PROCEEDINGS

NATIONAL CONFERENCE ON

"CIVIL & STRUCTURAL ENGINEERING PRACTICES IN POWER SECTOR AND INDUSTRIES"

AT FEDERATION OF GUJARAT INDUSTRIES (FGI), GOTRI, VADODARA, DATED 7 & 8 DEC 2023

Jointly Organized By:

THE SOCIETY OF POWER ENGINEERS (I)
VADODARA CHAPTER

CENTRAL BOARD OF IRRIGATION & POWER,
NEW DELHI

INSTITUTION OF ENGINEERS (INDIA)
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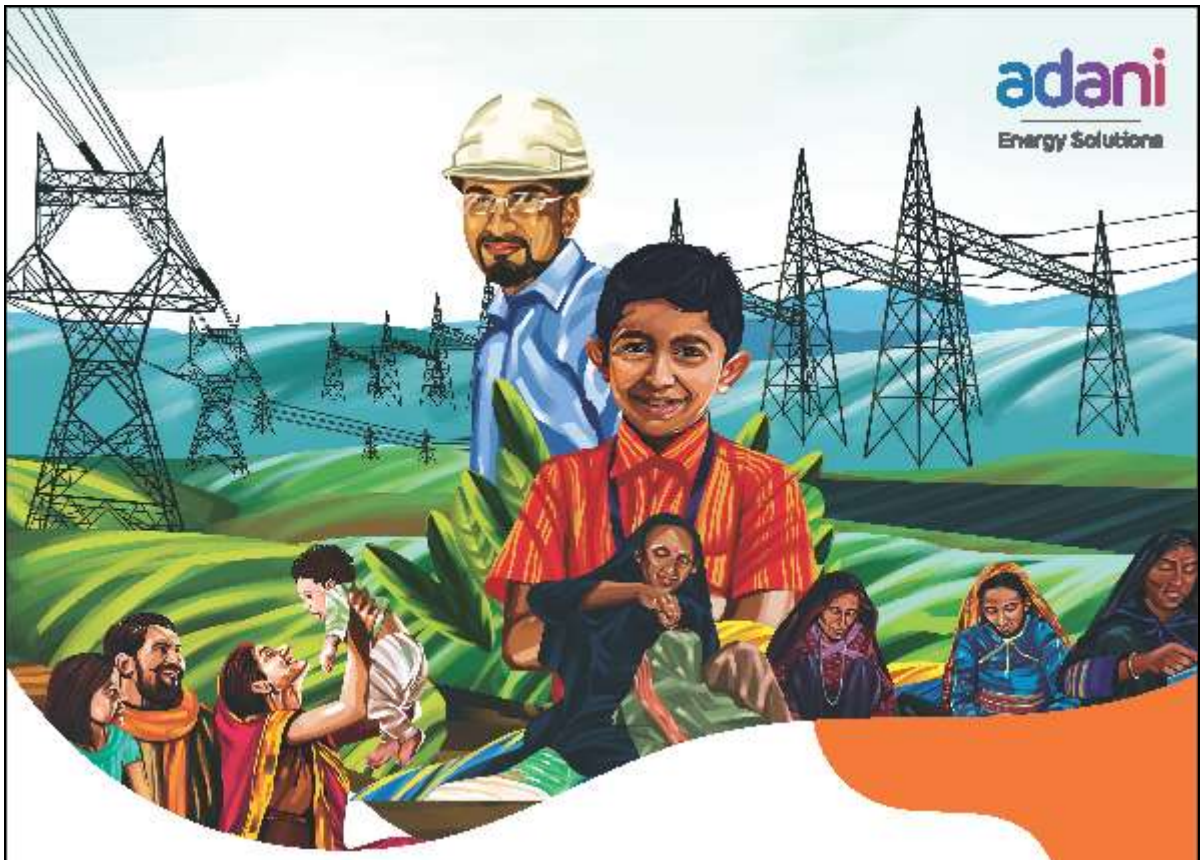

