN BY : RD
XX

DATE:
20.09.2011

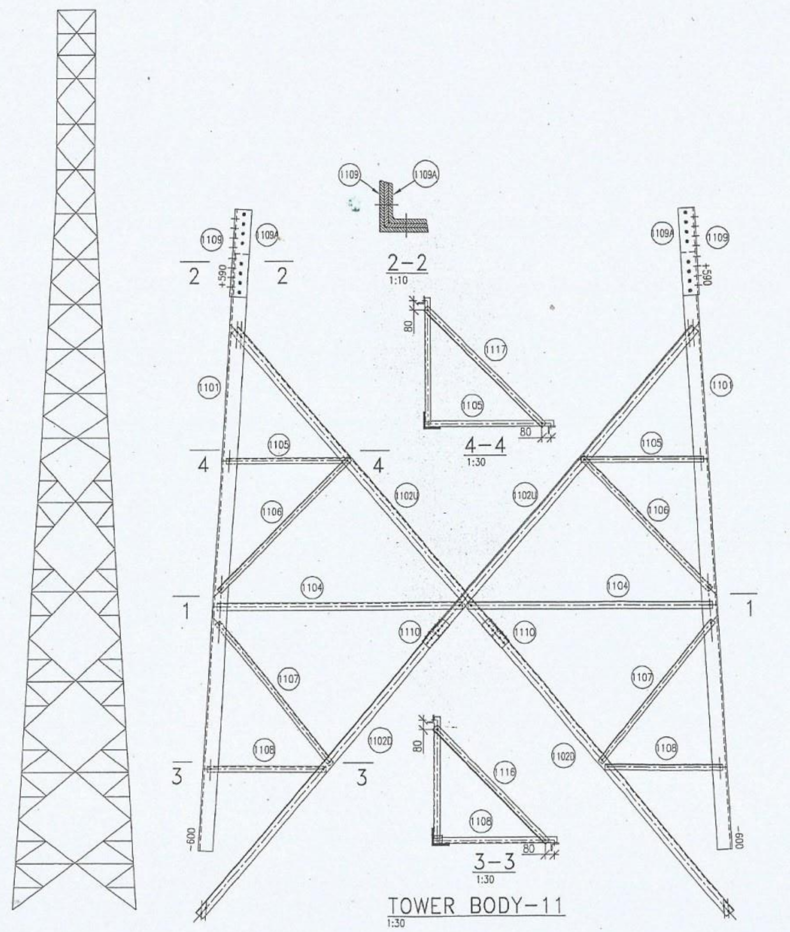
SCALES :
1 : 250,1:50,1:30 & 1:60

PROJECT NO : 252
XXX

DRAWING NO :
09

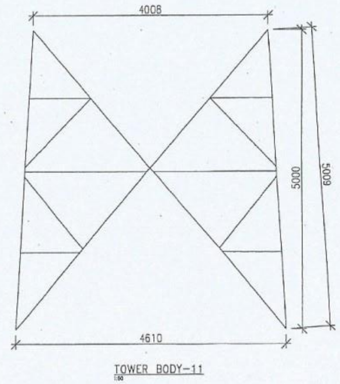
REVISION NO :
00

SHEET SIZE :
A3



MEMBERS LIST

NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	WGT. (kg) SUB.	NOTE
1101	L125x12	5009	4	113.7	454.8	Q345
1102U / 1102D	L80x7	3388 / 3065	8 / 8	28.9 / 26.8	231.2 / 214.4	
1103	–	–	–	–	–	
1104	L63x5	2083	8	10.0	80.0	
1105	L50x5	1050	8	4.0	32.0	
1106	L50x5	1570	8	5.9	47.2	
1107	L50x5	1520	8	5.7	45.6	
1108	L50x5	1005	8	3.8	30.4	
1109	L140x12	710	4	18.1	72.4	Q345 outer fold
1110	L90x7	260	8	2.5	20.0	outer fold
1111	L63x5	2875	4	13.9	55.6	
1112	L63x5	1800	1	12.5	12.5	
1113	L50x5	1030	4	3.9	15.6	
1114	L50x5	1070	4	4.0	16.0	
1115	L50x5	1462	4	5.5	22.0	
1116	L56x5	1343	4	7.1	28.4	
1117	L56x5	1410	4	7.1	28.4	
1118	L63x5	2872	2	13.8	27.6	
1119	L63x5	4133	1	19.9	19.9	
1120	–	–	–	–	–	
1121	L63x5	1800	1	8.7	8.7	
1109A	L110x10	710	4	11.8	47.2	Q345 inner fold
◐	M16x55	–	192		31.1	
◎	M24x80	–	64		44.2	
TOTAL					1585.2	



REVISION	DATE	DESCRIPTION

DRAWING TITLE :
50m TOWER
TOWER BODY - 11

DESIGNED BY : DH
xx

CHECKED & APPROVED BY : AE
xx

DRAWN BY : RD
xx

DATE:
20.09.2011

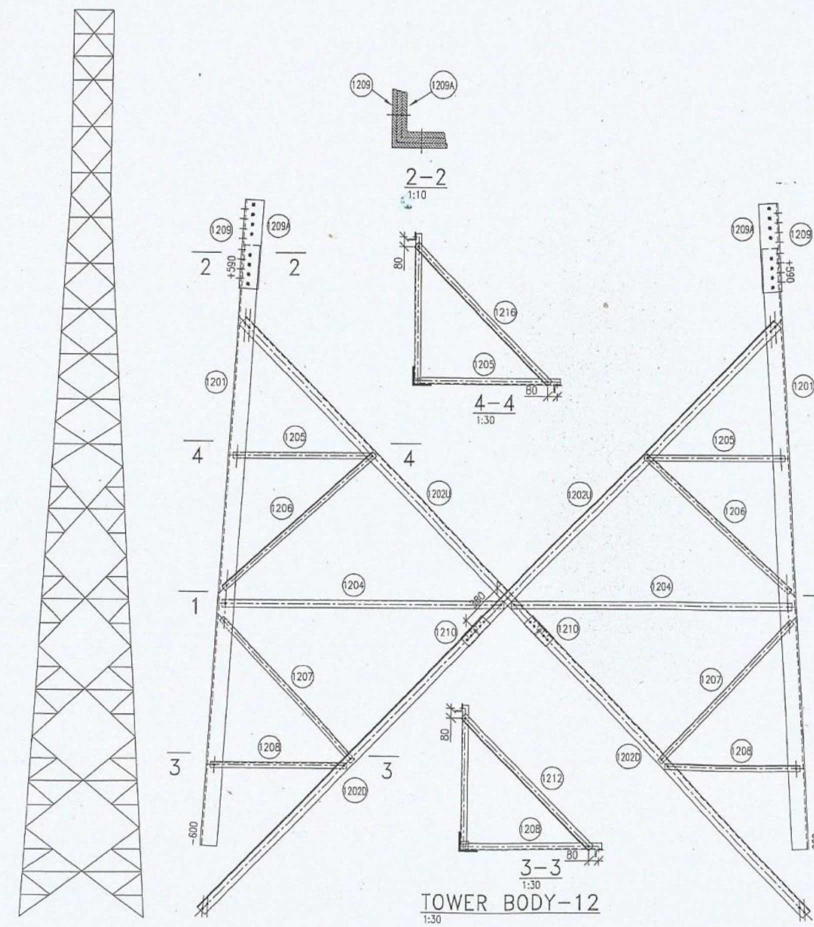
SCALES :
1 : 250, 1:50, 1:30 & 1:60

PROJECT NO : 252
xxx

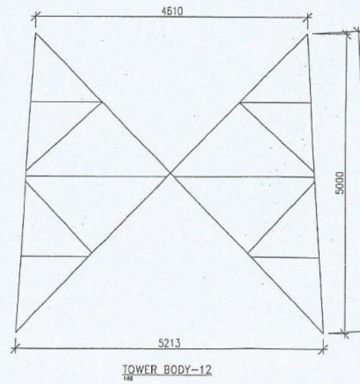
DRAWING NO :
11

REVISION NO :
00

SHEET SIZE :
A3



NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	WGT. (kg) SUB.	NOTE
1201	L140x12	5009	4	127.8	511.2	Q345
1202U / 1202D	L90x6	3614 / 3263	8 / 8	30.1 / 27.2	240.8 / 217.6	
1203	–	–	–	–	–	
1204	L63x5	2380	8	11.5	92.0	
1205	L50x5	1200	8	4.5	36.0	
1206	L50x5	1690	8	6.4	51.2	
1207	L50x5	1620	8	6.1	48.8	
1208	L50x5	1147	8	4.3	34.4	
1209	L160x12	710	4	20.9	83.6	Q345 outer fold
1210	L100x6	260	8	2.4	19.2	outer fold
1211	L63x5	3300	4	15.9	63.6	
1212	L56x5	1548	4	7.1	28.4	
1213	L56x5	1180	4	5.2	20.8	
1214	L56x5	1218	4	5.0	20.0	
1215	L56x5	1676	4	7.1	28.4	
1216	L56x5	1622	4	7.1	28.4	
1217	–	–	–	–	–	
1218	L63x5	3475	2	16.7	33.4	
1219	L63x5	4735	1	22.8	22.8	
1220	–	–	–	–	–	
1221	–	–	–	–	–	
1201A	L125x12	710	4	16.1	64.4	Q345 inner fold
◐	M16x55	–	192		31.1	
◎	M24x80	–	64		44.2	
TOTAL					1270.3	



