MODULUS

Society of Structural Engineers - Sri Lanka

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Society News & Technical Papers

DECEMBER 2023

EDITOR'S MESSAGE

"Safety of Rural bridges"

As the newly elected Editor of SSESL, I would like to welcome all of you for the December 2023 Edition of Modulus. The newly elected executive committee of SSESL under the leadership of Eng. Ananda Senarath, plans to conduct various events and activities for the benefit of the membership of the society in the coming year. We expect your fullest cooperation in this regard.

Modulus is always a platform for disseminating knowledge among members of the society on various structural engineering aspects. This edition of Modulus comprises of two technical papers, society news and the event calendar of the year 2024 of SSESL. I am very grateful to the authors who have sent technical papers for this edition.

Now, I will focus on the topic that was selected for the editorial message "Safety of Rural bridges".

Failure of at least four rural road and pedestrian bridges have been reported in various parts of the country during the last two months in Sri Lanka. Failure of a bridge in Meegahapitiya- Kottagala road in Maddula area of Monaragala district, failure of a bridge in a by road of Mahiyanganaya to Rohana Junction, a collapse of a bridge in a by road close to Dambulla town and a collapse of a Pedestrian bridge across Madakandura Oya in Haputale are those reported failures. Some of these failures causes injuries to relevant vehicle users and passengers during collapses. Even though none of these bridges are large scale structures, economic losses and social impacts are significant as some of these bridges had been the only access path to certain areas. Below figure shows the images taken after the failure of a bridge in Meegahapitiya- Kottagala road in Maddula area of Monaragala district.

Images taken after the failure of a bridge in Meegahapitiya- Kottagala road in Maddula area

The preliminary investigation carried out in this case revealed that combine effect of deficiencies of the design, aging of structural elements, issues in workmanships and maintenance related issues have caused the ultimate failure of the bridge while a truck having a defined load capacity of 25MT was moving on this bridge. Some of these concerns that would have contributed for the failure is visible in photographs. In case of failure of pedestrian bridge across Madakandura Oya in Haputale, media reports highlighted that the substandard construction as the reason for the collapse. This failure had been taken place while two women were walking on this bridge, and they were injured due to this failure.

These incidents should be considered as eye openers, and it is necessary to take immediate steps to effectively manage this type of rural bridges to avoid catastrophic failures in the future. Since most of these bridges are in rural road networks or in access roads or in foot paths, the responsible authorities to maintain these structures are not well defined in certain cases. Further, structural engineering knowledge and technical know-how of officers, who have the responsibility of maintain these structures may be insufficient to maintain and manage these types of structures. The lack of budgetary allocation for maintenance and rehabilitation with present economic crisis is another factor contributed for this issue.

Hence, as structural engineers, we shall also play role to aware relevant authorities regarding this serious issue and force them to take necessary actions in this regard. Further, we shall provide our expertise knowledge whenever possible to ensure safety of these rural bridges!!!

Eng. (Dr.) A.M.L.N. Gunathilaka
BSc Eng. (Hons),MEng(struct.), PhD, CEng, MIE(SL), MSSE(SL), GreenAP

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ACKNOWLEDGEMENT

The Editor wishes to thank authors of the research articles, Eng. R. M. A. Senarath for providing the write-up for the cover story.

CONTENT

COVER STORY

Introducing the impressive Head Office Building for Vallibel Finance PLC in Colombo, located at No: 310 Galle Road, Colombo 03. This building is a symbol of prestige, commissioned by Vallibel Finance PLC. It combines elegance and practicality, designed to meet the needs of Vallibel Finance and their clients. With modern features and carefully designed, it aims to create a dynamic workspace that encourages success and collaboration. This is the hub of modern corporate culture, reflecting Vallibel Finance PLC's vision and ambition.

The building stands tall at 15 levels and a semi-basement level. As you enter, there's a dedicated area for banking operations, blending functionality with style. The next three floors provide for parking, ensuring convenience for users. There are spacious office areas, promoting productivity and teamwork. Access to parking is made easy with a cutting-edge car lift system. The building strength comes from a robust reinforced concrete structure supported by board cast in-situ pile foundations. Every detail is carefully crafted from reinforced concrete, ensuring both beauty and durability, showcasing a commitment to excellence in design and construction.

Employer – Vallibel Finance PLC

Architect – Suchith Mohitti Associates Private Limited

Structural Engineer – Stems Consultants Private Limited

Main Contractor – Sanken Construction Privet Limited

Piling Contractor – Access Engineering Private Limited

Project Cost – LKR 1.2 billion

Commencement – November 2019

Completion – June 2023

Annual General Meeting – 2023

30th Annual General Meeting of the Society of Structural Engineers Sri Lanka was held on the 5th December 2023 at the Cinnamon Grand Hotel Colombo. The AGM was sponsored by Lanwa – Sanstha Cement Corporation (Pvt) Ltd. and Ceylon Steel Corporation.

Photograph 01: Eng. D.T. Rajasekaran; Hon. Secretary of the outgoing committee presenting the Annual Report.

At the 30th Annual General Meeting, a special speech on Thirty Years of Excellence SSESL was delivered by one of our eminent Past Presidents, Eng. (Prof.) Ruwan Weerasekara highlighting the history of the society and its development to the present state. The full speech of Eng. (Prof.) Ruwan Weerasekara is presented in page 9.

Photograph 02: The outgoing committee 2022/2023.

Photograph 03: Participants at the AGM

The Following Executive Committee members were elected for the year 2022/2023 at the Annual General Meeting.

Office Bearers 2023/2024	
President	Eng. R.M.A. Senerath
Vice President	Eng. S.S.A. Kalugaldeniya
Immediate Past President	Eng. (Mrs.) T.J. Jayasundara
Past President	Eng. N. Abeysuriya
Hon. Secretary	Eng. D.T. Rajasekaran
Treasurer	Eng. L. Gunawickrama
Editor	Eng. (Dr.) A. M. L. N. Gunathilaka
Public Relations Officer	Eng. G. Ramawickrama
Assistant Secretary	Eng. R.S.K. Thrimavithana
Assistant Treasurer	Eng. (Mrs.) A. D. S. Gunawardana
Assistant Editor	Eng. R.M.B. Prasad
Committee Members Over 40 Years	Eng. (Prof.) S. M. A. Nanayakkara Eng. A. S. B. Edirisinghe Eng. M. Gamage Eng. (Dr.) P. D. Dharmaratne
Committee Members Below 40 Years	Eng. B. Kiriparan Eng. K. P. M. D. D. N. Feridinando

The AGM was followed by the gust talk delivered by Dr. Indrajith Coomaraswamy, who is a former governor of Central Bank of Sri Lanka on the timely topic “Sri Lanka’s Economy: Where is it headed?”. Dr. Coomaraswamy expressed his thoughts on the present state of the economy and actions to be taken to move it in the right direction.

Photograph 04: Dr. Indrajith Coomaraswamy delivering Guest speech at the AGM

Every year the Society of Structural Engineers Sri Lanka honoured the best authors of papers presented at the Annual Sessions and they are published in the Modulus. This year too Prof. Raghu Chandrakeerthi Gold Medal for the best paper presented in the Annual Sessions and Access Engineering Gold Medal for the best paper published in the Modulus for the year 2023 were awarded at the AGM.

Gold Medal Winners of Years 2023	
Annual Sessions 2023 (Prof. Raghu Chandrakeerthi Gold medal)	Eng. (Ms.) M. L. T. Abeysingha
Modulus 2023 (Access Engineering Gold medal)	Eng. A. Jayasinghe

Photograph 05: Gold medal for the best paper presented at the Annual Sessions 2023 awarded to Eng. Ms. M. L. T. Abeysingha

Photograph 06: Participants at the AGM

Photograph 07: Cocktail and Fellowship Dinner

E - Library

Society of Structural Engineers, Sri Lanka launched the E-Library at the AGM, which is an online platform that enables you to refer to civil and structural engineering books and other publications. The facility will be provided free of charge at the initial stage when registered with the platform for our members.

Publications such as Modulus, Technical papers published at Annual sessions and Seminars, and other books published by the SSESL are presently available on the platform. Members are encouraged to register with the platform and get the maximum benefit from the newly developed E-Library.

Web Link: https://elibrary.ssesl.lk/

Event Calendar – SSESL for the Year 2024

Month	Date	Event
January	30th	Question Time
February		Design Course 1
	27th	Question Time
March	19th	Seminar 1
	26th	Question Time
April	30th	Question Time
May		Design Course 2
	28th	Question Time
June	25th	Question Time
July		Design Course 3
	23rd	Question Time
August	27th	Annual Session
September	24th	Question Time
October	29th	Question Time
November	19th	Seminar 2
	26th	Question time
December	3th	Annual General Meeting

Seminar on Trends, Drives and Challenges in High-Rise Building Design

The final seminar organized by the SSESL for the year 2023 was held on the topic Trends, Drives and Challenges in High-Rise Building Design. The event was successfully conducted on the 13th of November 2023 at the Cinnamon Grand, Colombo. The platinum sponsor for the seminar was Confab-CN Steel Pvt Ltd. The seminar aimed to aware practicing engineers on the innovative ways of addressing practical issues and emerging practices in the world in the field of Structural Engineering designs.

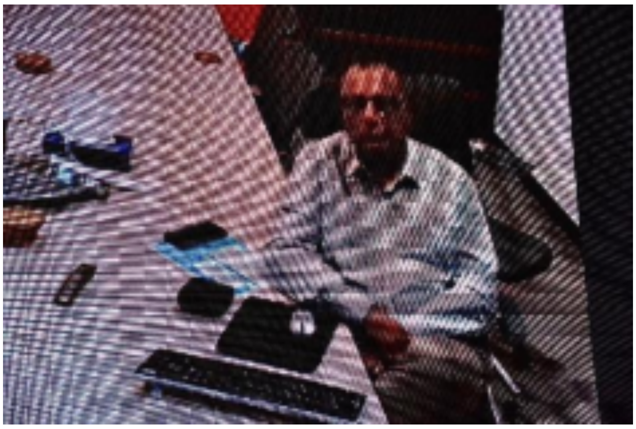
The two sessions in the morning and afternoon were chaired by Eng. K.L.S. Sahabandu and Eng. R.M.A. Senarath. Three presentations followed by a Questions and Answers session was comprised in the morning session.

Eng. Angelo Thurairajah joined online and presented on Resilient Design of high-rise buildings. A presentation on Design and Construction of facades in high-rise buildings was presented by Eng. Shiromal Fernando and Trends in designing and testing the foundations of high-rise buildings in Sri Lanka was present by Prof. Saman Thilakasiri.

In the afternoon, Eng. (Mrs.) Tharangika Jayasundara delivered a presentation on the evolution of structural designs of high-rise buildings: past, present & future. Presentations on challenges in designing MEP services for high-rise buildings and Post-tensioned applications in high-rise buildings were presented by Eng. Kosal Kamburadeniya and Eng. Pham Quy Duong respectively. Eng, Pham Quy Duong delivered his presentation as an online session. There was a Q & A session at the end of the three presentations.

Photograph 08: Eng. R.M.A. Senerath lighting the oil lamp at the Seminar.

Photograph 09: President of SSESL; Eng. (Mrs) Tharangika Jayasundara delivering the welcome address at the Seminar.



Photograph 10: Eng. Angelo Thurairajah delivering the Presentation on resilient design of high-rise buildings

Photograph 11: Eng. Shiromal Fernando delivering the presentation on Design and Construction of facades in high-rise buildings

Photograph 14: Eng. (Mrs.) Tharangika Jayasundara delivering the presentation on evolution of structural designs of high-rise buildings: past, present & future

Photograph 12: Prof. Saman Thilakasiri delivering the presentation on Trends in designing and testing the foundations of high-rise buildings in Sri Lanka

Photograph 15: Eng. Kosal Kamburadeniya delivering presentation on challenges in designing MEP servers for high-rise buildings

Photograph 13: Eng. K. L. S. Sahabandu conducting the panel discussion at the morning session

Photograph 16: Eng. Pham Quy Duong a delivering presentation on the post-tensioned applications in high rises buildings

Photograph 17: Eng. R.M.A. Senerath conducting the Question and Answer session at the afternoon session of the seminar

Webinar

Lessons in Structural Engineering from Mw 7.8 Gorkha Nepal Earthquake

Eng. (Dr.) Samith Buddhika was the resource person for the first webinar conducted for the year 2023 by the SSESL. The topic for the webinar was Lessons in Structural Engineering from Mw 7.8 Gorkha Nepal Earthquake was held on the 31st of October 2023.