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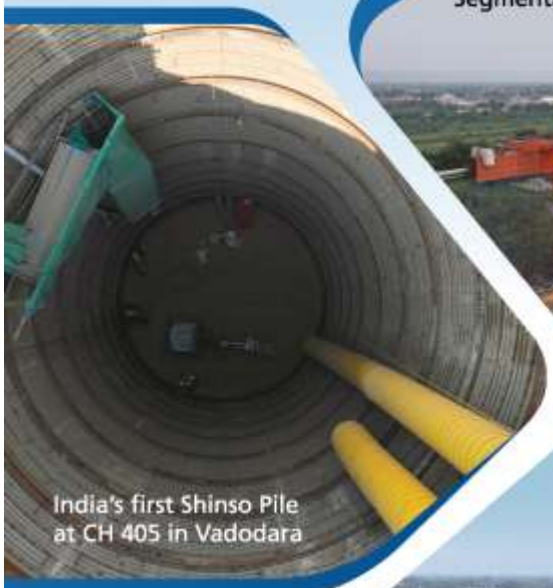

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Dated 1st December 2023

I am delighted that the Vadodara Chapter of The Society of Power Engineers (India) has organised a 2-Day Conference on the topic of "CIVIL & STRUCTURAL ENGINEERING PRACTICES IN POWER SYSTEM AND INDUSTRIES", on 7th & 8th Dec 2023, at Vadodara.

The conference is being organised in association with Central Board of Irrigation and Power (CBIP) New Delhi. I appreciate the topic of the conference which has never been touched upon by any utility or organisation.

The Society of Power Engineers (I) was established in the year 1945 in New Delhi with Chapters in other parts of the country. I am happy to note that the Vadodara Chapter is the most active amongst all the Chapters in terms of membership and consistency in the activities.

In the recent past, I have been a witness to a conference organised by Vadodara Chapter. I was overwhelmed by the manner in which the members of the chapter exhibited cohesion and made the event most successful.

I understand that large numbers of delegates are participating in this conference. I am sure the discussion and deliberation during the conference will be beneficial to all the participants.

My best wishes for the success of the conference.

Aditya K Dinkar
Secretary, CBIP

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FOREWORD

I am very happy to present the proceedings for the **2-Day Conference on "Civil & Structural Engineering practices in Power Sector and Industries**" organized by **The Society of Power Engineers(I), Vadodara**. The papers included herein present varieties of topics which are relevant to the present day practices in Civil and Structural Engineering all over the country and many parts of the world.

It is heartening to note that a conference on such a topic is being organized for the first time in the country. I also understand that participants are hailing from utilities, industries and academic institutions. I would like to appreciate here the efforts put in by each of the authors whose paper is included in this proceeding, which reflects the experience gained by them.

Civil and structural Engineering is the most basic engineering for any power or industrial installation. The papers presented herein are trying to project the requirements of interface between Civil / Structural Engineering with other disciplines such as Electrical, Mechanical, Chemical, Metallurgy, etc.

The nondestructive testing and modern construction practices are projected very nicely in different papers included in these proceedings.

I am sure the proceedings will be useful to the practicing engineers and academia.

I hope the discussions and deliberations during the conference will pave a way for furthering the cause of Civil and Structural Engineering in Power and Industrial sectors.

I would like to put on record the support given by the utilities, industries, consultants and individuals in making the conference successful.

Best wishes to all the delegates and the authors**.**

Er. Mohan R Tilwalli
Chairman
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MESSAGE FROM PATRON MEMBER

It gives me a great pleasure to write words of appreciation for the committee members of **The Society of Power Engineers (I) Vadodara Chapter** for organizing such a unique conference on the topic of Civil and Structural Engineering related to Power and Industries.

Civil Engineering is the oldest discipline of Engineering and no power or industrial project can come up without the structural & civil engineering designs. I am very happy to note that large numbers of delegates are participating in this conference from various parts of the country.

I have been part of the organizing committee of the conference and have found that the committee members have not left any stone unturned to see that the conference is organized in the best possible manner.

The conference was planned about 4 months ago and the planning is very meticulous. I am confident that the participants will enjoy the technical part of the conference & hospitality extended to them.

I wish the conference all the best.

Er. P H Rana
Patron Member
SPE (I) Vaodara Chapter
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ABOUT THE CONFERENCE

This Conference is being organized to upgrade the knowledge of the participants regarding the involvement of Civil & Structural Engineering Practices in Power Generation, Transmission and Distribution as well as Industries. The aim of the Conference is also to improve the Civil & Structural engineering knowhow of Practicing Engineers from the industry and various disciplines of Power Sector as well as academia in the country. Civil & Structural Engineering is the oldest Engineering discipline in the World. Cost wise Civil & Structural Engineering always have a lion's share in Thermal, Hydro, Gas & Wind Power Stations and major industries.