REVISION	DATE	DESCRIPTION

DRAWING TITLE :
50m TOWER
TOWER BODY - 12

DESIGNED BY : DH
xx

CHECKED & APPROVED BY : AE
xx

DRAWN BY : RD
xx

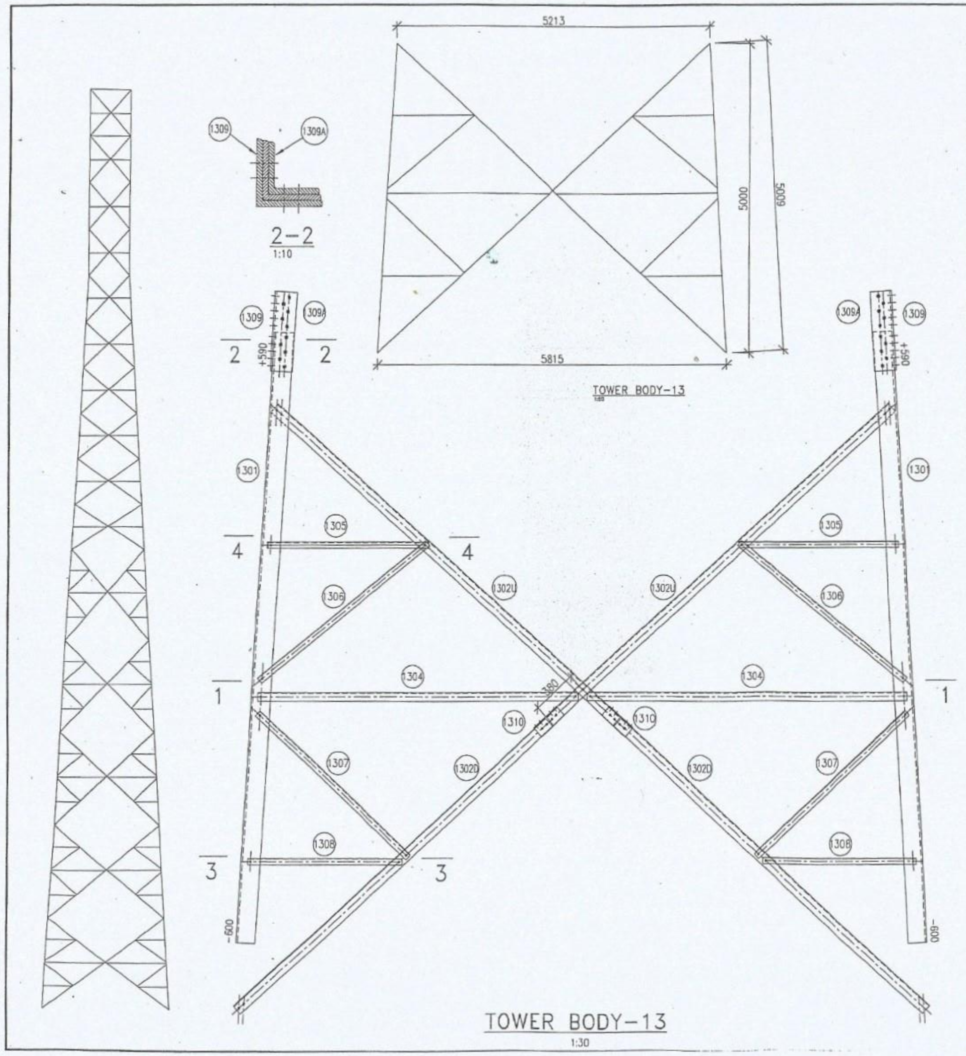
DATE:
20.09.2011

SCALES :
1 : 250,1:50,1:30 & 1:60

PROJECT NO : 252
xxx

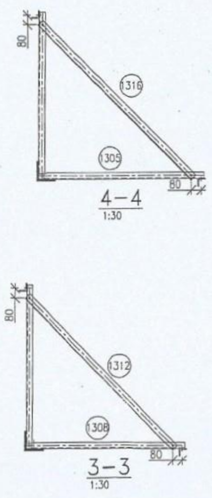
DRAWING NO :
13

REVISION NO : 00
SHEET SIZE : A3



MEMBERS LIST

NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	WGT. (kg) SUB.	NOTE
1301	L160x14	5009	4	170.3	681.2	Q345
1302U / 1302D	L90x7	3846 / 3475	8 / 8	37.2 / 33.6	297.6 / 268.8	
1303	–	–	–	–	–	
1304	L70x5	2662	8	14.4	115.2	
1305	L50x5	1330	8	5.0	40.0	
1306	L56x5	1788	8	7.6	60.8	
1307	L56x5	1705	8	7.2	57.6	
1308	L50x5	1275	8	4.8	38.4	
1309	L180x14	690	4	26.5	106.0	Q345 outer fold
1310	L100x7	282	8	3.1	24.8	outer fold
1311	L70x5	3730	4	20.1	80.4	
1312	L56x5	1727	4	8.0	32.0	
1313	L56x5	1372	4	5.8	23.2	
1314	L56x5	1330	4	5.7	22.8	
1315	L56x5	1887	4	8.0	32.0	
1316	L56x5	1804	4	8.0	32.0	
1317	–	–	–	–	–	
1318	L63x5	4080	2	19.7	39.4	
1319	L63x5	5340	1	25.7	25.7	
1320	L63x5	3543	1	17.1	17.1	
1321	L56x5	1800	1	8.7	8.7	
1309A	L125x12	690	4	20.3	81.2	Q345 inner fold
◐	M16x55	–	194		31.4	
◎	M24x80	–	80		41.4	
TOTAL					2157.7	