During his presentation, he highlighted the damage to cultural heritage, and low-rise and high-rise reinforced concrete structures. The causes of the failure of the masonry construction were discussed in detail providing explanations on preventive measures. Dr. Samith explained the major causes of collapse such as heavy load accumulation, aging, lack of bracing, etc.

The common failures of reinforced concrete structures were also discussed. Soft story effect, pounding, shear failure, and other failures associated with construction were explained while providing evidence of those failures. Structural design deficiencies connected with building symmetry, and reinforcement detailing issues were also highlighted during the presentation. The possibility of incorporating lessons learnt from this event to develop guidelines for further construction practices in Sri Lanka was also discussed.

How to Approach the GIStructE Exam on Structural Behaviour

The second webinar of the year 2023 organized by the Associate Member Chapter was successfully held on 14th November 2023. The event was moderated by Emeritus Prof. Priyan Dias and Eng. Isuru Nanayakkara delivered the presentation.

The following content was discussed during the presentation.

- Introduction to the certificate in the structural behaviour exam
- Understanding structural behaviour
    - Response of trusses/beams/frames/arches/cables
    - Indeterminacy and stiffness
- Material behaviour (mechanical and thermal)
- Vibration analysis

During the presentation, the procedure to approach the examination and the technical knowledge required to get through the examination were discussed.

Question Time

Question Time November 2023: Design and Construction of Deep Basements

Eng. Ananda Senarath shared his vast experience in the design and construction of deep basements in the QT that was successfully conducted on 28th November 2023 at 5.15 pm. The event was conducted physically at the Wimalasurendra Auditorium, IESL. It was sponsored by Tokyo Cement Group.

The real challenges faced by the design and construction team in the construction of the deep basements were explained by Mr Senarath while providing the solution to solve those issues. Important facts to be considered when designing deep basements and some design parameters to be considered in the design were also explained.

In summary, the session was focused on the basic design guidelines and methods of construction of deep basements.

Photograph 18: Eng. R.M.A. Senerath delivering the presentation on the design and construction of deep basements.

Full list of Question Time Sessions Conducted in 2023

SSESL conducted Question Time Sessions in most of the months of year 2023. Information on the events conducted for the year 2023 are as follows:

Month	Topic	Resource Person	Sponsor
January	Challenges in Planning and Design of Aviation and Airport Related Infrastructures	Eng. Gnanasiri Withanage	Confab Steel
February	Is Structural Timber Used Efficiently?	Dr. (Mrs.) Premini Hettiarachchi	Confab Steel
March	Eng. S A Karunaratne Memorial Lecture - Some Thoughts on Education of Structural Engineers	Dr. Nihal Somaratna	None
April	Paint Coatings	Mr. Prabath Jayawardena	Nippon paint
May	Response of Shear Critical Reinforced Concrete Elements	Prof. Kushan Kalmith Wijesundara	Tokyo Cement Group
June	Specification and Industrial Usage of Aluminium Alloys	Prof. Nanda Munasinghe	Alumex PLC
July	Advanced Concrete Technology in Large Volume Concrete Pouring	Eng. Samitha Jayakody	Tokyo Cement Group
September	Non-Destructive Testing Re-visited	Prof. Priyan Dias	Tokyo Cement Group
October	Lessons in Structural Engineering from Mw 7.8 Gorkha Nepal Earthquake "	Dr.(Eng.) H A D Samith Buddhika	
November	Design and Construction of Deep Basements	Eng. R M Ananda Senarath	Tokyo Cement Group

Development of a National Building Code for Sri Lanka - SSESL/ARUP Working Session

Inception of unified national building code for the country was identified as essential need by local construction professionals and with aids of World Bank, this process was initiated in the early part of this year. ARUP was appointed as the international consultant for the conceptualization of Sri Lankan building code and they sought the local consultation from SSESL.

Accordingly, SSESL actively engaged with ARUP in this task as this is a matter of national interest. SSESL and ARUP conducted a joint working session on 13th and 14th December 2023. Working group members of SSESL and ARUP, and Industry stakeholders participated in this session. The working session was successfully concluded marking the completion of the conceptualization phase.

Photograph 19: A discussion at the working session

Photograph 20: A Presentation at the Working session

Photograph 21: A group discussions at the Working session

Thirty Years of Excellence SSESL - Speech by Eng. (Prof.) Ruwan Weerasekera at AGM

Photograph 22: Eng. (Prof.) Ruwan Weerasekara

Good afternoon distinguished guests, esteemed colleagues, and fellow enthusiast,

Today, I am pleased to invite you to join me on a journey of reflection on this great organization SSESL.

The journey of the Society of Structural Engineers Sri Lanka (SSESL) is a testament to the power of resilience and the beauty that emerges from humble beginnings. What started as a modest venture, by a few structural engineers that got together to discuss minor structural engineering issues, grew in strength and stature over the last thirty three years. This origin story goes all the way back to 1990 when a group of senior structural engineers consisting of Eng. (Dr.) A.C. Visvalingam, Eng. Mathan Coomaraswamy, Eng.Daya de Sliva, Eng.(Prof.) Ragu Chandrakeerthy, Eng. (Dr.) Priyan Dias (now emeritus professor) and few others, led by Eng. (Dr.) B.M.A. Balasooriya joined hands with the mutual idea of forming a professional body of structural engineers when the Institution of Engineers Sri Lanka, at the time, refused to create a sectorial committee for structural engineering. In 1991, upon my return from Canada, after completing a PhD in structures, I was invited to join the society as the assistant secretary. Since then, I have held every other position in the society expect Treasurer and Assistant treasurer every year for 28 years excluding the year I was on sabbatical leave.

In the year 1991, we formalized the society and commenced its first main event, a more or less informal/interactive lecture, known as “Question Time”, which is carried out monthly and is quite popular yet to this day.

This was the time we were faced with some unfortunate circumstances when Eng. Daya de Silva moved to the United Kingdom and Eng. (Dr.) B.M.A. Balasooriya fell ill. The society was at the verge of collapsing even before it could have a chance to properly establish itself. A major turning point for the society was at this point when Eng. (Dr.) A.C. Visvalingam was elected, in his own words, “against his inclinations” and was given the responsibility to resurrect the society. His unique methodical style of leadership, out of which I, myself have adapted certain qualities from and admired throughout, was exactly what the doctor ordered at the time.

His tenure, just shy of a decade, was when the society really kicked off its glory days by making many pivotal decisions in proceeding to establish the Society of Structural Engineers Sri Lanka. This was also the time the society was incorporated under the Companies Act of 1982. Upon due incorporation on the 6th of July 1993, we had our first official committee of office bearers. Eng. (Dr.) A.C. Visvalingam (President), Eng. (Prof.) Raghu Chandrakeerthy(Vice President), Eng. Mathan Coomaraswamy (Honorary Secretary), Myself (Assistant secretary), Eng. D.P.T. Munasinghe (Treasurer), Eng. D.C.R. Wickramasighe (Assistant Treasurer), Eng. (Dr.) M.C.M. Fonseka (Editor) and Eng. S.H.J. Weerasuriya (Public relations officer).

As a new organization, the society was faced with a lack of funding and many other financial difficulties. Eng. (Dr.) A.C. Visvalingam and Eng. Mathan Coomaraswamy, both members of the engineering industry, stepped out to coordinate necessary arrangements and were faced with various difficulties. Their establishment as practicing engineers was what brought in the necessary funding as they sacrificed their personal bonuses for society activities. At this juncture Eng. (Dr.) A.C. Visvalingam was able to secure 1 million from Property Developers Lanka Ltd saving 200 million refurbishment of the Bank of Ceylon Building. Also, Builtmart made a contribution of Rs.60,000 and pledged to enhance their fund annually.

The initial secretarial work was conducted at Mr. Mathan Coomaraswamy’s office in Wellawatte. Shortly after, in 1993, we were faced with another complication when Mr. Mathan Coomaraswamy migrated to the United Kingdom. Fortunately, soon after Eng. (Dr.) A.C. Visvalingam had secured his private office space down Cotta Road and moved our secretariat there, fully serviced, furnished, and free of charge. Eng. (Dr.) Visvalingam’s actions brought the society some much needed stability and we were in a position to employee our first administrative officer, loved by all, now late, Mr. Cyril Abeysekera.

At the end of Eng. (Dr.) Visvalingam’s lease, the landlord offered the society a different office space on a different floor of the same building for a nominal rent.

Throughout the years of 1992-1999 the society was able to conduct many successful seminars and lectures. It was also between this period when the society, with great difficulty was able to secure an office space at the Organization of Professional Associations at which we were thrown more hurdles due to not being a member of the OPA. The National Building Research Organization graciously gave us provision to conduct our committee meetings at their premises at no additional costs between the period of the secretariat moving out of Eng. Mathan Coomaraswamy’s office and moving in to the OPA.

Another significant initiative of the early days was the introduction of the now well published “Modulus” introduced in 1990 by Eng.(Prof.) Priyan Dias.

A young me, was entrusted with the position of honorary secretary in 1995 when Eng. Saro Weerasuriya stepped down to form his own practice. I held the position for seven years amidst which Eng. (Dr.) Visvalingam, upon setting a strong foundation, decided to pass on his baton of leadership to Eng. A.D. Wickramasinghe in the year 2001. He held office for a term of 3 years during which the society was able to secure our office space at the OPA secretariat.

Around this time, I stepped down as secretary and went on sabbatical leave at which Eng. Nihal Premachandra took over the position of honorary secretary. Upon my return, I had to step back in for the same position when he decided to migrate and move abroad for an employment.

Eng. S.A. Karunaratne took over the Presidency in 2003. Many booklets were published during his tenure including a guideline for constructing Tsunami resilient buildings and earthquake detailing manuals. It was also at around this time when the “Modulus” made its transition from monochrome to coloured printing. After many years of hard work, in January 2007, a draft of a bill was proposed in order to get the society incorporated by an Act of Parliament at which Mr. Earle Gunasekera lent his assistance to us by publishing the trilingual texts in the Government Gazette. Upon submitting it to the parliament in July 2007, going through various different government organizations, a few slight amendments and many other steps and hurdles later, the society was incorporated under Act 40 in 2009, giving more legal recognition and other useful agilities and representations.

Eng. D. A. Jayasinghe succeeded Eng. S.A. Karunaratne who held office for five years. I was elected Vice President at this point. After many successful seminars and question time sessions over the years we held our inaugural annual sessions in 2011. It was for this booming event that Eng. (Prof.) Raghu Chandrakeerthy donated funding for a gold medal to be awarded to the best paper annually.

Up until then, “Modulus” was published 3 times a year, there on we started publishing a quarterly volume of “Modulus”, gaining it journal status.

I was elected as president in 2012 and held the title for four years. In 2015, to celebrate a 25-year long journey of hard work and sacrifice by our pioneers, we organized our Silver Jubilee International Conference. This was a time where the industry had a decline. Therefore securing funding and sponsorships for an event of such magnitude seemed a daunting challenge. How ever Eng. Saro Weerasooriya and Eng. Deepal Wickramasinghe took this challenge on with stride and ventured out and succeeded in securing sufficient funds. Many renowned names in engineering were flown in from all around the world, in which Eng. (Prof.) Priyan Dias played a key role by utilizing his foreign affiliations. The conference was a 3-day long success story and was well attended by many students, academics and members of the industry. I must commend the organizing committee comprising of Eng. K.L.S. Sahabandu and Eng. Nandana Abeysuriya for more than stepping up to the occasion, committing to make an event of such grand scale a success.

I also had a feeling that our families deserved to be celebrated and appreciated as well for the sacrifices they have made during these difficult years. As a response to their understanding, we commenced our Annual President’s dinner, which to date is well received and participated.
Eng. K.L.S. Sahabandu was the next in line to take over running the Society for 3 years during which the society, after many years, was finally able to secure its recognition at the OPA as a professional association. His hard work also led to publishing 4 issues of “Modulus" per year. His office staff is also was also did a commendable job supporting the society activities.