Similarly Transmission lines and Substations also need a lot of Civil & Structural Engineering. To be fair, it is not possible to create a power related establishment without an active involvement of civil & structural engineering. Same is the case with Industry. The industrial establishment of any engineering discipline cannot come up without civil & structural engineering. In addition, industrial civil & structural engineering varies with the type of industry.

However, for one or the other reason, civil & structural engineering is not able to get its due recognition once the power unit or industry comes up. To the engineers of other disciplines civil and structural engineering means the engineering related to the super structure and foundation. There are many other branches of civil engineering which provide a backup for this engineering. They include soil mechanics, concrete technology, nondestructive testing, retrofitting, material testing, etc.

SPE (I) Vadodara Chapter have organized many conferences in the last 27 years on various topics of the interest of electrical and mechanical engineers. However, after the chapter completed 25 years, the office bearers of the chapter deemed it fit to organize conferences on untapped topics. As a part of this diversification, the first conference was organized on Power Reforms in 2021 and followed by a conference on Metallurgy in Power sector in 2022.

The Civil engineering being most common to power and industry has been selected as a subject by the office bearers of the chapter.

I am happy to inform you that there has been an overwhelming response to the conference. More than 175 delegates from different parts of the country are participating in the conference. Many industries and business houses have come forward to lend a helping hand to the organizers and have generously contributed their might. The organizing committee is indebted to them.

The office bearers of the chapter also deemed it fit to recognize the contribution of some veteran civil and structure engineers and felicitate them during the conference.

The papers included for presentation in the conference cover varieties of subjects related to design, engineering, soil mechanics, nondestructive testing, mining infrastructure and power station civil works. The organizing committee of the conference appreciates the efforts put in by the authors for their contributions.

I am sure all the delegates, authors and dignitaries will enjoy the conference.

Er. SM Takalkar
Conference Convener
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PERSPECTIVE OF UTILITIES FOR CIVIL CONSTRUCTION IN POWER PLANTS

Er. Kailash C Yadav — Chief Engineer Civil (Rtd.)
Er. Jagdish D Darji — Superintending Engineer Civil (Rtd.)
Gujarat State Electricity Corporation Ltd. Vadodara

1.0 INTRODUCTION: –

1.1 Construction of Power Plant involves lot of civil work. To complete the project as per schedule the EPC contract has to be awarded in time.

1.2 Before preparing the EPC contract documents, much spade work has to be done by the engineers of the utility. This work leads to better estimation of tender work.

1.3 In the construction of power station civil engineering has a lion's share. Besides, construction and/or performance of mechanical & electrical equipment depends upon the accuracy and quality of civil work.

1.4 The presentation here under gives brief account of pre-bid work & project monitoring work to be done by the engineers of the utility, for successful and timely completion of project.

2.0 SITE SELECTION: -

2.1 Selection of a site for the project is very important from various perspectives. This is particularly true for the overall requirement of land for the project and also for future requirements. The land requirement may include main project site, offices, fuel storage, material stores, cooling system, switchyard, residential colony ash disposal system, silo, ash dyke and welfare establishment. The following are the few considerations for selection of site.

- The project location should be nearer & easily approachable from the State/National highway, Railway Station as well as from Airport.
- Availability of local raw material from nearby areas required for construction of civil works of the project. This may include cement, steel, sand, stones, shuttering, scaffolding etc.
- Reliable source of construction (Potable) water for use in construction activities and also during operation of the plant may be from River, Canal, Jack/French well etc. in sufficient quantity.
- Nature of land /Soil type to be suitable for construction work to an extent possible. Selection of good fertile agriculture land for the project should be avoided. Govt. waste land or Gochar land can be preferred.
- The site should be free from unnecessary interference of local population during construction & operation of plant/industry.
- Availability of skilled, semiskilled and unskilled laborers in the vicinity.
- Acquisition of forest land should be avoided as far as possible.
- Land having high water table or one with risk of flood should be avoided.

3.0 TENDERING PROCESS: -

3.1 For high value projects, it is usual now to call for Expressions of Interest (EOI). This is done to weed out incompetent bidders and thus reduce the time of evaluation of tenders. The bidders who qualify in EOI are to be only allowed to submit their bid.

3.2 In order to provide a levelled field to all the bidders and also to ensure hassle free execution of civil work, the following points are very important.

- On getting approval of project, an experienced consultant shall have to be appointed and for preparation of TENDER DOCUMENT experienced civil engineers of utility are to be involved in coordination with the project consultant. The skilled, knowledgeable & experienced civil & structural engineers should be engaged in construction of plant.
- Finalize the agency for detailed preliminary survey including contouring of area and geo-technical investigation of main area. The soil investigation report shall be prepared to

determine the soil classification, soil bearing capacity for important structure, water table, climatic condition, topography of land proposed for construction of project etc. Type of foundation open/pile suitable to soil may be finalized for preparing the tender documents.