REVISION	DATE	DESCRIPTION

DRAWING TITLE :
50m TOWER
TOWER BODY-13

DESIGNED BY : DH
XX

CHECKED & APPROVED BY : AE
XX

DRAWN BY : RD
XX

DATE:
20.09.2011

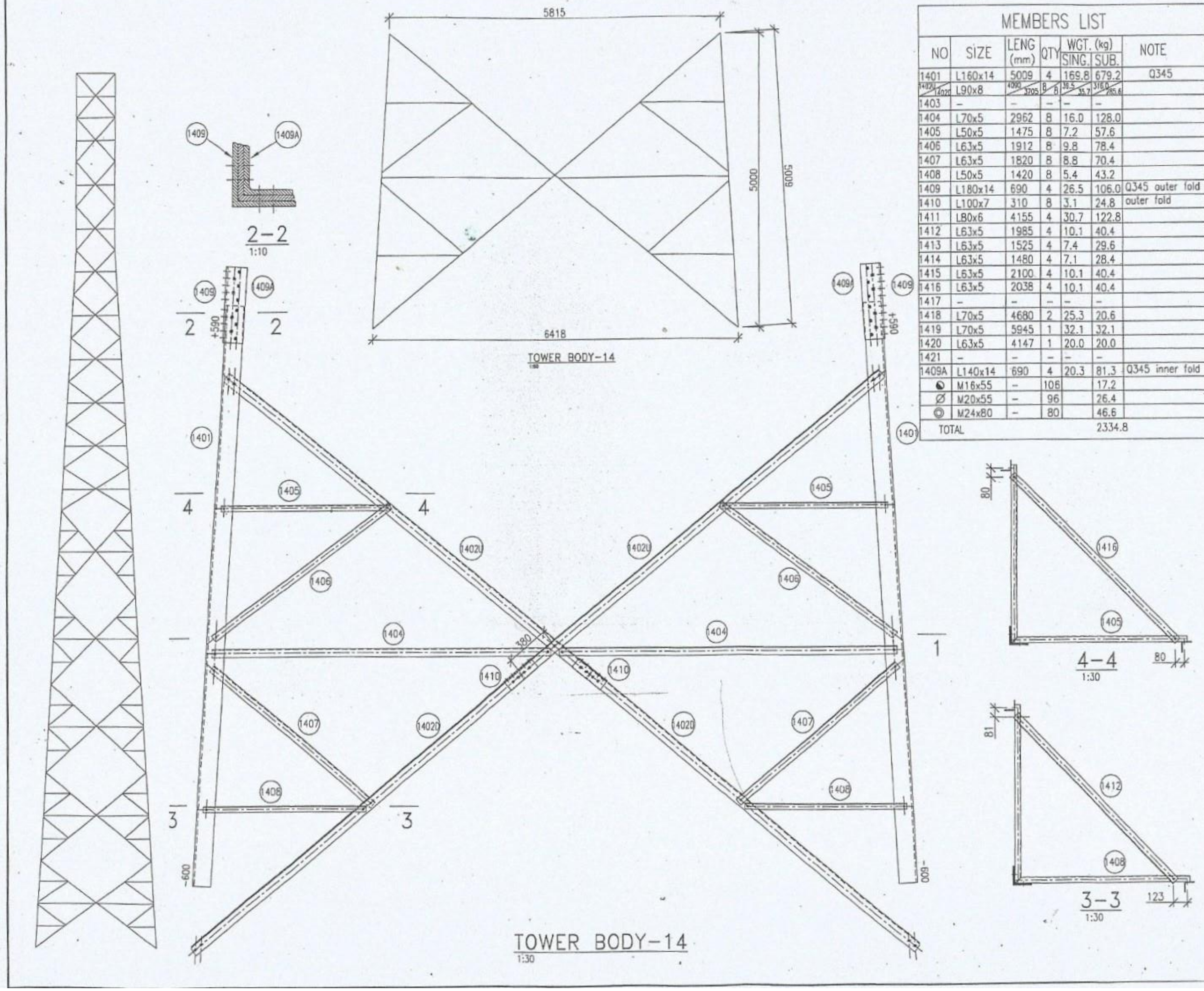
SCALES :
1 : 250, 1:50, 1:30 & 1:60

PROJECT NO : 258
XXX

DRAWING NO :
15

REVISION NO : 00

SHEET SIZE : A3



MEMBERS LIST						
NO	SIZE	LENG (mm)	QTY	WGT. (kg) SING.	SUB.	NOTE
1401	L160x14	5009	4	169.8	679.2	Q345
1402U / 1402D	L90x8	4090 / 3705	8 / 8	36.5 / 35.7	316.0 / 285.6	
1403	–	–	–	–	–	
1404	L70x5	2962	8	16.0	128.0	
1405	L50x5	1475	8	7.2	57.6	
1406	L63x5	1912	8	9.8	78.4	
1407	L63x5	1820	8	8.8	70.4	
1408	L50x5	1420	8	5.4	43.2	
1409	L180x14	690	4	26.5	106.0	Q345 outer fold
1410	L100x7	310	8	3.1	24.8	outer fold
1411	L80x6	4155	4	30.7	122.8	
1412	L63x5	1985	4	10.1	40.4	
1413	L63x5	1525	4	7.4	29.6	
1414	L63x5	1480	4	7.1	28.4	
1415	L63x5	2100	4	10.1	40.4	
1416	L63x5	2038	4	10.1	40.4	
1417	–	–	–	–	–	
1418	L70x5	4680	2	25.3	20.6	
1419	L70x5	5945	1	32.1	32.1	
1420	L63x5	4147	1	20.0	20.0	
1421	–	–	–	–	–	
1409A	L140x14	690	4	20.3	81.3	Q345 inner fold
◐	M16x55	–	106		17.2	
Ø	M20x55	–	96		26.4	
◎	M24x80	–	80		46.6	
TOTAL					2334.8	