Eng. Nandana Abeysuriya succeeded him, and it was a new era of young structural engineers being elected to the committee as myself and Eng. S.A. Karunaratne along with a few others stepped down to pursue personal commitments. Along with youngsters came new ideas and many events were conducted during this time including the now widely popular Hopper night which is held at the conclusion of the monthly question time session. Eng. Nandana Abeysuriya also took the initiative to establish the Associate member chapter of SSESL.

His 2-year term came to an end with the election of, for the first time in over 3 decades, our first female president, Eng. (Mrs.) Tharangika Jayasundara. If this was an intentional occurrence throughout the years or not remains a mystery. One of our former guest speakers once stated that, “women are better leaders”, which I was initially somewhat sceptical of, but she has more than stepped up to her role and proven a great point. So, well done Tharangika.

Okay folks, so that’s where we are at today. During the 33 years we conducted over 40 seminars and numerous volumes of Modulus improving and enhancing the quality of content and digital footprint with each year. We must acknowledge a job well done by the editors throughout the years that worked tirelessly to keep up with the timely publication and accumulating content.

As they say, you learn as you grow, which truly was the case for us throughout the years. So, all I have to say to the future committees is to learn from anything that may have been lacking in the past and don’t be afraid to take risks. I wish the youngsters all the very best and I’m thrilled to witness the society blossoming to new heights in the future.

Hope you all have a pleasant and enjoyable evening.

Thank you!

Eng. (Prof.) Ruwan Weerasekera
BSc. Eng. (Hons),
PhD (Calgary),
CEng., MIE(SL), HFSSESL

Past President
Society of Structural Engineers
Sri Lanka

Outgoing President's Speech at the AGM

Photograph 23: Eng. (Mrs.) T.J. Jayasundara; The Outgoing President addressing the AGM.

Past Presidents, Honorary Fellows, Fellows, and Members

All protocols Observed, ladies and gentlemen

It gives me great pleasure and honor to be standing in front of you today, at the exit of the second term of my portfolio as the 9th President of the Society of Structural Engineers, Sri Lanka.

Looking back on the past 12 months, it has been an exceptionally busy year. We have had to deal with a rapid succession of important events and situations. Indeed, for me, personally, it has been a very eventful period throughout. Times challenging, but on the whole a very rewarding experience.

Today, it is not my intention to list out all that we have accomplished. I believe they speak for themselves. Rather, I would like to take this opportunity to briefly offer some of my impressions on how things have fared in the work of the SSESL.

What comes to my mind first and foremost is the strong and active engagement of the Executive Committee in achieving the core objectives of the SSESL

I would like to thank the Executive committee members who served with a high level of commitment. As we all know, our work demands a high level of integrity and competence as a variety of skills are expected by all Exco Members. I am privileged to have had the opportunity to work with a well-balanced team blended with highly experienced practicing engineers and renowned academics, together with young blood who came forward to take any kind of challenges beyond their comfort zones.

I have received the support of the immediate past and past presidents, including the late (Eng.) S A Karuanratne, the fourth president of SSESL who passed away during the term. If I can start unpacking this one; we will never be out of this place. The constitution of SSESL does not have any provision for a president forum, but we have put it in place and engaged them on any issues that crop up on the SSESL leadership and they have no hesitations to sacrifice their time and are prepared to go an extra mile every time upon my request.

Our sponsors have been so wonderful, their level of commitment has relieved me a lot. Their plans to provide continued support to our Seminars could not be aborted by our initial plans to organize one seminar for the year. As a result, we were compelled to organize the second. I thank them for being there to ensure that SSESL succeeds in delivering on our mandate of an exceptionally high standard of quality of the events. I must gratefully acknowledge the generous sponsorship made by our sponsors, as they are a part of this success.

SSESL Secretariate staff, Indika Dilki, and Hashan gave us untiring support.

I hope the Hon. Secretary's Annual Report has succeeded in indicating to you the amount of work that has been done in our volunteer positions within SSESL. Having done much within this term does not mean that there is no work for the incoming exco. The to-do bag just from the AGM is already full. I would like to urge the young members who are passionate about SSESL and have a commitment to serve, to avail yourselves when it comes to the election of the next Executive Committee. It is important to get loyal members on board to steer the SSESL in the right direction for the next couple of decades too.

At this point, it is appropriate for me to overlook my journey at SSESL and salute those who brought me to SSESL and encouraged me to continuously engage in it. I take this opportunity to pay tribute to the late Eng. B. A. Dayananda who has guided me and encouraged me to engage in SSESL activities from the early days of my career. Then I wish to place on record the guidance received by Eng. K. L. S. Sahabandu, Past President who brought me to the Executive Committee. I also wish to thank all members of SSESL for electing me as the president.

In closing, it has been my honor to represent SSESL as the President over the last two years which, I have thoroughly enjoyed my tenure. I am humbled to have the opportunity to lead this esteemed Society and look forward to its continued success.

Thank you very much.

Eng. (Mrs.) T.J. Jayasundara
BSc. Eng. (Hons),
MEng.(Struct. Eng. Design),
CEng., MIE(SL), MSSESL, MConsE(SL)

Immediate Past President
Society of Structural Engineers
Sri Lanka

President's Message

Photograph 24: Eng. R.M.A. Senerath; The President addressing the AGM.

Dear Members of SSESL and fellow Engineers

It is with great honor that I send this message as the newly appointed President of the Society of Structural Engineers, Sri Lanka, the representative body of practicing engineers and academics active in the field of structural engineering. I express my sincere gratitude to all of you for having trust in me to lead the Society. I am committed to serving the Society to the utmost, in order to uphold the current standing and propel it to the next level.

Firstly, I extend my gratitude to the outgoing President and the Executive Committee, for their invaluable contribution to the success of SSESL. I also take this opportunity to extend a warm welcome to the newly appointed Executive Committee and express my eagerness to collaborate with all of you for the betterment of the Society, our members, and indeed, the entire nation.

The establishment of the SSESL was to safeguard the interests of our members and provide them with a wide range of knowledge in the field of Civil and Structural engineering through various platforms. Promoting and advancing the science and practice of structural engineering, organizing and arranging; seminars, lectures, symposiums and discussions, maintaining libraries and publishing papers and books on engineering, etc. among the objectives set forth as its inception. As the newly appointed President, I am committed to strive for excellence and achieve our goals during my tenure.

As we are all aware of the challenges posed by COVID 19 and the political instability in the country have significantly impacted the construction industry, resulting in a decrease in major projects for all Structural Engineers. However, there are promising signs of the industry picking up lately, which will eventually benefit all of us. Let us unite, work together and re-build our industry.

Lastly, I want to emphasize that our industry is currently smaller in scale than it was in the past, necessitating our openness to the global market. Several local practices have been offered overseas assignments indicating opportunities to tap the global construction sector. Therefore, it is imperative that we collaborate to excel in the global arena and provide our expertise in structural engineering to meet their international standards and satisfaction.

Thank you.

Eng. R. M. A. Senerath
BSc. Eng. (Hons),
CEng., MIE(SL), MICE (UK), FSSESL, MSLGS, MConsE(SL)

President
Society of Structural Engineers
Sri Lanka

Controlling lateral deformations in pretensioned precast elements

T.W.M.C. Wanigasooriya[1], W.G.D.R. Wathshala[2], K. Baskaran[3]

Abstract

The design and manufacturing of pretensioned precast elements such as bridge beams and electricity poles in Sri Lanka is a challenge task. Lateral deformations in such pretensioned precast elements can significantly affect their use, structural performance, and serviceability of these vital infrastructure components. This study focuses on methods to reduce lateral deformations in prestressed concrete elements, a crucial aspect of ensuring structural integrity and safety. The study begins with a comprehensive identification of the factors that can influence lateral deformations in pretensioned precast elements. Through an in-depth literature review and consultation with experts in the field, key factors affecting lateral deformations, such as material properties, design parameters, construction methods, and environmental conditions, were identified and analysed. This comprehensive understanding of the contributing factors formed the basis for the subsequent stages of the research. In this study, MIDAS Civil finite element (FE) software was employed to investigate the lateral moments induced at the midspan of the precast beam when changing the order of the destressing sequence. Additionally, the study delved into a parametric investigation to explore a range of variables and scenarios. The results of the analysis indicate that variations in prestress forces, as simulated in Alternative 5 with a 10% higher prestress load, lead to higher lateral moment values during the destressing process. The research outcomes contribute to the development of guidelines and recommendations for controlling lateral deformations in pretensioned precast elements.

1. Introduction

Prestressed concrete is a structural engineering technique used to enhance the performance of concrete in structures. It involves introducing internal stresses to the concrete before it is subjected to external loads, which helps to counteract the tensile stresses that typically cause cracking and structural failure in concrete (Yee et al., 2001). Prestressed concrete relies heavily on high-strength steel wires or bars, known as tendons, to introduce compressive forces into the concrete. Prestressed concrete is a type of concrete in which the concrete is initially compressed before an external load is applied (Mateckova et al., 2021). This allows the stress from the external load to be counteracted in the desired way during the service period. After lengthy beams have been destressed, there have been multiple instances of lateral deformation of long prestressed beams. There are various problems encountered during the construction of the bridge deck due to laterally deformed beams.

Lateral deformations refer to the movement or deflection of the precast elements in a direction perpendicular to their longitudinal axis. These lateral deformations significantly impact the quality of the prestress precast elements. Therefore, further attention should be paid to maintain deformations within a permissible level. Otherwise, users may not be able to use these elements for their intended purposes. If excessive lateral deformation occurs; appearance may be severely affected, and there may be additional moments or deformations. Several factors contribute to the lateral deformations in pretensioned precast elements. According to Murugan Usha Rani (2021), finishes, partitions, and related structures are likely to sustain damage from excessive lateral deflections. As a result of that, prestressed elements that have undergone lateral deformation are cast away without use in the industry, and it is a substantial economic loss.

Therefore, understanding the root causes for lateral deformations, remedies, current practices used for manufacturing pretensioned precast elements in Sri Lanka, and methods that can be used to reduce lateral deformation in prestressed elements are critical areas considered in this study. The objectives of this study are to minimise and control the lateral deformations during the design and manufacturing process and to prepare guidelines with scientific reasoning to control the lateral deformations of pretensioned elements. It will help to minimise the cost and wastage while improving the design and manufacturing process of pretensioned precast elements in Sri Lanka.

2. Literature Review

Prestressing effectively balances or neutralises the tensile stresses of applied loads by transferring compressive forces from the tendons to the concrete (Naaman, 2004). Prestressed concrete enhances load-bearing capacity, reduces deflections, but raises concerns about lateral deformations due to various contributing factors. The bending moment distribution along the span and the flexural rigidity of the members control the short-term or instantaneous deflections of prestressed members. Self-weight and superimposed load, the magnitude of the prestressing force, the cable profile, the second moment of the area of the cross-section, the modulus of elasticity

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of the concrete, shrinkage, creep, and relaxation of the steel stress, the span of the member, and the fixity conditions are the factors that affect deflections (Libby, 1990; Nilson, 1987).

The final deflection that develops after the construction of partitions and finishes should not be greater than the lesser of span/350 or 20 mm (not including the effects of temperature, creep, and shrinkage). The entire upward deflection should be calculated before finishing, not exceeding span/300. The ultimate deflection (including creep, shrinkage, and temperature effects) should not exceed span/250 (Usha Rani, 2021). The temperature gradient will not exist when curing with wetted gunny bags in Sri Lankan conditions because the temperature will be nearly uniform throughout the beam. Some factors causing lateral deformations are described in the subsequent sections.

2.1 Creep and Shrinkage

According to Kwak and Seo (2002), "creep" describes the time-dependent deformation or gradual increase in strain that a material undergoes when subjected to a sustained load. The constant application of prestressing force to the tendons causes creep in prestressed concrete girders. Over time, the concrete gradually deforms and is subjected to more stress. The viscoelastic behaviour of concrete is thought to be responsible for this phenomenon (Zhu et al., 2020). Creep in pretensioned concrete girders alters stress distribution, leading to tendon stress increase and concrete stress reduction. This can induce long-term deflection, so engineers consider it during design to limit deflection.