- Fundamental Principles for drafting the scope of work in tender documents:– The scope of work shall be properly framed for all the items which are necessary in successful completion of project. To work out the item wise quantities minutely & define in tender documents based on previous record of site (in case if extension). If any specific items or works are required, it should be added to avoid future complication & unnecessary delay in approval procedure during execution of the work at site as also to avoid disputes.
- The consultant of the project shall design all-civil structures based on available super structure loading. While releasing the final drawings for construction it should be cross verified by the engineers of utility, with mechanical & electrical drawings to avoid re-work in civil. This particularly relates to embedment & fittings.
- To encourage use of locally available material and reduce price impact the bidders may be allowed to use such material subject to all the required acceptance tests (as per IS) on each ingredient.
- For ensuring realistic & competitive rates the price schedule of all items quantity wise should be prepared & attached separately with +/- percentage variation during actual execution for reference to the agency.
- The time schedule for completion of project to be specified clearly, considering all aspects such as design duration, finalization of other related contracts & their execution in sequential manner. For completion of entire project, keep provision for extension of time limit. At the time of contract finalization of agency, the delay attributed to contractor/consultant/clients can be decided based on the record.
- Earthquake resistant design as per seismic zone of the project site shall be specified in tender document.
- Terms of payment, in the EPC contract should be specifically mentioned in tender documents. The release of payments to the agency must be proportionate to the actual progress of work in monthly R A Bills. This should be specifically brought out in the tender documents. Payment of advance and statutory taxes & duties shall be clearly spelled out.
- The item of strutting & shoring for open deeper foundation & dewatering the provision of coffer dam should be included in the tender. There should not be any separate item on unit rate of consumption of electricity or on HP hour of pump for dewatering. In case if it is excluded looking to site requirement and considered separate item of dewatering, then payment terms should be on pro-rata to amount of work done.
- The items of road with drainage, compound wall with entrance gate, fencing etc. are to be included in the tender documents.
- Item for painting, false ceiling concealed wiring & Furniture for all the building, labelling shall be included in the contract.
- The item for water proofing in the RCC slab and water retaining structure to be added in the tender documents. Item of termite treatment shall be part of the scope & price bid.
- The battery limit for the various terminal points like sweet water, brackish water, storm water drain, sewage drain connection ash disposal pipeline etc. shall be specified as per site condition and to be included in EPC contract document.
- The sub agency of major civil works in EPC contract should be capable, reliable & experienced and also the past background of civil sub agencies should be verified before awarding the contract. Benchmark for sub-contracting can be included in the tender.
- As per the norms of environmental compliance, it is necessary to have a green belt. This should be included in specification. Besides for aesthetic look, horticulture needs to be included as one of the concluding activities.

- Basic facilities like Canteen and parking also to be proposed & include in the tender documents.
- The general terms and condition of contract should cover important clauses such as labor laws, arbitration, force majeure, performance security, retention of amount from the bill, contract documentation, applicable taxes & duties, penalty, liquidated damages, termination of contract, extension of time limit etc.

4.0 Preparation of technical specification of items

4.1 Preparation of specifications for various civil & structural construction items needs a lot of precision. The experience of the engineer drafting the specification matters much above other aspects. The terms & conditions of specification need to be balanced. It should be ensured that it takes care of the interest of the utility but simultaneously promotes competition and allows a reasonable profit to the bidder/contractor. The commercial terms & conditions should have room for unavoidable eventualities during the execution of the entire project work. The technical terms & conditions prescribed in the tender should be unbiased and should afford a level playing field to the bidders. The local factors affecting the technical & commercial terms & condition should be taken into account.

4.2 There are large number of perspectives of drafting the specifications but the following few are covered which are out of the experience of the authors for more than 3 decades.