REVISION	DATE	DESCRIPTION

DRAWING TITLE :
50m TOWER
TOWER BODY-14

DESIGNED BY : DH
XX

CHECKED & APPROVED BY : AE
XX

DRAWN BY : RD
XX

DATE :
20.09.2011

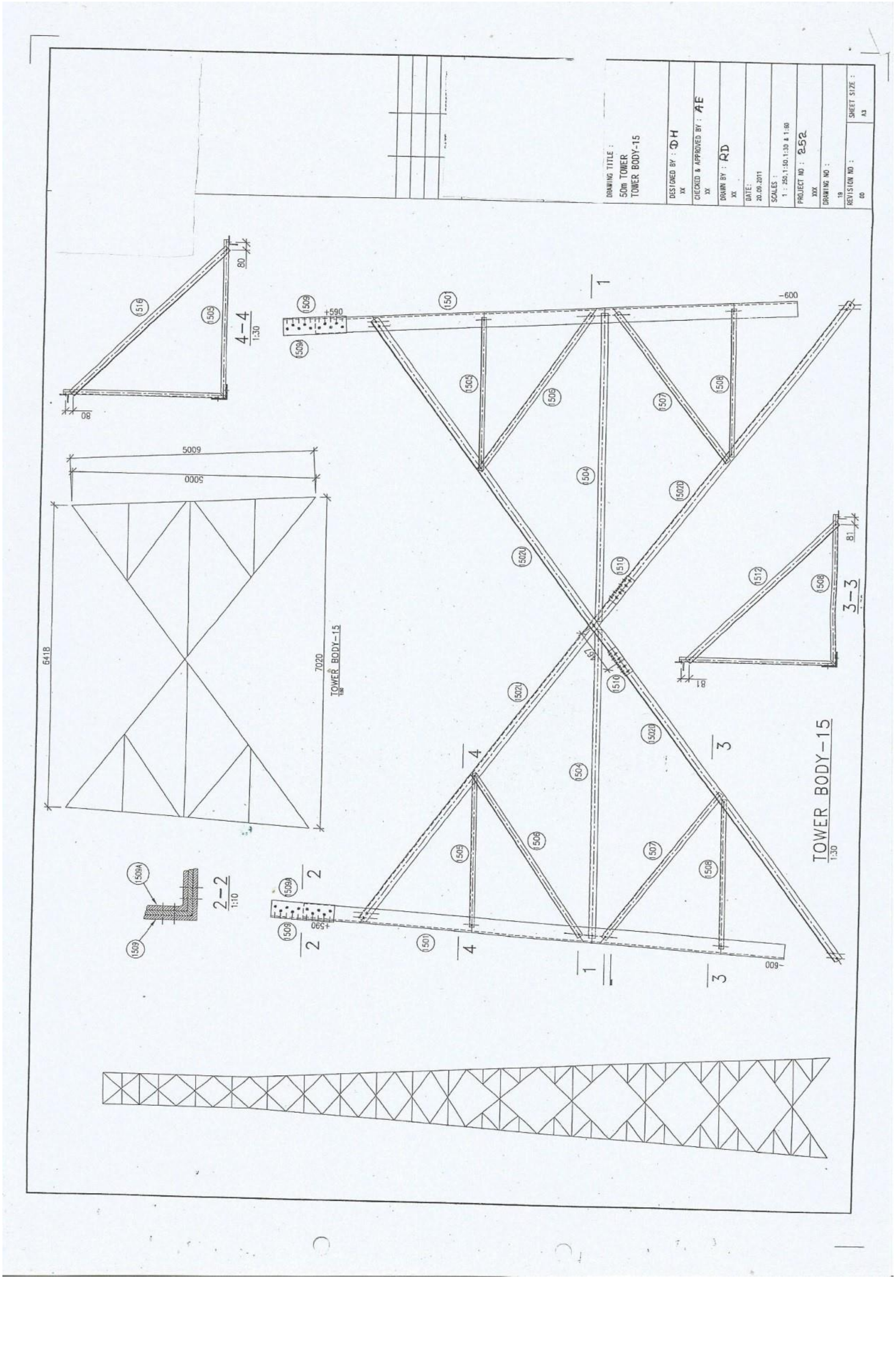
SCALES :
1 : 250,1:50,1:30 & 1:60

PROJECT NO : 252
XXX

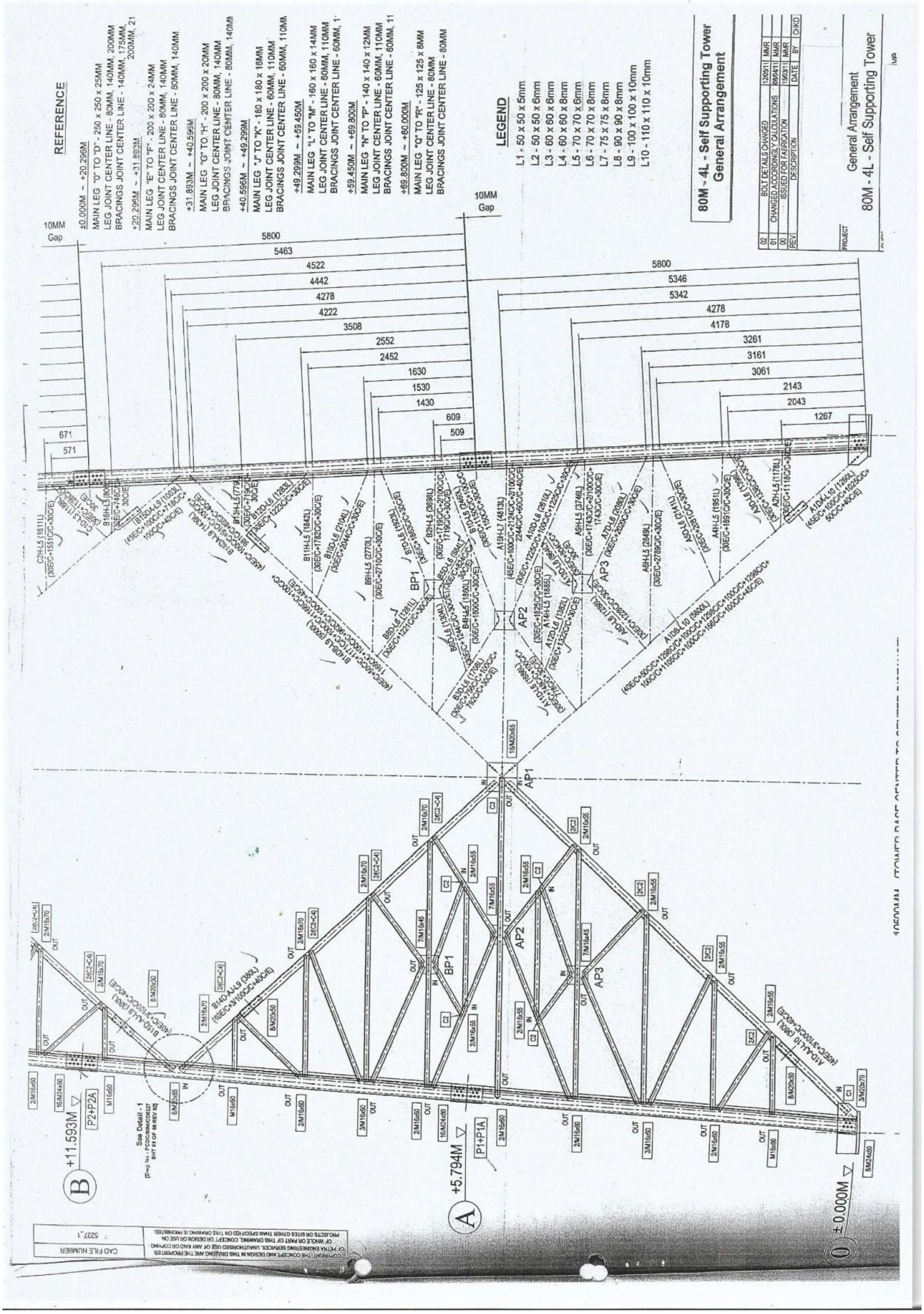
DRAWING NO :
17

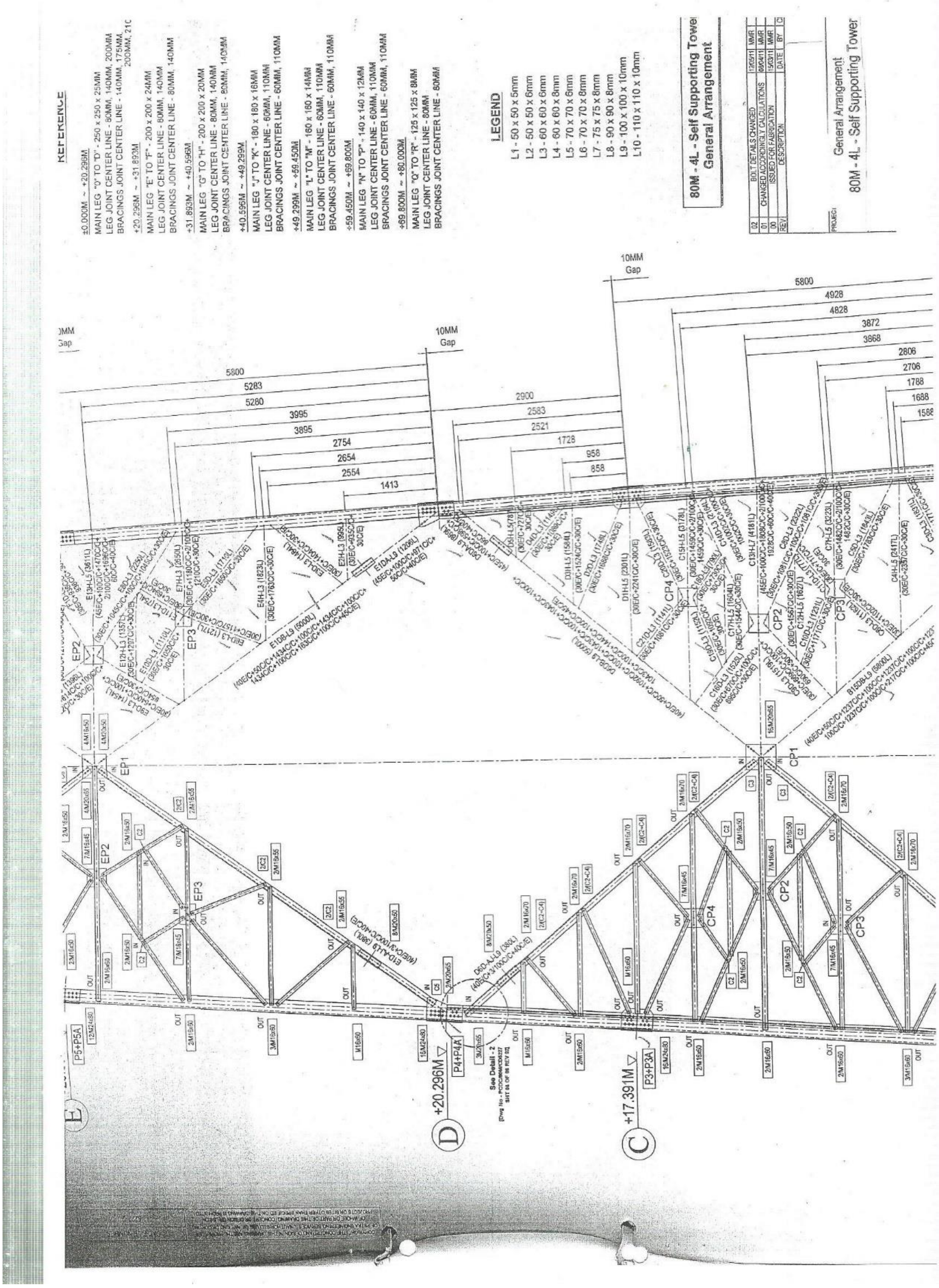
REVISION NO : 00

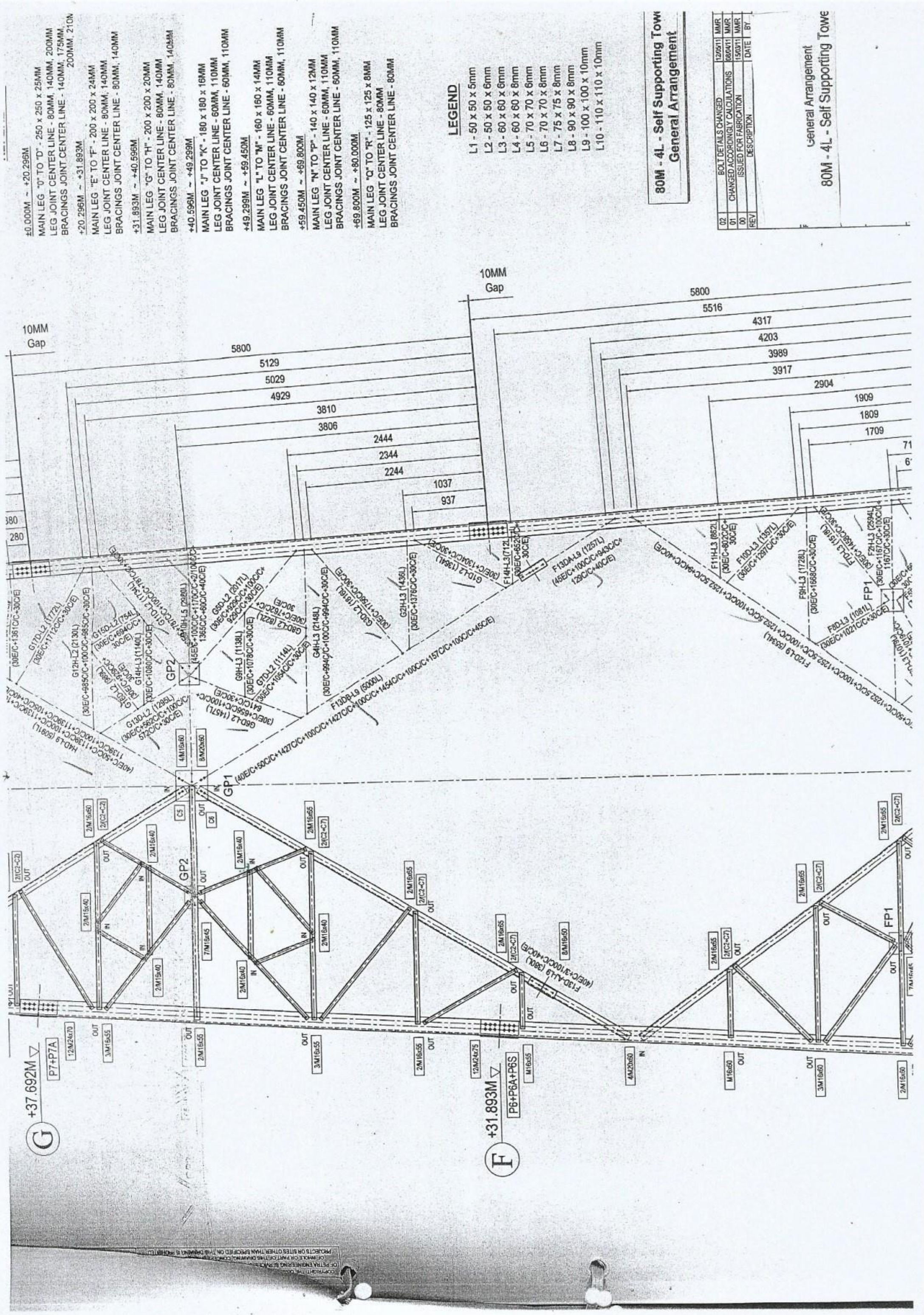
SHEET SIZE : A3

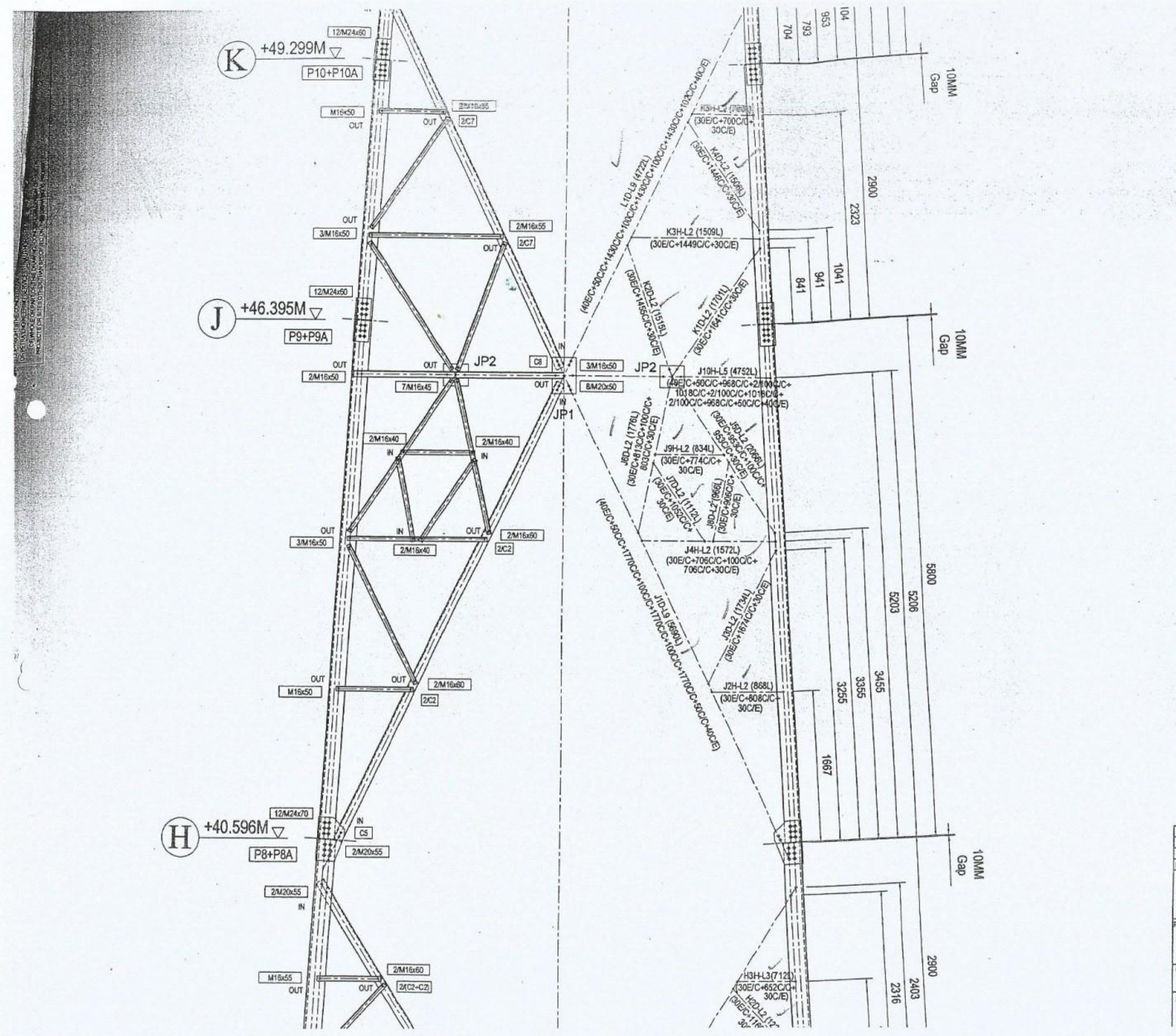

DRAWING TITLE :
50m TOWER
TOWER BODY-15
DESIGNED BY : DH
CHECKED & APPROVED BY : AE
DRAWN BY : RD
DATE:
20.09.2011
PROJECT NO : 252
DRAWING NO :
REVISION NO :
SHEET SIZE :
A3
TOWER BODY-15
1:30
4-4
1:30
2-2
1:10
3-3

3. 80m Four leg tower drawings




REFERENCE
±0.000M ~ +20.296M
MAIN LEG "0" TO "D" - 250 x 250 x 25MM
+20.296M ~ +31.893M
MAIN LEG "E" TO "F" - 200 x 200 x 24MM
+31.893M ~ +40.596M
MAIN LEG "G" TO "H" - 200 x 200 x 20MM
+40.596M ~ +49.299M
MAIN LEG "J" TO "K" - 180 x 180 x 16MM
+49.299M ~ +59.450M
MAIN LEG "L" TO "M" - 160 x 160 x 14MM
+59.450M ~ +69.800M
MAIN LEG "N" TO "P" - 140 x 140 x 12MM
+69.800M ~ +80.000M
MAIN LEG "Q" TO "R" - 125 x 125 x 8MM
LEGEND
L1 - 50 x 50 x 5mm
L2 - 50 x 50 x 6mm
L3 - 60 x 60 x 6mm
L4 - 60 x 60 x 8mm
L5 - 70 x 70 x 6mm
L6 - 70 x 70 x 8mm