"Shrinkage" refers to the reduction in material volume due to moisture loss. Pretensioned concrete girders shrink as they dry and undergo hydration. Water evaporation during curing causes this volume loss. Autogenous shrinkage, from chemical reactions, and drying shrinkage, due to moisture loss, can both occur. Shrinkage in pretensioned concrete girders may lead to cracks and internal force variations, affecting the structure's appearance and durability. Mitigation measures include shrinkage-reducing admixtures, control joints, and proper curing techniques used to reduce the effects of shrinkage (Zhu et al., 2020).

2.2 Imbalance of Tendon Prestress Force

Unbalanced prestressing forces are one of the most frequent causes of lateral deformation in pretensioned precast bridge girders. When high-strength steel strands are tensioned, and the load is later transferred to the concrete of a beam, a prestressing force is produced. Doing this generates a force that opposes the bending moment brought on by active loads, such as traffic. If the prestressing force is not balanced, it can create a bending moment that causes the beam to deflect laterally. Common factors that can create imbalanced prestress force are improper placement of the prestressing strands, damage to the prestressing strands, and variations in the concrete strength. Therefore, the best way to prevent lateral deformation is to balance the prestressing force. This can be done by carefully placing the prestressing strands and by monitoring the concrete strength.

2.3 Destressing Sequence

The destressing sequence in pretensioned concrete involves releasing or cutting prestressing strands in a specific order once the concrete attains adequate strength. This is critical for precast concrete elements, as it affects structural integrity, minimises lateral deformation, and enhances load-bearing capacity. High-strength steel strands are tensioned and anchored before concrete casting, generating compressive forces within the element once destressed.

The destressing sequence aims to minimise lateral deflection in concrete elements. Cutting prestressing strands can create imbalances and induce lateral deformation, affected by factors like strand arrangement, cutting sequence, and concrete stiffness. Engineers and construction professionals carefully plan the destressing sequence to prevent excessive lateral deformation. They may use computer simulations and structural analysis to determine the optimal order in which strands should be cut. The goal is to distribute the loss of prestress force as evenly as possible to minimise unbalanced bending moments due to prestress and subsequent lateral deflection (Ghali et al, 2002). Monitoring and quality control measures are essential during the destressing process to ensure that the actual behaviour of the concrete element aligns with the design specifications. Adjustments may be made as necessary to achieve the desired structural performance.

In summary, the destressing sequence is a critical aspect of pretensioned concrete construction. Proper planning and execution of this sequence are essential to control and minimise lateral deformation, ensuring that the finished precast concrete elements meet the required standards and perform as intended in their final application.

3. Field Survey

To gain practical insights, visits were made to two precast yards: the precast yard A of the State sector and the precast yard B of the Private sector. During these visits, thorough measurements of deviations in the lateral direction were taken, mainly focusing on excessively deformed beams. Photographs taken during the field measurements are shown in Figure 1.

Figure 1: Getting measurements of lateral deformations in electricity poles

Additionally, the current manufacturing practices employed for electricity poles and bridge girders were studied, providing valuable real-world context. In both visited sites, the long line method is used in the manufacturing process. The formwork, or mould, for the electricity poles is prepared. It is usually made of steel and is designed to create the desired shape and dimensions of the pole. The formwork includes provisions for the placement of prestressing tendons. High-strength steel tendons are placed in the formwork according to the design requirements.These tendons will provide the prestressing force to the pole once the concrete has hardened. The following Figures, 2 and 3, show the long line method and formwork used to produce electricity poles, respectively.

Figure 2: Long line method

Figure 3: Formwork arrangement for electricitypoles

Once the reinforcement and prestressing tendons are in place, concrete is poured into the formwork. The concrete mixture is designed to meet the specified strength and durability requirements. It is carefully poured and compacted to ensure proper consolidation. After the concrete has fully cured, the tendons are precisely cut at the anchorages, transferring the prestress to the concrete, and allowing the member to elastically shorten to its original length.

Figure 4: Top end of the electricity pole before plastering

Figure 5: Top end of the electricity pole after plastering

4. Validating of the FE model

Finite element (FE) models are often used to predict the behaviour of pretensioned bridge girders under various loading conditions. However, it is essential to validate these models against experimental data to ensure that they are reliable. One way to validate FE models of pretensioned bridge girders is to compare the predictions of the model to the results of static loading tests.

In the present study, a static loading test was performed to validate the FE model of a pretensioned bridge girder. The girder was subjected to a concentrated load at mid-span, and the downward deformations were measured at three specific locations: at the centre of the girder and 1/4th of the span on either side of the centre, as shown in Figure 6. FE model of the pretensioned bridge girder is modelled using MIDAS Civil FE software. The excellent agreement between the experimental results and the predictions of the FE model indicates that the model is valid and can be used to predict the behaviour of the bridge girder under various loading conditions.

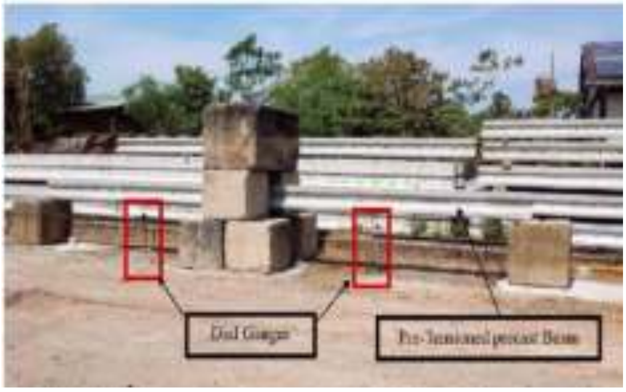

Figure 6: Test arrangement to measure downward deflection

The following figure shows the locations (B, C and D) where the deflection measurements were taken for the validation of the FE model.

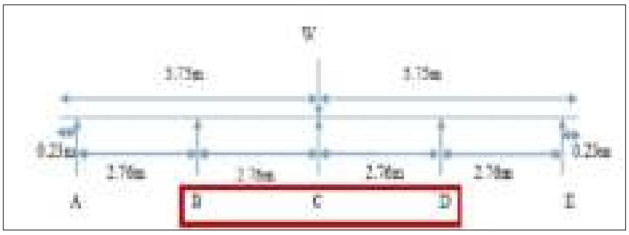

Figure 7: Load arrangement

Figures 8, 9 and 10 show the deflection of the bridge girder with respect to different loading conditions, such as self-weight, prestress force, and point load, respectively.

Figure 8: Deflection due to self-weight of the girder

Figure 9: Deflection due to prestress force

Figure 10: Deflection due the point load

Deflections based on the load-deflection test and the deflections based on the FE model of point B (0.25L), point C (mid span) and point D (0.75L) were compared, and the following graph indicates the summary of the results obtained from the comparison.

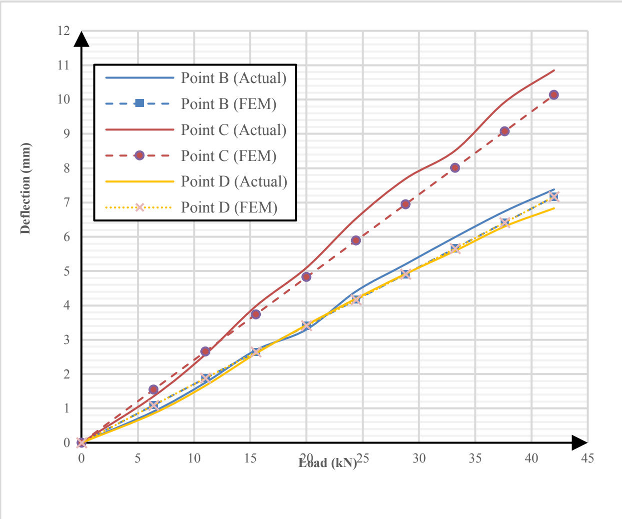

Figure 11: Experiment and Numerical Results

The above graph indicates the load values along with the actual and FEM net deflection values at three different points (Point B, centre (Point C) and Point D) of the precast girder beam, along with the deviation percentages between actual and FEM values for each point. The analysis of the provided data reveals notable trends and discrepancies between actual and finite element analysis (FEM) net deflection values for varying loads at different points. Notably, the FEM model consistently underestimates deflection, particularly at Point B and the Centre, across the load range. Deviation percentages, although generally within 10%, may have significance in structural engineering applications.

5. Results and Discussion

The optimisation of a destressing sequence is a critical step in the construction and maintenance of prestressed concrete structures. These sequences involve the order in which prestressing strands or tendons are cut or released from a concrete element. The goal is to minimise the impact on the structural integrity of the element while achieving the desired prestress balance. In this study, four alternative destressing sequences were compared, and Figures 12, 13, 14 and 15 illustrate those four alternative destressing sequences.

Figure 12: Alternative 01

Figure 13: Alternative 02

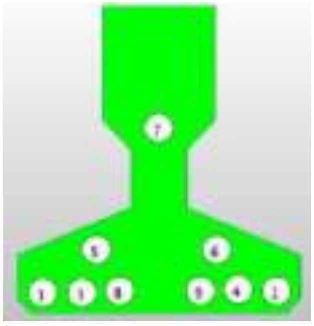

Figure 14: Alternative 03

Figure 15: Alternative 04

In this study, MIDAS Civil software was employed to thoroughly examine and evaluate the lateral moments induced at the mid span of the precast beam when changing the order of the destressing sequence. The resulting test data, tabulated in Table 1, offers valuable insights into the structural behaviour of the beams when subjected to lateral loading conditions.

Destressing steps	Lateral Moment at Midspan (kNm)			
	Alt_1	Alt_2	Alt_3	Alt_4
1	8.8	17.5	75.4	13.1
2	0	0	0	0
3	9.3	12.7	49.7	8.6
4	0	0	0	0
5	16.6	8.3	12.1	16.6
6	0	0	0	0
7	0	0	0	0
8	23.6	23.6	7.8	23.6
9	0	0	0	0

Table 1: Lateral moments induced due to destressing sequences

The above table presents the data related to lateral moments at mid span for different alternatives at various destressing steps. According to the obtained results, Alternative 3 exhibits significantly higher lateral moments than the other alternatives, particularly at destressing steps 1 and 3. It has the highest moment value at step 1, indicating a substantial initial lateral force. This is due to the reason that the initial destressing step was done for the tendon, which is most away from the symmetrical axis in

Alternative 3. Therefore, it induces a higher lateral moment.

Nevertheless, in Alternative 1, the initial destressing step was done to the tendon, which is located closer to the symmetrical axis. Therefore, it caused a relatively lower lateral moment. Destressing steps 1,3,5 and 8 have nonzero lateral moments, indicating that lateral forces are active during these steps. In destressing, steps 2,4,6 and 9 have zero resultant lateral moments for all alternatives since the occurrence of the lateral forces in steps 1.3.5 and 8 are equal and opposite to the 2,4,6 and 8 steps. Therefore, the destressing process must be done symmetrically. If someone initially destresses the left-side tendons and then goes to the right-side tendons, there is a higher possibility of getting lateral deformation considerably.

Lateral deformation in concrete structures can often be attributed to variations in prestress force during the initial stressing stage. Such variations may arise from issues with the jacking machines used in the prestressing process. In the context of this study, an attempt was made to simulate this behaviour using an alternative, denoted as "Alternative 5," which shares the same destressing sequence as "Alternative 4". However, the critical distinction lies in the fact that Alternative 5 applies a prestress load that is 10% higher on the left-hand side tendons. This deliberate adjustment aims to replicate the impact of varying prestress forces during the stressing stage and assess its influence on lateral deformation. It offers valuable insights into the structural behaviour and potential deformation patterns in concrete elements under these conditions.