- Provision of strutting and shoring for deeper foundation shall be included with design of strutting and shoring to safeguard adjoining running structures (Case of EPC contract of Extension unit Power Station wanakbori) thermal power station.
- Provision of minimum extra width of excavation at top and bottom in case of open foundation & adequate side slope considering the soil investigation report of project site. (Case of Power station main foundation work at Gandhinagar unit 3)
- In the case of extension of powerhouse with existing unit, provision of dismantling of underground structure and rerouting/diversion of running structure, etc. shall be included. (Case of all extension unit for P H construction)
- Provision of lead and lift (specify crow-fly distance and depth) for excavation in all types of soil and disposal of excavated extra earth, materials. Also, for back filling on completion of foundation, utilization of excavated earth for back filling or, to dispose of outside in stockpile area or to be spread in layer as per requirement of site, if any, shall be specified.
- All the water retaining structure must have condition of hydro test to ensure water tightness. There should be specific conditions of epoxy grouting with low viscosity material.
- Detailed working drawings shall be given to site duly scrutinized by consultant covering all the items required to be executed as per contract specification, to avoid mistake during the work execution at site. The consultant has to submit precise estimation of all the items to avoid much variation in excess/ saving quantities while work is under execution. (Case of Gandhinagar General civil works item wise contract unit 3&4).
- The detailed Billing Breakup (BBU) shall be approved only after approval of drawings & shall cover all major items to be executed in the buildings/structures. Generally, there is tendency of EPC contractor to take more payment from initially executed work which should be taken care of while bifurcating the total value of contract, percentage wise. To eliminate this possibility, Civil Engineers of the utility should be involved before conveying approval of BBU.
- Sequence of construction should be indicated to avoid delay in completion of work and proper coordination between all the stake holders, daily meeting and follow up for progress, monthly progress review meeting with agency etc. to be specified.
- Bar chart for completion of project need to be devised. Each activity and sub-activity shall be covered in the bar chart. Target and achievement shall always be compared during co-ordination meetings. If the completion period is extended, the bar chart will also stand revised.

- To make provision in tender document in the present scenario of market and effective utilization of latest technology of material available for construction so that in future expense of maintenance is minimized. The following may be considered.
  i. Concealed PVC pipeline for water supply and sanitary work.
  ii. Epoxy coated reinforcement steel or corrosive resistance steel.
  iii. Fly ash bricks of minimum 50 kg/sq.cm compressive strength.
  iv. Reinforced concrete road with curbstone and paved footpath with rubber molded paver block on both sides including effective drain out arrangement of rainwater.
  v. Provision of water harvesting well and to be connected with surface water drain of project area.
  vi. Provision of aluminum anodized sliding windows, doors, and glazed partition.
  vii. Provision of granite, kota stone, mat finished vitrified tiles, ceramic tiles, aluminum false ceiling, laminated board wall cladding.
  viii. For inside building oil bound distemper or acrylic emulsion decorative plastic paint, texture painting. For outside waterproof exterior paint like Apex of Asian/ weather shield max of ICI or equivalent.
  ix. For construction of extension units, existing facilities are being used by agencies. The provision of regular maintenance of road footpath etc. are included in item description and contract condition.

5.0 CARE DURING EXECUTION OF CIVIL WORKS AT SITE

5.1 In spite of drafting specifications meticulously and asking the EPC (or other) contractors to agree to it, there are many chances of slippage from the requirement, during actual execution of work. Some of the matters which need to be considered are indicated below.

- The suitability of water to be used for construction purpose shall be ascertained and should conform the water testing as per IS 3025 in laboratory before use. Preferably potable water to be allowed in construction of project work. Due to use of salty water in project construction work, structure/reinforcement start rusting and pitting in very short time.
- The samples of materials to be used for construction purpose are to be submitted for inspection and testing well in advance as per item description of contract. Approved samples are to be preserved for future record duly signed & with a seal of authority.
- Construction material such as steel cement bricks course aggregate, grit ,sand, galvanization to steel section aluminum section rubble, murrum, acid resistance tiles, RCC. Pipe should be tested in National Accreditation Board (NAB) approved laboratory as per IS code and as per item of contract and tender specifications. For all the grade of concrete carry out cube test design mixes in NAB approved laboratory.
- Site should have the well-equipped laboratory to maintain the quality control for civil works as per I.S. code and specification of works. This will save time to take samples to distant places & also avoid delay in receiving test reports. The proper record of laboratory test results is to be maintained also dully signed by all concern for future.
- There should be appropriate arrangement for curing up to tallest structure in which cement is used. In present scenario curing compound is also being used.
- For finishing items of project such as plaster, Kota stone, tiles, I.P.S. granite, Doors, Windows, roof sheeting, painting, false ceiling, wall cladding, road footpath, false flooring are to be supervised stage wise during progress at site, in line and level to have good aesthetic view on completion
- From the beginning of project all the preliminary drawings of civil, electrical and mechanical sent by consultant are to be studied in detail by concerned engineers and offer comments if there is any deviation from contract specifications. Also, our observation on the basis of experience needs to be discussed with consultant before giving clearance for releasing construction drawing.