L7 - 75 x 75 x 8mm
L8 - 90 x 90 x 8mm
L9 - 100 x 100 x 10mm
L10 - 110 x 110 x 10mm
80M - 4L - Self Supporting Tower
General Arrangement
+20.296M
+17.391M
P4+P4A
P3+P3A
P5+P5A
EP1
EP2
EP3
CP1
CP2
CP3
CP4
10MM Gap


LEGEND
L1 - 50 x 50 x 5mm
L2 - 50 x 50 x 6mm
L3 - 60 x 60 x 6mm
L4 - 60 x 60 x 8mm
L5 - 70 x 70 x 6mm
L6 - 70 x 70 x 8mm
L7 - 75 x 75 x 8mm
L8 - 90 x 90 x 8mm
L9 - 100 x 100 x 10mm
L10 - 110 x 110 x 10mm
80M - 4L - Self Supporting Tow
General Arrangement
+37.692M
+31.893M



REFERENCE

±0.000M ~ +20.296M
MAIN LEG "0" TO "D" - 250 x 250 x 25MM
LEG JOINT CENTER LINE - 80MM, 140MM, 200MM
BRACINGS JOINT CENTER LINE - 140MM, 175MM, 200MM, 210MM

+20.296M ~ +31.893M
MAIN LEG "E" TO "F" - 200 x 200 x 24MM
LEG JOINT CENTER LINE - 80MM, 140MM
BRACINGS JOINT CENTER LINE - 80MM, 140MM

+31.893M ~ +40.596M
MAIN LEG "G" TO "H" - 200 x 200 x 20MM
LEG JOINT CENTER LINE - 80MM, 140MM
BRACINGS JOINT CENTER LINE - 80MM, 140MM

+40.596M ~ +49.299M
MAIN LEG "J" TO "K" - 180 x 180 x 16MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110MM

+49.299M ~ +59.450M
MAIN LEG "L" TO "M" - 160 x 160 x 14MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110MM

+59.450M ~ +69.800M
MAIN LEG "N" TO "P" - 140 x 140 x 12MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110MM