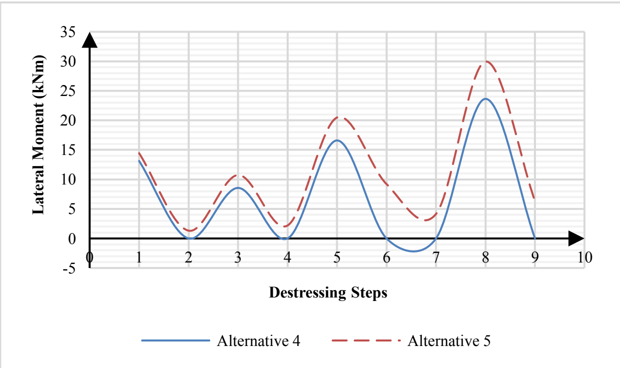

Figure 16: Lateral moment at mid-point of the precast

Figure 16 represents data on lateral moments at mid span for two alternatives, Alternative 4 and Alternative 5, at various destressing steps. In all the cases, Alternative 5 exhibits higher lateral moments compared to Alternative 4. This indicates that the application of a 10% higher prestress load on the left side tendons in Alternative 5 results in increased lateral forces and moments during the destressing process. At destressing step 1, Alternative 5 has a slightly higher lateral moment (14.4 kNm compared to 13.1 kNm) than Alternative 4, suggesting that the increased prestressed load on the left side tendons to produce a slightly higher initial lateral moment. At step 2, Alternative 5 shows a nonzero moment (1.3 kNm), while Alternative 4 remains at zero. This implies that the higher prestress load in Alternative 5 has initiated some lateral force, which is not observed in Alternative 4. Similar trends continue at subsequent steps, with Alternative 5 consistently displaying higher lateral moments than Alternative 4. The deliberate increase of the prestress load on the left-hand side tendons in Alternative 5 has a noticeable impact on lateral moments during the destressing process. Higher prestress loads result in greater lateral forces and moments, contributing to increased lateral deformation potential. These findings emphasise the sensitivity of prestressed concrete structures to variations in prestress forces and underscore the importance of precise prestressing procedures. Therefore, engineers and construction professionals should carefully control and monitor prestress loads and sequences to minimise the risk of excessive lateral deformation in concrete elements.

In summary, the analysis indicates that variations in prestress forces, as simulated in Alternative 5 with a 10% higher prestress load, lead to higher lateral moments during the destressing process. This information highlights the significance of proper prestressing techniques and their direct influence on the lateral behaviour of prestressed concrete structures.

6. Conclusions

This study focuses on investigating strategies to effectively control lateral deformations in pretensioned precast elements, a crucial aspect of ensuring structural integrity and safety. To gain comprehensive insights, the research commenced with field visits to identify and document current manufacturing and design practices. Notably, the study singled out electricity poles displaying excessive lateral deformations, using the deviation from straightness as a pivotal indicator of the problem. Data collection was thorough, and awareness regarding the experimental process was heightened.

Subsequently, the gathered data was leveraged to facilitate a rigorous validation process that compared FE Model (FEM) results with actual observations. This critical step aimed to gauge the accuracy of the FEM predictions in simulating lateral deformations. Furthermore, the study delved into a parametric investigation, utilising the MIDAS CIVIL model to explore a range of variables and scenarios. Ultimately, this comprehensive research endeavour culminated in drawing insightful conclusions that can inform best practices and strategies for effectively managing lateral deformations in pretensioned precast elements, contributing to safer and more robust construction practices.

During the field visit conducted as part of this study, several significant observations were made, highlighting areas that require improvement in the design and manufacturing processes for pretensioned precast elements. One notable finding was the inadequacy of detailed drawings related to the design and manufacturing procedures. These drawings were found to contain limited basic information about the pre-tensioning elements, lacking essential details such as a proper destressing sequence with clear reasoning.

To address these shortcomings, a series of modifications were identified during the field visit.

- It was recommended to use high-strength concrete during the pouring process. This choice not only enhances the structural integrity of the elements but also emphasises the importance of stringent quality control measures in manufacturing.
- To ensure the stability and accuracy of the precast elements, the provision of a sufficient number of supports was advised. Additionally, frequent checks of support levels were recommended to maintain precise alignment and positioning.
- Maintaining consistent cover thickness on both sides of the elements was identified as an essential practice. This uniformity contributes to the structural uniformity and durability of the precast elements.
- A critical aspect of the recommendations involved adopting a more efficient destressing sequence. Special attention was emphasised on determining the optimum destressing sequence that minimises lateral loads.
- Frequent calibration of the jacking system was suggested as good practice. The use of jacking systems equipped with elongation and gauge reading capabilities was recommended for improved accuracy and control.

Regarding the destressing sequence, Alternative 1 was identified as the most suitable option. This sequence suggests that tendons located further away from the symmetrical axis should not be destressed initially but rather in the final stages of the process. This approach aims to minimise lateral loads and deformations, contributing to the overall structural integrity and safety of pretensioned precast elements. These recommendations collectively serve to enhance the quality and reliability of precast element manufacturing processes while minimising the potential for lateral deformations and structural issues. Table 2 illustrates the checks that should be considered during the manufacturing process of pretensioned precast elements.

Table 2: Checklist for manufacturing pretensioned precast elements

HOLD POINT	ITEM TO BE CHECKED
Before Concreting	Detailed design plans and specifications are available and approved
	Ensure the design drawings, including dimensions, materials, prestressing requirements, and concrete mix design
	Ensure the moulds and formworks are fabricated to the correct dimensions and profiles
	Cleanliness of formwork and free of debris and contaminants
	Placement of prestressing strands or bars according to the design requirements
	Confirm proper spacing and alignment of tendons
	Apply tension to prestressing strands according to the design requirements
	Monitor and record the applied tension force
	Verify proper anchoring and bonding of tendons
	Check for adequate cover spacings
	Verify the grade of the concrete according to the design requirements
	Check the accuracy of the batching equipment
	Ensure proper batching procedures
During Concreting	Inspection for stability of formwork, grout leaks, etc.
	Eliminate air voids and achieve uniform concrete density using vibrators
	Prevent segregation and ensure proper consolidation
	Monitor concrete temperature and slump during concrete
	Check for the final level of the finished concrete
After Concreting	Implement curing procedures immediately after concrete placement
	Destress the tendons in an orderly manner after the strengthening of concrete
	Avoid damages in de-moulding precast girders
	Check for the surface finishes and coatings as per design standards.

This above-mentioned checklist serves as a guideline to ensure that each phase of manufacturing pretensioned precast bridge girders is carefully monitored and controlled to produce structurally sound and reliable components.

7. Acknowledgement

The authors wish to acknowledge the assistance given by the academic and technical staff of the Department of

Civil Engineering, University of Moratuwa and Eng. R.M. Rathnasiri, who assisted with the field survey and Dr. P.L.N. Fernando, who assisted with the finite element modelling part of this study.

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Fatigue Verifications of Reinforcement & Concrete in Wind Turbine Foundations

Sumedha Mayadunna

Abstract

This abstract encapsulates the structural design and verification process of a wind turbine foundation for the large-scale wind farm project on Mannar Island, Sri Lanka The project, awarded to Vestas Asia Pacific AS by the Ceylon Electricity Board, entailed the installation of 30 cutting-edge wind turbines, each rated at 3.45 MW, resulting in a total capacity of 103.5 MW. The foundation's intricate design, composed of a 15.5m diameter pile cap supported by 15 bored cast-in-situ piles and a 5.2m diameter plinth, aimed to ensure stability and durability amidst fluctuating fatigue loads. The strategic use of counterbalance weight atop the pile cap was explored as a cost-effective means to enhance stability against strong winds and uneven loads. The document detailed the foundation loads, extreme and serviceability loads, fatigue loads, and rain flow count, employing EN 1992-1-1 guidelines. Specific focus was given to flexure design, fatigue verification of concrete and reinforcement, and verification procedures for concrete under shear-radial moment. Extensive calculations, S-N curves for reinforcement steel, and Palmgren-Miner rule applications ensured robust fatigue verification. Verification of concrete under compression or shear was examined using EN 1992-1-1 & EN 1992-2:2005 standards. The comprehensive analysis and verification processes outlined in this abstract demonstrate a rigorous approach to ensuring the structural integrity and reliability of wind turbine foundations, crucial for sustainable and resilient renewable energy infrastructure.

1. Introduction

The EPC contract for the 1st large-scale wind farm in Sri Lanka, located on the southern coast of Mannar island, was awarded to Vestas Asia Pacific AS, a well-known Danish wind turbine manufacturer, by the Ceylon Electricity Board (CEB). The project consists of 30 state-of-the-art wind turbines, each with a rating of 3.45 MW, resulting in a total installed capacity of 103.5 MW for the wind farm.

As the subcontractor for the design of the substructure foundation and civil construction of the project, the Design Division of Access Engineering PLC was involved in the structural design of WTG foundations. Due to the newness of the area of design verifications for fluctuation fatigue loads according to the En 1992 1-1; 2004, I would like to share our experience with professionals in this industry.

Figure 1:Installed Wind Turbines at Mannar Island

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1.1 Basic Layouts of Wind Turbine Foundation

The wind turbine foundation consists of a 15.5m diameter pile cap that varies in height from 2480mm at the plinth to 900mm at the edge of the pile cap. The pile cap is supported by 15 bored cast-in-situ piles, each with a diameter of 800mm. The foundation also includes a 5.2m diameter plinth with a height of 3450mm and a 92x2 anchor cage. To provide counterbalance and stability, plum concrete is designed to be placed on top of the pile cap. A section is provided to better visualize and understand the arrangement.

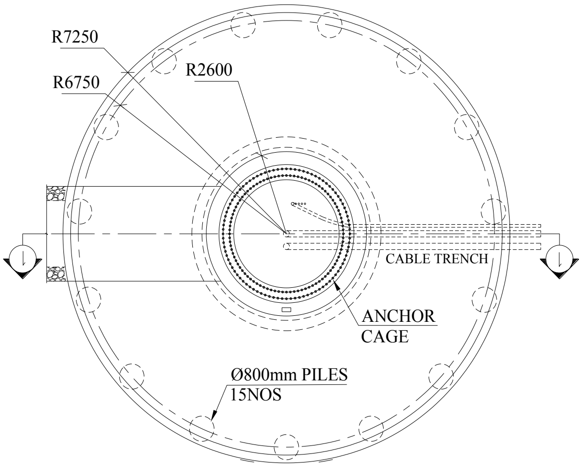

Figure 2: Basic Layouts of Wind Turbine Pile Cap

Adding additional counterbalance weight on top of a wind turbine foundation's pile cap can indeed help stabilize and optimize the cost and stability of its structure. By strategically placing the counterbalance weight, it can reduce the potential for tilting or tipping caused by strong winds or uneven loads. This approach can be an effective way to enhance the overall stability of the wind turbine foundation while minimizing additional construction costs.

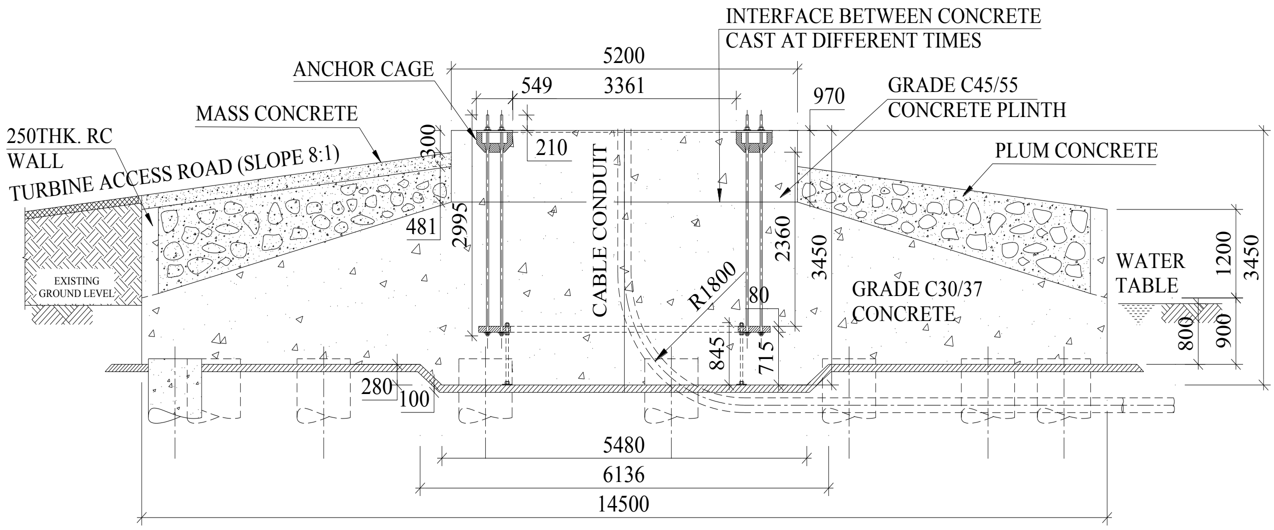

Figure 3:Section through Wind Turbine Pile Cap

2. Methodology

2.1 Foundation loads

This document presents the foundation loads from the V126-3450 kW, GS, 50/60Hz, HH 87.0 on the site; Mannar Island. The foundation loads are based on the site-specific climate in Table 1-2 and calculated for a design lifetime of 20.0 years.