- Civil foundation drawings lay out plan and mechanical equipment base plate dimension, bolts center distance (PCD), levels, opening for electrical panels, conduits, cable sleeve opening and bus duct connection etc. are to be checked in the drawing properly to avoid miss matching while erection.
- To avoid the cement slurry leakage while vibration of concrete, shuttering should be water tight, proper cover to reinforcement bars should be provided, over vibration to concrete should be avoided, concrete placement should be without cold joint.
- The back filling with selected earth should be carried out properly as per item specification in layer with rolling and watering at optimum moisture content to achieve proctor density more than the 95 % on test.
- Concrete should be placed in such a way that ingredients do not get separated and no segregation of concrete takes place.

6.0 ILLUSTRATIVE PHOTOGRAPHS

6.1 Some photographs related to the deficiency in the civil work in a power project are exhibited below:

Figure 1A & 1B Settlement and crack development due to improper

Figure 2
Importance of shoring in construction works

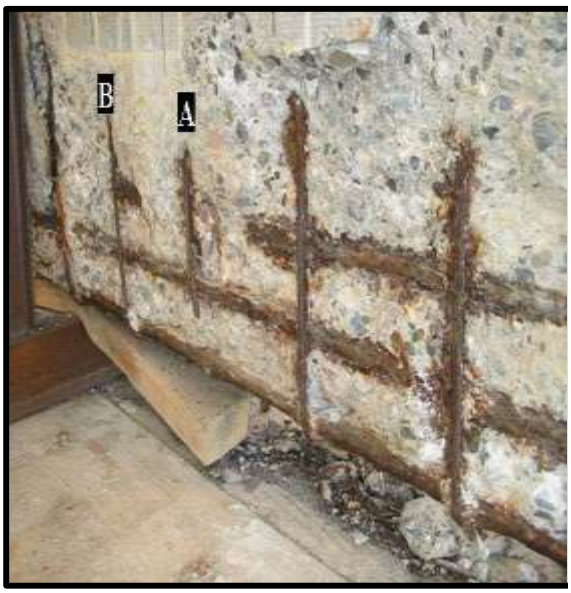

Figure 3
Corrosion of Reinforcement at later stage due to Insufficient Cover

Figure 4 A & B Slurry Leakage & Segregation of Concrete

Figure 5
Building of finished Concrete due to weak shuttering

Figure 6A
Water Leakage during Testing Work

Figure 6B
Water Leakage during Testing Work

Figure 6C
Water Leakage during Testing Work

Figure 6D
Water Leakage during Testing Work

Figure 7 Honey Combing in Concrete

Figure 8
Gravel *filling with small size in pit of transformer*

Figure 9
Concrete chipping *not taken care of finished floor*

Figure 10 Rework due to error in civil drawing

7.0 CONCLUSION

7.1 The civil work in a power plant is a very precise work and calls for accuracy in drafting specification, awarding contracts and execution of work.

7.2 Improper execution of work and lack of supervision leads to many issues and the resultant cost of repairs and maintenance.

About Author

Er. Kailash C Yadav, on completion of BE (Civil) joined a private construction company at Kadana Hydro Project in Jul-1981. Thereafter joined as Junior Engineer (Civil) at Wankbori TPS in 1981.

He worked for civil works construction of various power stations in Gujarat State in various capacities from Junior Engineer up to Chief Engineer (Civil). During his tenure he worked at various power stations such as Kadana Hydro, Wankbori TPS, Gandhinagar TPS, Ukai TPS, Utran & Dhuvaran Gas Units, Charanka Solar Plant and lastly at Vadodara Corporate Office as a Civil Head.

Presently he has been working as GM (Tech.) in mining sector with M/s Durga Infra Mining Pvt. Ltd since Sep-2016

- "Enhancing Consolidation in Problematic Soils with Sand-Filled Drains and Prefabricated Vertical Drains using Jute fibers and Geosynthetics"
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Abstract:

The consolidation of problematic soils with high compressibility and low permeability presents a significant challenge in geotechnical engineering. Ground improvement techniques play a crucial role in strengthening weak, compressible soils, and accelerating the consolidation process. Among these techniques, preloading or pre-compression in conjunction with vertical drains has remained a widely adopted method for in-situ soil consolidation. Prefabricated Vertical Drains (PVD) have been instrumental in expediting primary consolidation, particularly in soft clays, by reducing drainage paths. This paper investigates the consolidation rate of soft soil using two types of PVD: Sand drain and Jute wrapped with Polyamide Polyester drain.