+69.800M ~ +80.000M
MAIN LEG "Q" TO "R" - 125 x 125 x 8MM
LEG JOINT CENTER LINE - 80MM
BRACINGS JOINT CENTER LINE - 80MM

LEGEND

L1 - 50 x 50 x 5mm
L2 - 50 x 50 x 6mm
L3 - 60 x 60 x 6mm
L4 - 60 x 60 x 8mm
L5 - 70 x 70 x 6mm
L6 - 70 x 70 x 8mm
L7 - 75 x 75 x 8mm
L8 - 90 x 90 x 8mm
L9 - 100 x 100 x 10mm
L10 - 110 x 110 x 10mm

80M - 4L - Self Supporting Tower
General Arrangement

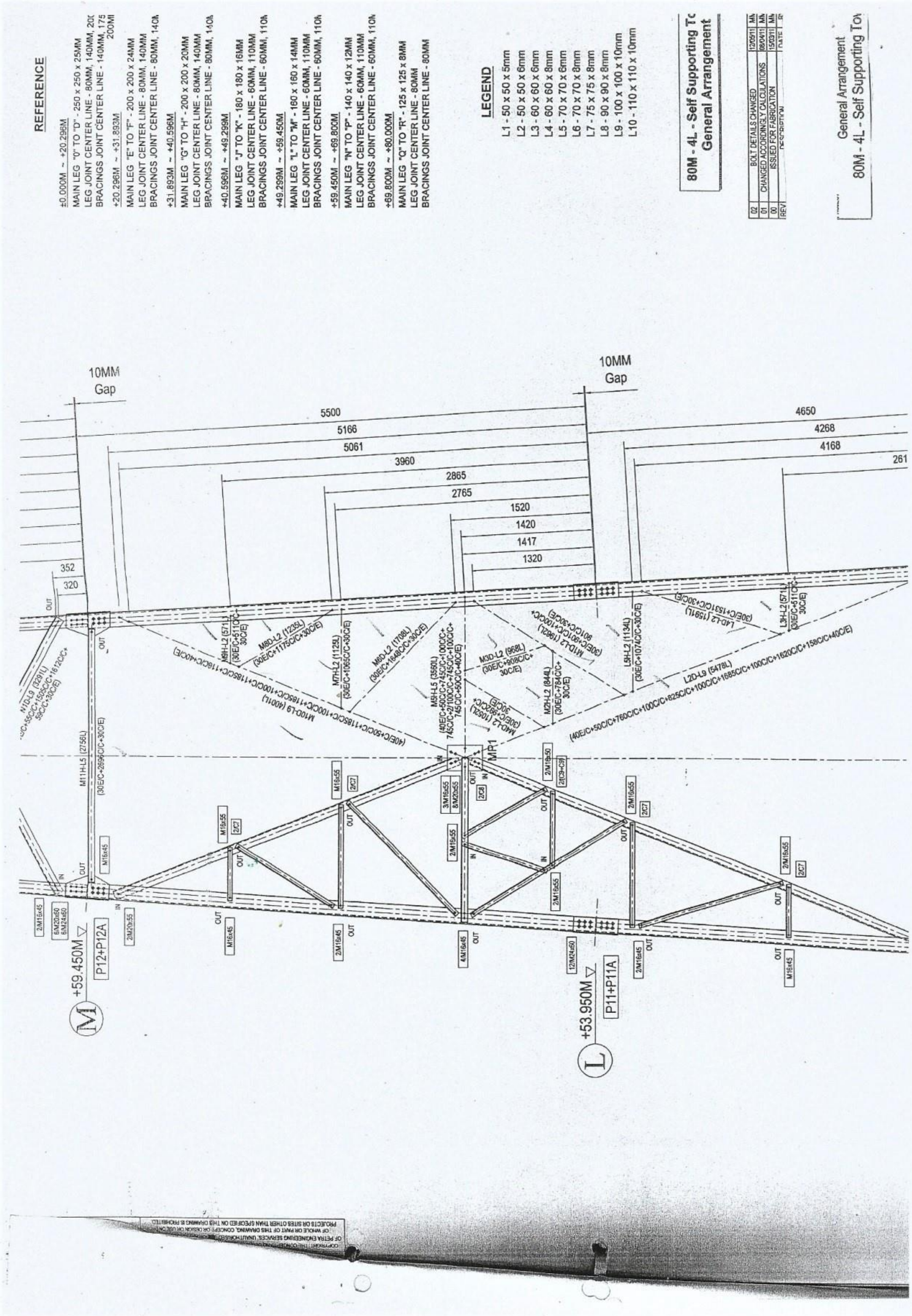
REV	DESCRIPTION	DATE	BY	CHKD	AP
02	BOLT DETAILS CHANGED	12/05/11	MMR		
01	CHANGED ACCORDINGLY CALCULATIONS	08/04/11	MMR		
00	ISSUED FOR FABRICATION	15/03/11	MMR		

PROJECT: General Arrangement
80M - 4L - Self Supporting Tower

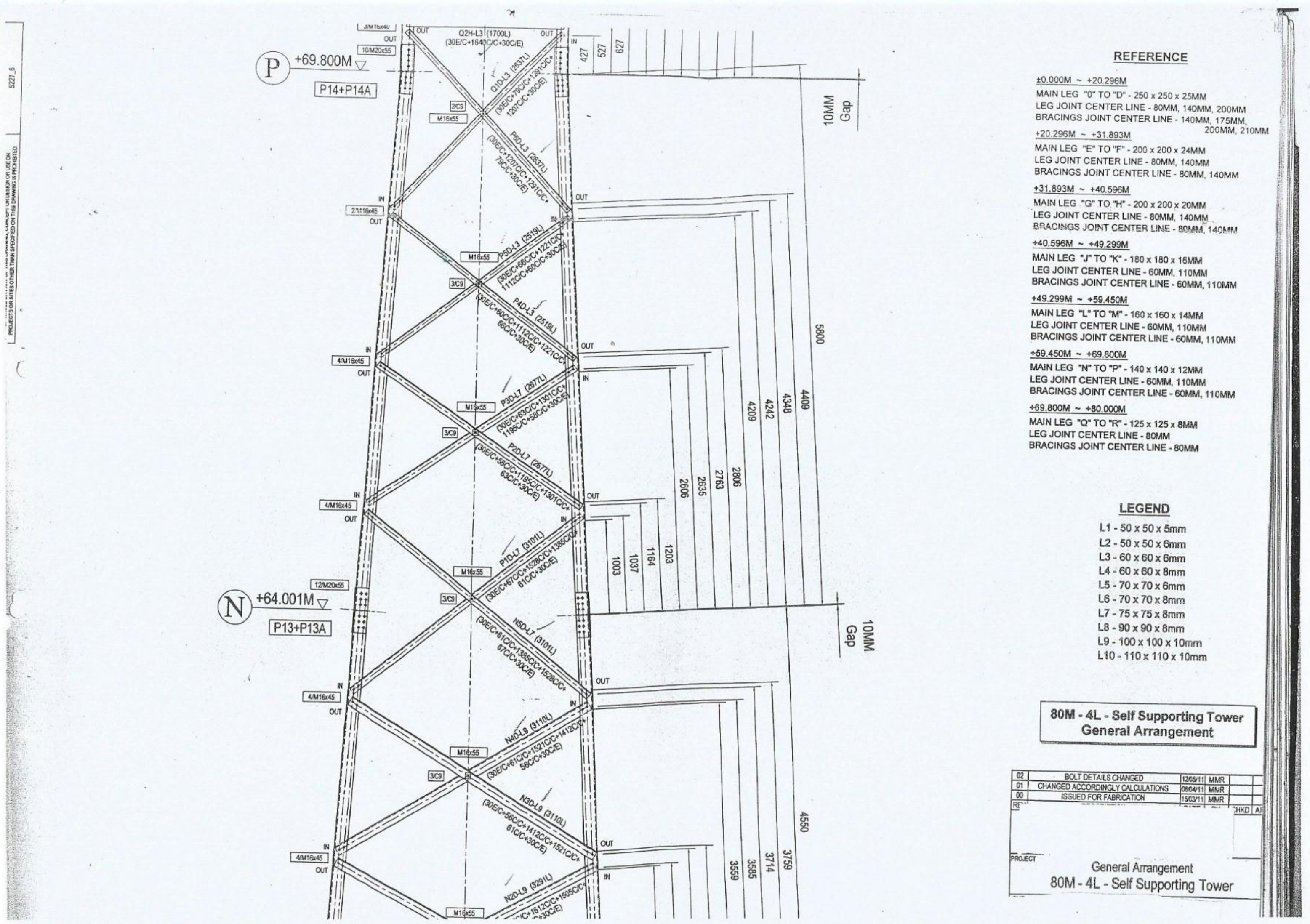
CLIENT: Sri Lanka Telecom - Mullaitive

DRN: Mayuri Ranasinghe
Petra Engineering Services (Pvt) Ltd.