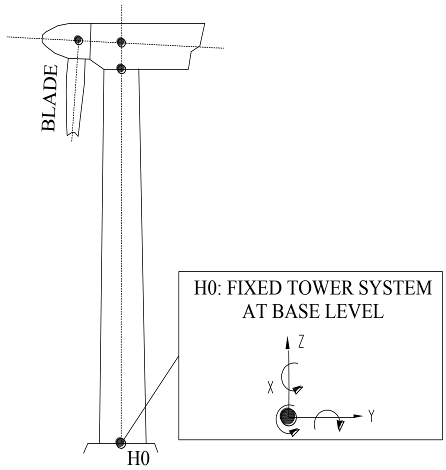

Figure 4:Wind mast indicating the location of the tower bottom forces considered for foundation design

2.1.1 Extreme loads

Extreme foundation loads are presented in the following table. The lead sensor indicates the extreme value and the other sensors are time-coherent to the extreme value

Table 1; Characteristic Extreme Load cases (Part 1 of 2)

Lead	**Load Case**	**PLF**	**Type**	**Mbt1**
Sensor	[-]	[-]	[-]	[kNm]
Mbt1	Ext-1	1.1	Abs	78920
Mzt1	Ext-2	1.35	Abs	18680
FndFr	Ext-3	1.1	Abs	78630
Fzt1	Ext-4	1.35	Abs	26310

Table 2; Characteristic Extreme Load cases (Part 2 of 2)

Lead	**Load Case**	**Mzt1**	**FndFr**	**Fzt1**
Sensor	[-]	[kNm]	[kN]	[kN]
Mbt1	Ext-1	2623	969.8	-3759
Mzt1	Ext-2	-6336	217.3	-3744
FndFr	Ext-3	2682	982.2	-3765
Fzt1	Ext-4	2106	338.5	-4007

2.1.2 Serviceability loads

The serviceability loads are calculated as the 99% fractile of the load level during normal production.

Table 3;Characteristic Extreme Serviceability Loads

Sensor	**Unit**	**Load**
M_{res}	[kNm]	46500.00
M_z	[kNm]	-2711.00
F_{res}	[kN]	523.20
F_z	[kN]	-3912.00

2.1.3 Fatigue loads

For the foundation, the mean loads have to be considered. The mean loads must be combined with either the equivalent loads or the fatigue load spectrum.

2.1.3.1 Mean fatigue load

The mean fatigue load refers to the average or steady-state component of cyclic loading experienced by a structure over time. It represents the constant or quasi-static portion of the cyclic loading, such as gravitational forces, wind loads, or other steady-state loads acting on the structure. In fatigue analysis, the mean fatigue load is important because it influences the overall stress level experienced by the structure and can contribute to fatigue damage accumulation.

Table 4; Equivalent and Mean Fatigue Foundation Loads

Sensor	Unit	Mean load
Fy	[kN]	279.40
Mx	[kNm]	-23640.00
Mz	[kNm]	-171.20

2.1.5 The Markov matrix

The Markov matrix is utilized in rain flow counting to efficiently identify and characterize the stress cycles. It helps in grouping similar stress ranges and frequencies together, simplifying the analysis process. By using the Markov matrix, transitions between different stress levels can be tracked, aiding in the accurate determination of stress cycles and their respective amplitudes.

2.1.4 Rain flow count

The table below presents the rain flow spectrum extracted from the fatigue load spectrum for shear force, bending moment, and torsional moment at the base of the tower. The frequencies indicated in the table correspond to the entire lifespan of the turbine.

The stress-life curve for wind turbine analysis is selected based on the intended lifespan of the turbine. Factors such as expected operating conditions, maintenance schedules, industry standards, safety margins, material properties, and environmental conditions are considered to ensure reliability and durability over the turbine's lifespan

Table 5; Rain flow spectrum extracted from the fatigue load spectrum (Extract from Markov Matrices)

LC	Tower shear, bottom Fy [kN]		Tower bending, bottom Mx [kNm]		Tower torsion, bottom Mz [kNm]	
	Range	Frequency	Range	Frequency	Range	Frequency
1	1.0751E+3	7.5778E-1	8.0845E+4	1.0666E+0	1.1464E+4	9.0500E+1
2	1.0321E+3	3.5484E-1	7.7611E+4	1.0348E+0	1.1006E+4	1.8225E+2
3	9.8912E+2	9.4434E+0	7.4377E+4	1.4269E+1	1.0547E+4	1.2599E+3
4	9.4611E+2	4.9589E+0	7.1144E+4	9.2168E+0	1.0088E+4	1.5751E+3
5	9.0310E+2	7.2359E+0	6.7910E+4	2.7780E+1	9.6299E+3	3.2717E+3
6	8.6010E+2	3.4304E+1	6.4676E+4	5.4338E+1	9.1714E+3	9.2114E+3
7	8.1709E+2	1.8183E+1	6.1442E+4	4.0096E+1	8.7128E+3	1.9403E+4
8	7.7410E+2	3.7572E+1	5.8209E+4	6.2208E+1	8.2542E+3	6.7429E+4
9	7.3109E+2	5.0829E+1	5.4974E+4	5.1348E+1	7.7957E+3	1.3953E+5
10	6.8808E+2	3.5894E+2	5.1741E+4	8.0019E+1	7.3371E+3	3.3491E+5
11	6.4508E+2	1.0881E+3	4.8507E+4	1.5287E+2	6.8785E+3	5.3899E+5
12	6.0207E+2	3.3500E+3	4.5273E+4	2.6823E+2	6.4200E+3	6.6368E+5
13	5.5906E+2	8.4140E+3	4.2039E+4	1.7883E+3	5.9614E+3	7.5886E+5
14	5.1606E+2	1.7990E+4	3.8806E+4	3.0230E+3	5.5028E+3	1.2618E+6
15	4.7305E+2	3.0762E+4	3.5571E+4	1.4945E+4	5.0442E+3	1.7811E+6
16	4.3005E+2	6.7287E+4	3.2338E+4	4.2998E+4	4.5857E+3	2.8286E+6
17	3.8705E+2	1.1445E+5	2.9104E+4	8.0035E+4	4.1271E+3	4.8508E+6
18	3.4404E+2	2.0452E+5	2.5870E+4	1.6287E+5	3.6685E+3	8.0701E+6
19	3.0103E+2	3.9050E+5	2.2636E+4	2.5225E+5	3.2100E+3	1.4304E+7
20	2.5803E+2	8.7296E+5	1.9403E+4	4.6495E+5	2.7514E+3	2.3591E+7
21	2.1502E+2	1.9184E+6	1.6169E+4	1.1080E+6	2.2928E+3	3.7255E+7
22	1.7202E+2	5.4474E+6	1.2935E+4	2.6350E+6	1.8343E+3	5.6429E+7
23	1.2902E+2	1.9252E+7	9.7013E+3	7.7638E+6	1.3757E+3	7.4554E+7
24	8.6010E+1	8.7244E+7	6.4676E+3	3.5548E+7	9.1714E+2	9.2156E+7
25	4.3005E+1	6.6067E+8	3.2338E+3	5.5989E+8	4.5857E+2	1.9835E+8

2.2 Structural modelling and analysis 0f WTG Pile Cap

2.2.1 FEM Modeling & Lateral soil spring

The structural analysis for the pile cap involves finite element modeling, utilizing shell elements. Piles within the wind turbine generator foundation are represented using frame elements. To simulate soil and pile interaction, joint springs are employed. Lateral soil Spring constants was calculated according to the following Methods;

For cohesionless soils [Chen (1978)]: $k_s = \frac{3E_{pm}}{B}$

And for cohesive soils: $k_s = \frac{1.6E_{pm}}{B}$

Es = 650N kPa [Yoshida and Yoshinaka (1972)]

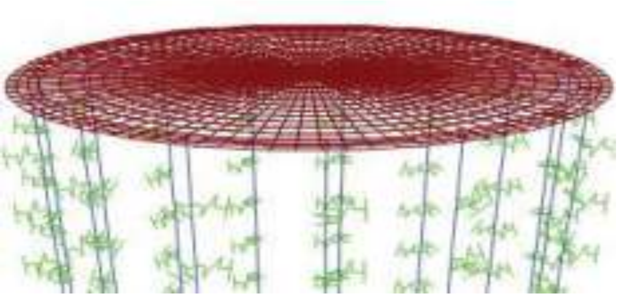

Figure 5:Structural FEM Analysis Model

2.2.2 Equivalent load at shell elements at Anchor Cage

Equivalent load at shell elements due to relevant tower bottom moments, shear forces and reactions are transformed to the forces at shells at anchorage level & locations distributed based on the pair of 92 nos. of Anchor Cage and applied for the analysis.

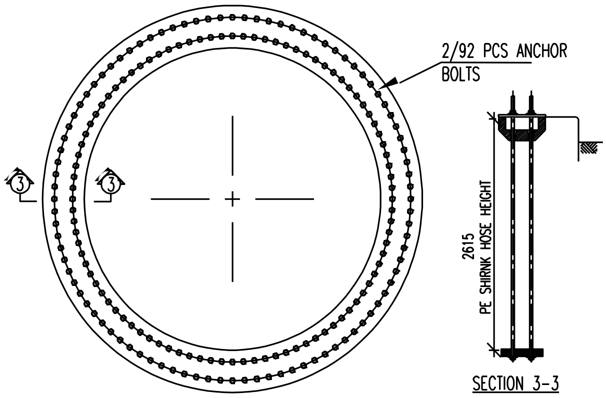

Figure 6: Details of Anchor Cage

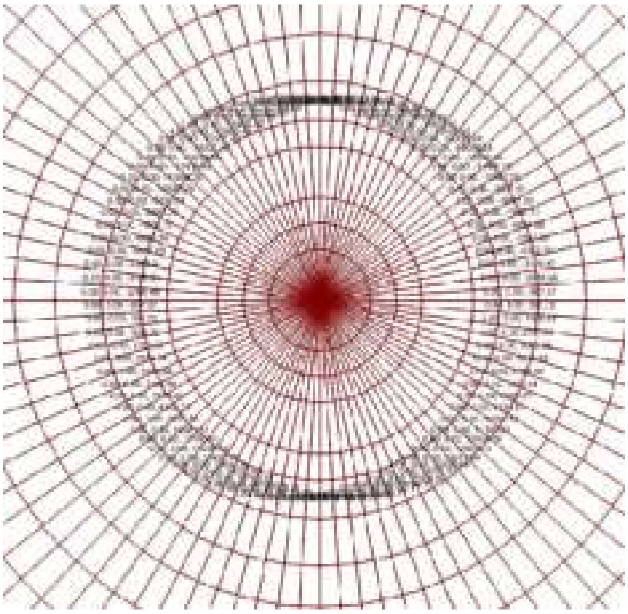

Figure 7: Sample Load input to FEM Model

2.2.3 Design Effects

Design Effects in radial circumferential directions to fluctuating loads are obtain from the finite element analysis various load cases for the evolutions of fatigue design situation.

Relevant design effects are listed under the relevant sections in this report.