The focus of this study centers on pore water dissipation, the impact of radial drainage flow on consolidation, and the gain in soil strength for both Sand and Jute wrapped with Polyamide Polyester PVD. To carry out this investigation, a 254 mm diameter Rowe Type Oedometer is employed. The applied load increment of $\Delta P/P = 1$ allows for the measurement of settlement using conventional methods.

The results of this study reveal that Jute wrapped with Polyamide Polyester drain as a PVD outperforms Sand drain in terms of consolidation rate and enhanced strength gain. The findings of this research have significant implications for geotechnical engineers and professionals involved in soil improvement projects. The superior performance of the Jute-wrapped PVD suggests its potential as a more efficient and sustainable option for consolidating soft soils. This study contributes to the body of knowledge on PVD techniques, offering valuable insights into the selection of appropriate materials for achieving improved soil consolidation in geotechnical applications.

Key words: *Ground improvement technique, Kaolin soil, prefabricated vertical Drains, Co-Efficient of consolidation due to radial drainage, Shear strength of the soil*

1.1. Introduction:

A ground improvement technique is a method used to enhance the engineering properties and performance of natural soil or ground for construction, geotechnical, or infrastructure projects. These techniques are employed to address soil-related challenges such as low bearing capacity, excessive settlement, poor drainage, or other geotechnical issues. Ground improvement techniques aim to make the soil more suitable for the intended purpose, reducing the risks and costs associated with construction.

In practical applications, there is a growing need to use poor-quality soil for foundation support and earthwork construction. Among various ground improvement techniques, consolidation due to radial flow using vertical Geodrain is an effective method. This technique alters the rate of settlement through the use of vertically installed prefabricated Geodrains. The consolidation time varies with the square of the drainage path length, and most soil deposits have greater permeability in the horizontal direction than in the vertical direction.

The installation of vertical drains intercepts the horizontal flow of pore water and reduces the drainage path, leading to an increased rate of consolidation as shown in Fig.1. In any pattern of drain installation and under a consolidating load, the drainage path can be significantly reduced, often by a factor of four. To achieve this, a sand layer is placed below the embankment and above the soil that needs treatment, with vertical drains inserted to expedite the consolidation process