DESIGNED: Ravindra Me[illegible]



REFERENCE
±0.000M ~ +20.296M
MAIN LEG "0" TO "D" - 250 x 250 x 25MM
LEG JOINT CENTER LINE - 80MM, 140MM, 20
BRACINGS JOINT CENTER LINE - 140MM, 175 200MM
+20.296M ~ +31.893M
MAIN LEG "E" TO "F" - 200 x 200 x 24MM
LEG JOINT CENTER LINE - 80MM, 140MM
BRACINGS JOINT CENTER LINE - 80MM, 140
+31.893M ~ +40.596M
MAIN LEG "G" TO "H" - 200 x 200 x 20MM
LEG JOINT CENTER LINE - 80MM, 140MM
BRACINGS JOINT CENTER LINE - 80MM, 140
+40.596M ~ +49.299M
MAIN LEG "J" TO "K" - 180 x 180 x 16MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110
+49.299M ~ +59.450M
MAIN LEG "L" TO "M" - 160 x 160 x 14MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110
+59.450M ~ +69.800M
MAIN LEG "N" TO "P" - 140 x 140 x 12MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110
+69.800M ~ +80.000M
MAIN LEG "Q" TO "R" - 125 x 125 x 8MM
LEG JOINT CENTER LINE - 80MM
BRACINGS JOINT CENTER LINE - 80MM
LEGEND
L1 - 50 x 50 x 5mm
L2 - 50 x 50 x 6mm
L3 - 60 x 60 x 6mm
L4 - 60 x 60 x 8mm
L5 - 70 x 70 x 6mm
L6 - 70 x 70 x 8mm
L7 - 75 x 75 x 8mm
L8 - 90 x 90 x 8mm
L9 - 100 x 100 x 10mm
L10 - 110 x 110 x 10mm
80M - 4L - Self Supporting To
General Arrangement
10MM Gap
+59.450M
P12+P12A
+53.950M
P11+P11A
General Arrangement
80M - 4L - Self Supporting Tov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BRACINGS JOINT CENTER LINE - 140MM, 175MM, 200MM, 210MM
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LEG JOINT CENTER LINE - 80MM, 140MM
BRACINGS JOINT CENTER LINE - 80MM, 140MM
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MAIN LEG "G" TO "H" - 200 x 200 x 20MM
LEG JOINT CENTER LINE - 80MM, 140MM
BRACINGS JOINT CENTER LINE - 80MM, 140MM
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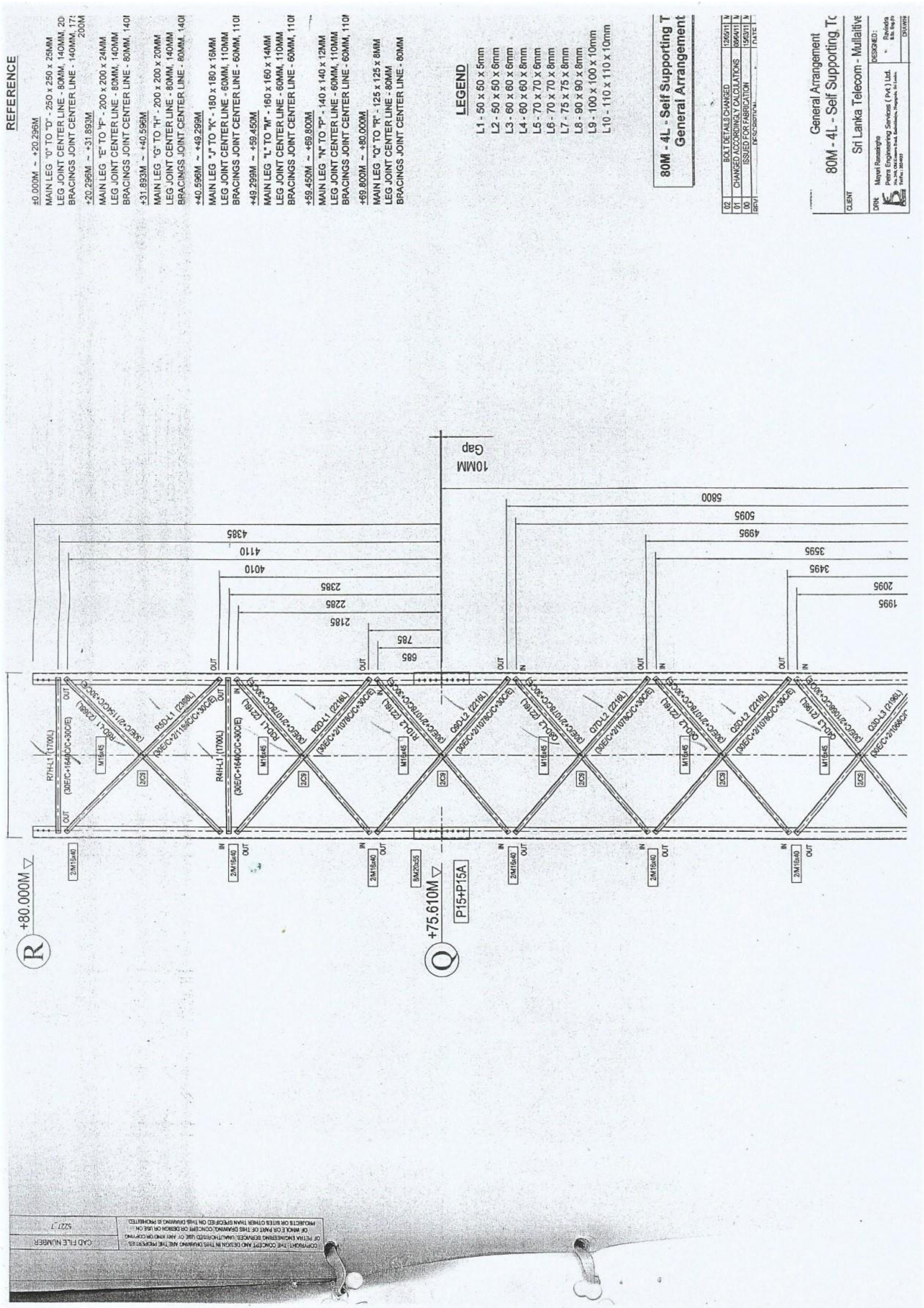
MAIN LEG "J" TO "K" - 180 x 180 x 16MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110MM
+49.299M ~ +59.450M
MAIN LEG "L" TO "M" - 160 x 160 x 14MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110MM
+59.450M ~ +69.800M
MAIN LEG "N" TO "P" - 140 x 140 x 12MM
LEG JOINT CENTER LINE - 60MM, 110MM
BRACINGS JOINT CENTER LINE - 60MM, 110MM
+69.800M ~ +80.000M
MAIN LEG "Q" TO "R" - 125 x 125 x 8MM
LEG JOINT CENTER LINE - 80MM
BRACINGS JOINT CENTER LINE - 80MM
LEGEND
L1 - 50 x 50 x 5mm
L2 - 50 x 50 x 6mm
L3 - 60 x 60 x 6mm
L4 - 60 x 60 x 8mm
L5 - 70 x 70 x 6mm
L6 - 70 x 70 x 8mm
L7 - 75 x 75 x 8mm
L8 - 90 x 90 x 8mm
L9 - 100 x 100 x 10mm
L10 - 110 x 110 x 10mm
80M - 4L - Self Supporting Tower
General Arrangement
02 BOLT DETAILS CHANGED 12/05/11 MMR
01 CHANGED ACCORDINGLY CALCULATIONS 08/04/11 MMR
00 ISSUED FOR FABRICATION 15/03/11 MMR
PROJECT
General Arrangement
80M - 4L - Self Supporting Tower
P +69.800M
P14+P14A
N +64.001M
P13+P13A
10MM Gap
5800
4409
4348
4242
4209
2806
2763
2635
2606
1203
1164
1037
1003
4550
3759
3714
3585
3559
427
527
627
Q2H-L3 (1700L)
(30E/C+1640C/C+30C/E)
Q1D-L3 (2637L)
P6D-L3 (2637L)
P5D-L3 (2519L)
P4D-L3 (2519L)
P3D-L7 (2677L)
P2D-L7 (2677L)
P1D-L7 (3101L)
N5D-L7 (3101L)
N4D-L9 (3110L)
N3D-L9 (3110L)
N2D-L9 (3291L)
M16x55
3/C9
4/M16x45
12/M20x55
10/M20x55
IN
OUT