2.3 Design for flexure

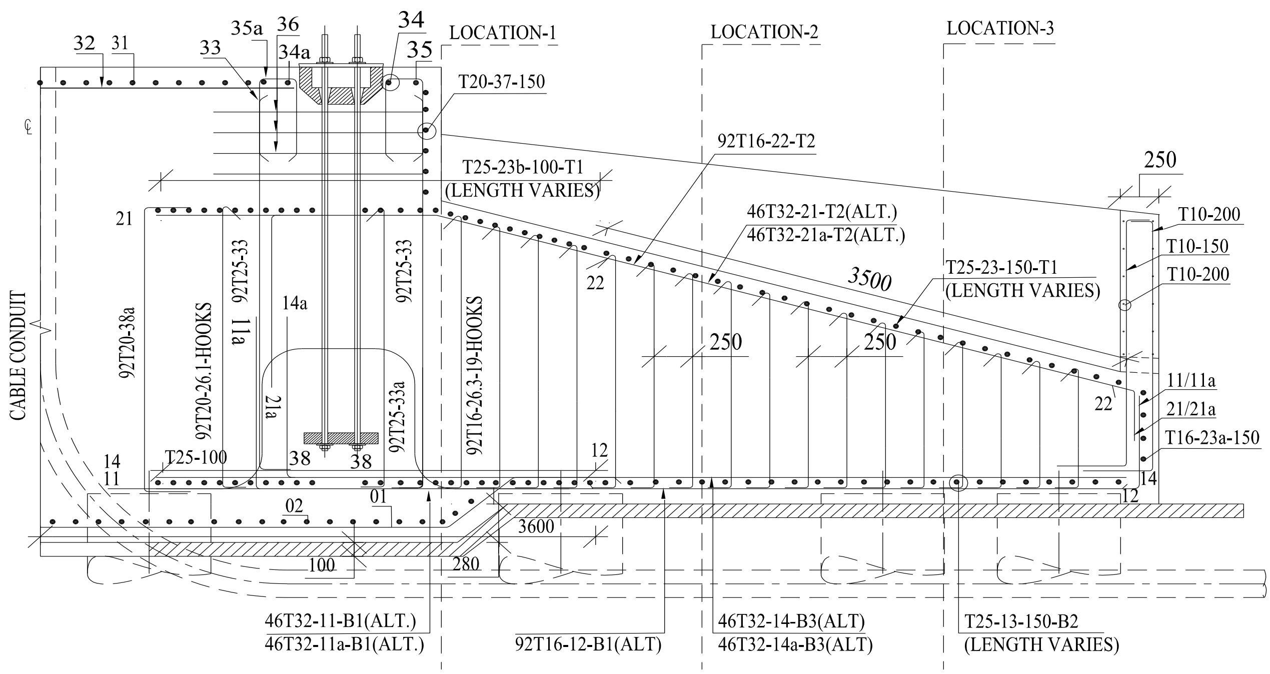

Figure 8:Portion of WTG Pile Cap reinforcement arrangement

In the case of designing for flexure in wind turbine structures, the maximum bending moment and shear forces are obtained based on the extreme loads provided by the wind turbine supplier.

The wind turbine supplier typically provides the design loads, including the maximum wind speed, turbulence intensity, and other relevant parameters. These loads are used to determine the maximum bending moment and shear forces acting on the structure.

Once these extreme loads are known, the design process for flexure can proceed by analyzing the structure and calculating the required reinforcement in the radial and circumferential directions, as per the guidelines specified in EN 1992-1.

2.4 Fatigue Verification of Concrete and Reinforcement (EN 1992-1-1;6.8)

The resistance of concrete and Reinforcement steel is verified to the effect of tower bottom fatigue load provided by the Wind Turbine mast Analysis.

In this wind turbine foundations, fatigue verifications have been performed to study how materials handle cyclic loads, including both tensile stress variation at reinforcements and compressive stresses variation at concrete due to the variation of bending moments under fluctuating load conditions.

2.4.1 Combination of actions (EN 1992-1-1; 6.8.3)

$$\left[\sum_{\geq 1} G_{k,j} \text{"+ } P \text{" + "} \psi_{1,1} Q_{k,1} \text{" + "} \sum_{\geq 1} \psi_{2,i} Q_{k,i}\right] \text{" + "} Q_{fat}$$

Equation 6.69

where:
Q_{fat} - the fatigue load
$Q_{k,1}$ and $Q_{k,i}$ are non-cyclic, non-permanent actions

2.4.2 Fatigue Verification of Reinforcement

2.4.2.1 Fatigue Verification to Top Radial Reinforcement @ Location 1

for i=1; $M_{Ed,\,max,\,i}$ = 1761 kNm, $M_{Ed,\,min,\,i}$= -3069kNm

2.4.2.2 Top Reinforcement Provided for ULS

Total Top Radial Reinforcement provided**;** `
92 T-32-T1; AsT_1 = 4529 mm²/m

Total Top Ring Reinforcement provided;
T25-150 -T2; AsT_2 = 3272 mm²/m

2.4.2.3 Calculations of the stress variation at the reinforcement

Characteristic Strength of Concrete, f_{ck} = 30 N/mm²
The partial factor for concrete, γ_c = 1.5, α_{cc} = 0.85
Design Value of concrete, $f_{cd} = \alpha_{cc}.f_{ck} / \gamma_c$ = 17 N/mm²
Char. Strength of Reinforcement, f_{yk} =500N/mm²
h = 2285 mm, Effective depth, d = 2165 mm

Stress at top radial reinforcing @ Location 1 due to Maximum fatigue moment for Case i=1: $\sigma_{Ed,max}$ = 0 N/mm²(M > 0)

$k = M/bd^2f_{ck}$ = 0.022, δ = 1 ((0.7<δ<1)
k' = 0.60δ-0.18δ2-0.21 = 0.21, (k < k'); No. Com. RF
z = 2056 mm, x = 271 mm
$\sigma_s = M_{Ed}/As.z$ (when k < k')
$\sigma_{Ed,min}$ = 330 N/mm²
Hence; $\Delta\sigma_s$ = 330 N/mm²

Table 6: Range of Stress Variation of Top Radial Reinforcement @ Location 1 with Load cases

Load Case	$M_{Ed,max}$ [kNm]	$M_{Ed,min}$ [kNm]	Δσs [N/mm²]	Frequency
1	1761	-3069	330	1.0666E+0
2	1664	-2973	319	1.0348E+0
3	1568	-2876	308	1.4269E+1
4	1471	-2779	298	9.2168E+0
5	1374	-2683	288	2.7780E+1
6	1278	-2586	277	5.4338E+1
7	1181	-2490	267	4.0096E+1
8	1085	-2393	256	6.2208E+1
9	988	-2296	246	5.1348E+1
10	891	-2200	236	8.0019E+1
11	795	-2103	225	1.5287E+2
12	698	-2007	215	2.6823E+2
13	602	-1910	205	1.7883E+3
14	505	-1813	194	3.0230E+3
15	408	-1717	184	1.4945E+4
16	312	-1620	173	4.2998E+4
17	215	-1524	163	8.0035E+4
18	119	-1427	153	1.6287E+5
19	22	-1330	142	2.5225E+5
20	75	-1234	124	4.6495E+5
21	-171	-1137	103	1.1080E+6
22	-268	-1041	82	2.6350E+6
23	-944	-944	51	7.7638E+6
24	-461	-847	41	3.5548E+7
25	-558	-751	20	5.5989E+8

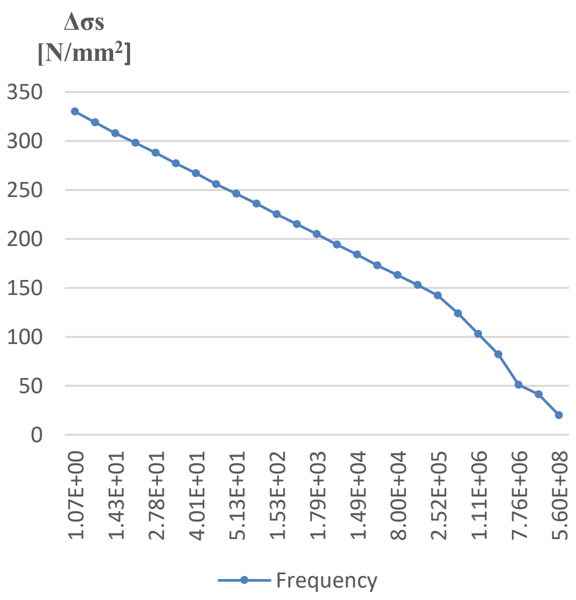

Figure 9: Range of stress Variation in top radial Reinforcements and its frequency of occurrence

2.4.2.4 Verification procedure for reinforcing steel (EN 1992-1-1; 6.8.4)

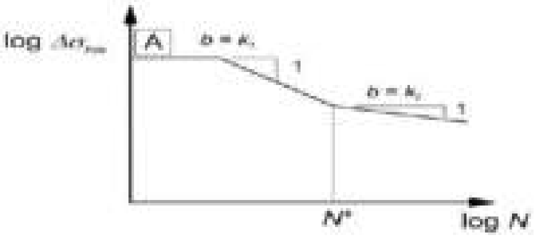

Figure 10: Shape of the characteristic fatigue strength curve (S-N-curves for reinforcing and prestressing steel)

The damage of a single stress amplitude $\Delta\sigma$ was determined by using the corresponding S-N curves (Figure 6.30) for reinforcing steel.

$\Delta\sigma_{Rsk}$ - The resisting stress range at N* cycles
A = Reinforcement at Yield (500N/mm^2)
N- No. of cycles

2.4.2.5 Parameters for S-N curves for reinforcing steel for straight bars (Table 6.3N)

Table 7: *Parameters for S-N curves*

Type of reinforcement	N*	stress exponent		ΔσRsk (MPa) at N* cycles
		k1	K2	
Straight and bent bars[1]	10^6	5	9	162.5
Note 1: Values for $\Delta\sigma_{Rsk}$ are those for straight bars. Values for bent bars should be obtained using a reduction factor $\zeta = 0{,}35 + 0{,}026\ D/\varphi$. where: D diameter of the mandrel φ bar diameter				

$N(\Delta\sigma i) = N^*(\Delta\sigma_{Rsk}/\Delta\sigma i)^5$ for $\Delta\sigma i \geq \Delta\sigma_{Rsk}$
$N(\Delta\sigma i) = N^*(\Delta\sigma_{Rsk}/\Delta\sigma i)^9$ for $\Delta\sigma i \leq \Delta\sigma_{Rsk}$

2.4.2.6 Application of Palmgren-Miner Rule for multiple cycles with variable amplitudes

"Palmgren-Miner Rule" is applied to assess the cumulative damage caused by repeated stress cycles on a material or structure. It states that failure occurs when the cumulative damage, represented by the sum of the ratio of applied stress cycles (n) to the endurance limit (N) for each stress cycle, reaches or exceeds 1.

The process of determining applied stress cycles (n) involves identifying stress cycles, normalizing stress amplitudes, and calculating the equivalent number of cycles that cause similar fatigue damage. These crucial steps provide valuable insights into the fatigue life of the component and help assess potential failure risks. Typically, these data are obtained from wind turbine superstructure analysis.

For multiple cycles with variable amplitudes, the damage is added by using the Palmgren-Miner Rule. Hence, the fatigue damage factor D_{Ed} of steel caused by the relevant fatigue loads should satisfy the condition:

$$\sum_i \frac{n(\Delta\sigma i)}{N(\Delta\sigma i)} < 1:$$

$n(\Delta\sigma_i)$: applied number of cycles for a stress range $\Delta\sigma_i$
$N(\Delta\sigma_i)$: resisting number of cycles for a stress range $\Delta\sigma_i$

For i = 1;
$\Delta\sigma_s = 330$ N/mm^2
$n(\Delta\sigma i) = 1.0666$
$N(\Delta\sigma i) = 2.91E{+}04$

Repeat the above step for all the load cases listed above (for i= 1, 2..25) and for top reinforcement for Location 1 as summarized in the table below.

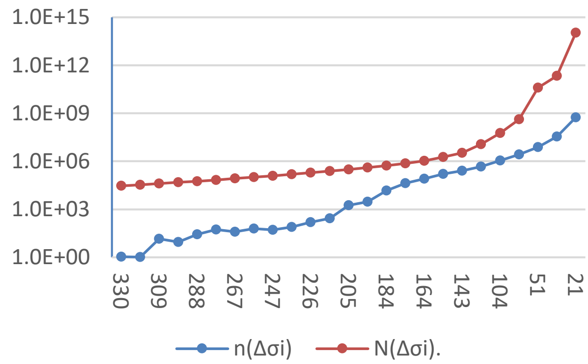

Figure 11: Applied & Resisting Number of cycles with range of variation of stress at top radial reinforcements at Location 1

Table 8:Fatigue damage factor of steel caused by the relevant fatigue loads

Load Case	$\Delta\sigma_{s,max}$ N/mm^2	frequency		$\frac{n(\Delta\sigma i)}{N(\Delta\sigma i)}$
		n(Δσi)	N(Δσi)	
1	330	1.07E+0	2.91E+04	3.66E-5
2	319	1.03E+0	3.42E+04	3.03E-5
3	309	1.43E+1	4.03E+04	3.54E-4
4	298	9.22E+0	4.79E+04	1.93E-4
5	288	2.78E+1	5.71E+04	4.86E-4
6	278	5.43E+1	6.86E+04	7.92E-4
7	267	4.01E+1	8.30E+04	4.83E-4
8	257	6.22E+1	1.01E+05	6.15E-4
9	247	5.13E+1	1.24E+05	4.13E-4
10	236	8.00E+1	1.54E+05	5.19E-4
11	226	1.53E+2	1.93E+05	7.93E-4
12	215	2.68E+2	2.44E+05	1.10E-3
13	205	1.79E+3	3.12E+05	5.73E-3
14	195	3.02E+3	4.05E+05	7.47E-3
15	184	1.49E+4	5.32E+05	2.81E-2
16	174	4.30E+4	7.11E+05	6.05E-2
17	164	8.00E+4	9.67E+05	8.28E-2
18	153	1.63E+5	1.70E+06	9.60E-2
19	143	2.52E+5	3.19E+06	7.91E-2
20	124	4.65E+5	1.10E+07	4.22E-2
21	104	1.11E+6	5.69E+07	1.95E-2
22	83	2.64E+6	4.24E+08	6.22E-3
23	51	7.76E+6	4.05E+10	1.92E-4
24	41	3.55E+7	2.17E+11	1.64E-4
25	21	5.60E+8	1.11E+14	5.04E-6

$$D_{Ed} = \sum \frac{n(\Delta\sigma i)}{N(\Delta\sigma i)} = 0.4 < 1$$

Since the $D_{Ed} = \sum \frac{n(\Delta\sigma i)}{N(\Delta\sigma i)} = 0.4 < 1$ the fatigue Verification is satisfactory for the top Radial Reinforcements at Location 1

2.4.3 Fatigue verification of concrete under compression (6.8.7; EN 1992-2:2005)

Miner's rule should be applied for the verification of concrete; $\sum_{i}^{m} \frac{n_i}{N_i}$ Eq 105 where:

m = number of intervals with constant amplitude
n_i = actual number of constant amplitude cycles in interval "i"

N_i = ultimate number of constant amplitude cycles in interval "i" that can be carried before failure. N_i may be given by National Authorities (S-N curves) or calculated on a simplified basis using Expression 6.72 of EN 1992-1-1 substituting the coefficient 0,43 with ($\log N_i$)/14 and transforming the inequality in the expression.

$$N_i = 10 \exp\left[14\left(1 - \frac{E_{cd,max,i}}{\sqrt{1-R_i}}\right)\right] \text{ Eq 6.106}$$

$$R_i = \frac{E_{cd,min,i}}{E_{cd,max,i}} \text{ Eq 6.107}$$

$$E_{cd,min,i} = \frac{\sigma_{cd,min,i}}{f_{cd,fat}} \text{ Eq 6.108}$$

$$E_{cd,max,i} = \frac{\sigma_{cd,max,i}}{f_{cd,fat}} \text{ Eq 6.109}$$

Where;
R_i is the Stress ratio
$E_{cd,min,i}$ is the minimum compressive stress level
$E_{cd,max,i}$ is the maximum compressive stress level
$f_{cd,fat}$ is the design fatigue strength of concrete according to (EN 1992-1-1:Cl 6.76)

$\sigma_{cd,max,i}$ is the upper stress in a cycle
$\sigma_{cd,min,i}$ is the lower stress in a cycle

$$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd}\left[1 - \frac{f_{ck}}{250}\right] \text{ Eq 6.76}$$

where:
$\beta_{cc}(t_0)$ is a coefficient for concrete strength at first load application (see 3.1.2 (6) of EN 1992-1-1)
t_0 is the time of the start of the cyclic loading on concrete in days
k_1=0.85

2.4.3 .1 Calculation for Top Concrete -Radial Moment (Location 1)

For i = 1
$M_{Ed,\,max}$= 1761 kNm
$M_{Ed,\,min}$= -3069 kNm
Maximum stress on top concrete;

$$\sigma_{cd,max} = \frac{M}{\eta b_w \lambda z} = \frac{1761E6}{0.85x1000x271x2056}$$

$$\sigma_{cd,max} = 3.95 \frac{N}{mm^2}$$

Minimum stress on top concrete; $\sigma_{cd,min} = 0 \frac{N}{mm^2}$ ($M_{Ed,min}$ <0)

f_{ck} = 30 N/mm^2

γ_c = 1.5, α_{cc} = 0.85$f_{cd} = \alpha_{cc}\frac{f_{ck}}{\gamma_c}$

$f_{cd} = 0.85x\frac{30}{1.5} = 17 \frac{N}{mm^2}$, k1 = 0.85

$$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd}\left[1 - \frac{f_{ck}}{250}\right]$$

$$f_{cd,fat} = 0.85x\beta_{cc}(t_0)17\left[1 - \frac{30}{250}\right]$$

$$f_{cd,fat} = 12.7\ N/mm^2$$

$$E_{cd,max,i} = \frac{\sigma_{cd,max,i}}{f_{cd,fat}} \text{ Eq 6.109}$$

$$E_{cd,max,1} = \frac{3.95}{12.7} = 0.31$$

$$E_{cd,min,i} = \frac{\sigma_{cd,min,i}}{f_{cd,fat}} \text{ Eq 6.108}$$

$$E_{cd,min,1} = \frac{0}{12.7} = 0$$

$$R_i = \frac{E_{cd,min,i}}{E_{cd,max,i}} \text{ Eq 6.107}$$

$$R_i = \frac{0}{0.31} = 0$$

$$N_i = 10 \exp\left[14\left(1 - \frac{E_{cd,max,i}}{\sqrt{1-R_i}}\right)\right] \text{ Eq 6.106}$$

$$N_i = 10 \exp\left[14\left(1 - \frac{0.31}{\sqrt{1-0}}\right)\right] = 155378$$

n_i = 1.067; n_i/Ni = 1.067/155378 =6.86E-0.6

Repeat the above steps for all the load cases (for i = 1, 2, 25) and results are tabulated as shown below in the table 9.

Table 9: Fatigue damage factor of Concrete caused by the relevant fatigue loads

Fatigue Load Case	$\sigma_{cd,max,i}$	$\sigma_{cd,min,i}$	**Frequency**		**n_i/N_i**
	N/mm^2		**n_i**	**N_i**	
1	3.95	0	1.07E+0	1.55E+5	6.86E-6
2	3.73	0	1.04E+0	1.97E+5	5.25E-6
3	3.52	0	1.43E+1	2.50E+5	5.70E-5
4	3.30	0	9.22E+0	3.18E+5	2.90E-5
5	3.08	0	2.78E+1	4.04E+5	6.88E-5
6	2.87	0	5.43E+1	5.12E+5	1.06E-4
7	2.65	0	4.01E+1	6.50E+5	6.17E-5
8	2.43	0	6.22E+1	8.26E+5	7.54E-5
9	2.22	0	5.14E+1	1.05E+6	4.90E-5
10	2.00	0	8.00E+1	1.33E+6	6.01E-5
11	1.78	0	1.53E+2	1.69E+6	9.05E-5
12	1.57	0	2.68E+2	2.14E+6	1.25E-4
13	1.35	0	1.79E+3	2.72E+6	6.57E-4
14	1.13	0	3.02E+3	3.46E+6	8.75E-4
15	0.92	0	1.50E+4	4.39E+6	3.41E-3
16	0.70	0	4.30E+4	5.57E+6	7.72E-3
17	0.48	0	8.00E+4	7.07E+6	1.13E-2
18	0.27	0	1.63E+5	8.97E+6	1.81E-2
19	0.05	0	2.52E+5	1.14E+7	2.21E-2
20	0	0	4.65E+5	-	-
21	0	0	1.11E+6	-	-
22	0	0	2.64E+6	-	-
23	0	0	7.76E+6	-	-
24	0	0	3.55E+7	-	-
25	0	0	5.60E+8	-	-

$$\sum_{i=1}^{m} \frac{n_i}{N_i} = 0.07 < 1$$

Hence the fatigue verifications for top Concrete under compression at Location 1, Radial Moment is satisfactory.

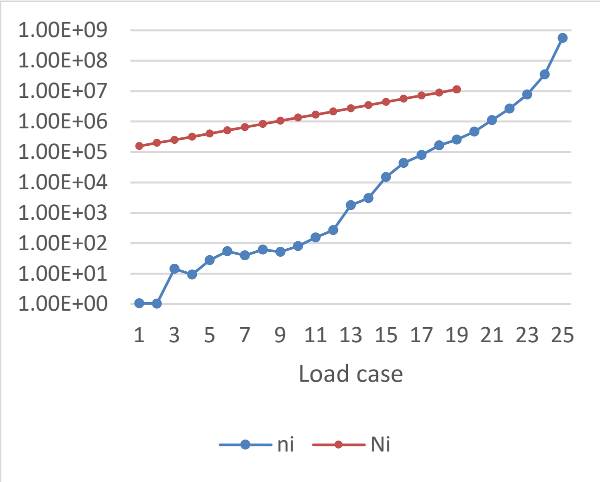

Figure 12: Actual & ultimate number of constant amplitude cycles with Fatigue Load cases at Location 1

Hence the fatigue verifications for Pile Cap Concrete under compression is satisfactory

3. Results and Discussion

3.1 Summary of Fatigue verifications to Reinforcement

For Locations 1,2 and 3 along the Pile Cap, Top and Bottom Reinforcement in Radial and Ring directions under fatigue is calculated and outcomes are tabulated as shown below in the table 10

Table 10:Summary of Fatigue Damage to the Reinforcement

Location	**Top/Bot**	**Radial/Ring Reinforcement**	$\sum \frac{n(\Delta\sigma i)}{N(\Delta\sigma i)}$	**Remarks**
Location 1	Bot	Radial	0.004	< 1
Location 1	Top	Radial	0.4	< 1
Location 1	Bot	Ring	0.01	< 1
Location 1	Top	Ring	0.0005	< 1
Location 2	Bot	Radial	0.003	< 1
Location 2	Top	Radial	0.0005	< 1
Location 2	Bot	Ring	0.0002	< 1
Location 2	Top	Ring	0.0001	< 1
Location 3	Bot	Radial	0.0002	< 1
Location 3	Top	Radial	0.1	< 1
Location 3	Bot	Ring	0.0001	< 1
Location 3	Top	Ring	0.0001	< 1

3.2 Summary of Fatigue verification of Concrete

For Location 1,2 and 3 for Top and Bottom concrete for fatigue damage is calculated and verified the concrete resistance to fatigue as tabulated below in the table 11.

Table 11: Summary of Fatigue Damage to Concrete

Location	**Top/Bot**	**Radial/Ring Moment**	$\sum \frac{ni}{Ni}$	**Remarks**
Location 1	Bot	Radial	0	< 1
Location 1	Top	Radial	0.07	< 1
Location 1	Bot	Ring	0	< 1
Location 1	Top	Ring	0.07	< 1
Location 2	Bot	Radial	0	< 1
Location 2	Top	Radial	0.07	< 1
Location 2	Bot	Ring	0	< 1
Location 2	Top	Ring	0.06	< 1
Location 3	Bot	Radial	0	< 1
Location 3	Top	Radial	0.06	< 1
Location 3	Bot	Ring	0	< 1
Location 3	Top	Ring	0.06	< 1

4. Conclusions

The Wind Turbine Foundation's fatigue verifications for radial and ring reinforcements at Location 1, Location 2, and Location 3, have been deemed satisfactory. These results are detailed in the "Results and Discussion" chapter.

Similarly, the fatigue verifications for pile cap concrete at the aforementioned locations under fatigue load have also been found satisfactory, as outlined in the same section.

Considering both reinforcements and concrete meet the fatigue load criteria, it can be concluded that the Wind Turbine Foundation is deemed safe under fatigue load conditions for its intended design working life of 35 years.

Supporting Documents

The paper is an excerpt from the main report titled "Structural Design Calculations for proposed Wind Turbine Foundations-Pile Cap-Mannar Wind Power Project-Phase 1 of Construction of 100 MW Semi Dispatchable Wind Farm with Associated facilities in Mannar Island of Sri Lanka," which was prepared by the Design Division of Access Engineering PLC under the BoP Subcontractor for the project.

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