REFERENCE
+0.000M ~ +20.296M
MAIN LEG "0" TO "D" - 250 x 250 x 25MM
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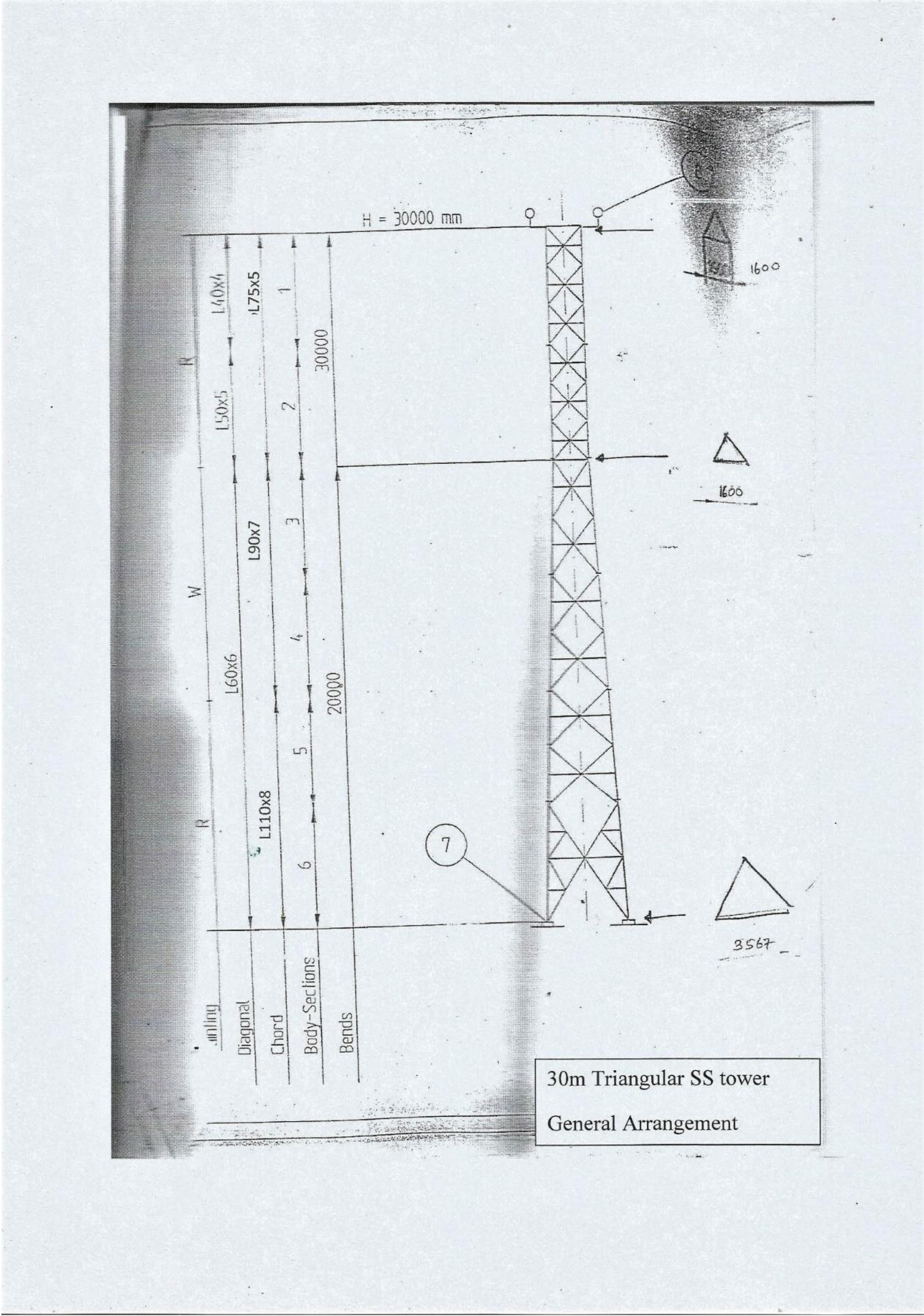
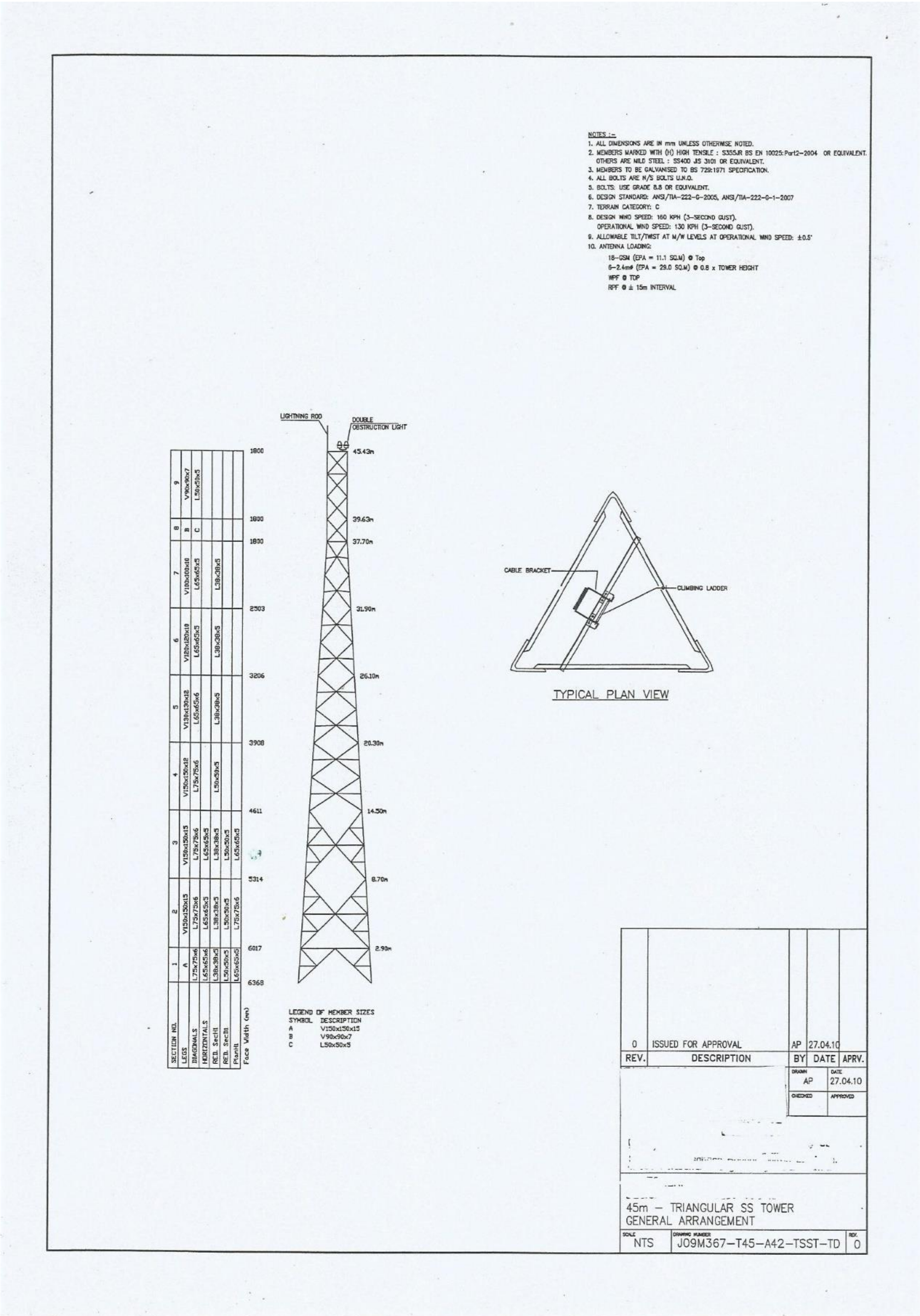
MAIN LEG "E" TO "F" - 200 x 200 x 24MM
+31.893M ~ +40.596M
MAIN LEG "G" TO "H" - 200 x 200 x 20MM
+40.596M ~ +49.299M
MAIN LEG "J" TO "K" - 180 x 180 x 16MM
+49.299M ~ +59.450M
MAIN LEG "L" TO "M" - 160 x 160 x 14MM
+59.450M ~ +69.800M
MAIN LEG "N" TO "P" - 140 x 140 x 12MM
+69.800M ~ +80.000M
MAIN LEG "Q" TO "R" - 125 x 125 x 8MM
LEGEND
L1 - 50 x 50 x 5mm
L2 - 50 x 50 x 6mm
L3 - 60 x 60 x 6mm
L4 - 60 x 60 x 8mm
L5 - 70 x 70 x 6mm
L6 - 70 x 70 x 8mm
L7 - 75 x 75 x 8mm
L8 - 90 x 90 x 8mm
L9 - 100 x 100 x 10mm
L10 - 110 x 110 x 10mm
80M - 4L - Self Supporting T
General Arrangement
R
+80.000M
Q
+75.610M
P15+P15A
10MM Gap
General Arrangement
80M - 4L - Self Supporting To
Sri Lanka Telecom - Mullaitive

4. 30m Three leg tower drawing



5. 45m Three leg tower drawing



6. 60m Three leg tower drawing