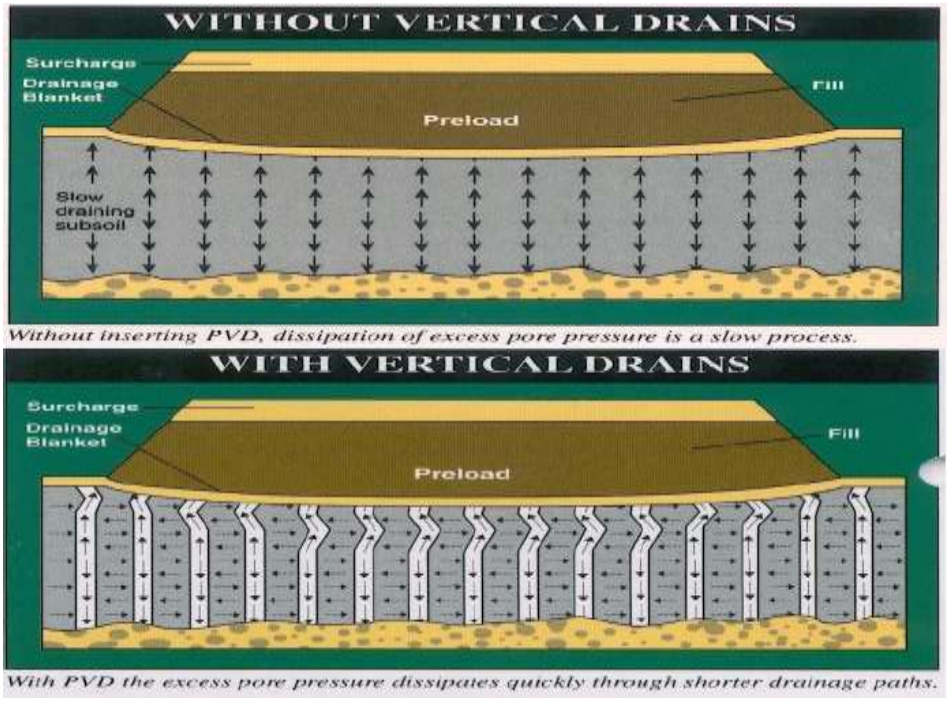


Fig. 1: Use of PVD in soil blanket for Ground improvement technique

1.2 Literature Survey:

Singh et.al (1979), Brednev et al (1979) , McGown et al. (1981), Hansbo (1981),P.L.Newland (1960) , Dastidar S. Gupta (1969), Hansbo (1983), Kremer (1983), Jamiolkowski et.al (1984),Bergado et.al (1991), Koerner et.al (1994-2004), Chai. J.C. (1999), Sharma et.al. (2000), Hird et.al (2000,2002), J. Chu et.al (2003, 2005, 2008), J. N. Mandal et.al (2003, 2007) Indraratna et.al (1998, 2008, 2010), Chen Et al (2007) , Lee et.al (2007), Shroff et.al (2006, 2007,& 2012), Indraratna et. al (2015), Holtz et.al (1991), Madhav et al (1993), Ladd (1989), Nguyen & Hung (2010), Hazi (2014) and G.V.Rao (2008) have carried out experimental and field study to assess the effect of various geometric shapes of mandrel, consolidation by means of band- shape drains, drain Installation methods, field application of sand wick, horizontal Oedometer test, laboratory test with jute coir, geogrid, quality control test of PVD viz. rate of loading, discharge capacity ie. hydraulic conductivity of the drain, filtration requirement, bending/kinking deformation of drain, drain spacing factor including disturbance factor, average drain resistance factor, uniformity of the treated mass considering various patterns of drains, etc.

1.3. Objective:

The objective of the present investigation is to examine the effect of cost effective, eco- friendly and recycled waste materials of drain on strength of consolidated soft soil mass due to radial drainage using central drain from fundamental considerations. The present investigation provides economical materials for preparation of Drain, comfortable operational Procedure, Effective results for improving the bearing capacity of the soil.

Modern modified Rowe type hydraulically pressurized Oedometer is employed to maintain exactitude of measurements of settlement and pore pressure. The hydraulically pressurized Oedometer (Rowe Type) with central Geodrain is employed in present investigation and Cvr value is determined for both settlement and pore pressure readings. Measurements of pore pressures are planned to carry out by conventional Bishop's pore pressure setup and extensometer. It is intended to conduct a series of Consolidation tests with radial drainage through vertical Geodrain centrally placed and measurement of Settlement and pore water pressure at 3 radial distances will be taken so as to prove efficacy against various physical factors namely type of drain material like Sand and Jute wrapped with Polyamide Polyester Geosynthetics to fabricate the vertical drain.

1.4. Laboratory Investigation:

The soil used for this investigation was clay mineral Kaolinite obtained commercially in the form of powder. To ensure full saturation of the sample the clay was mixed to form slurry with twice the liquid limit using de-aired distilled water.

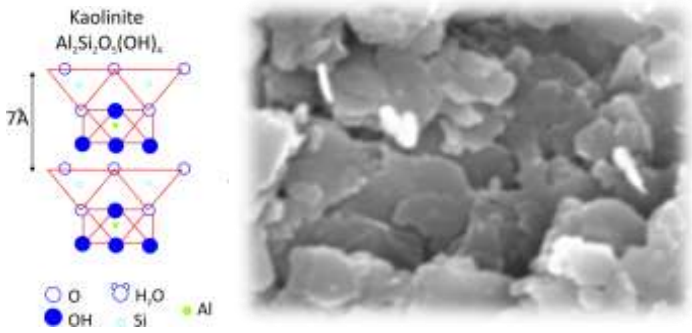


Fig.2: Clay mineral structure

The atomic structure of the clay minerals consists of two basic units, **an octahedral sheet and a tetrahedral sheet**.

The octahedral sheet is comprised of closely packed oxygen's and hydroxyls in which aluminium, iron, and magnesium atoms are arranged in octahedral coordination fig.2.

Table-1 Property of Kaolinite Soil

Type of clay	**Kaolinite**
Specific Gravity	2.456
Liquid Limit (LL)	59.6%
Plastic limit (PL)	33.33 %
Plasticity Index (PI)	26.27 %
Classification by A-line Casagrande Chart	CH
Co-efficient of Permeability	0.6 x 10^{-6} cm/sec

Table-2 Properties of Sand Used in Sand Drain

1	Specific Gravity	2.645
2	Co-efficient of Permeability	8.45 x 10^{-3}cm/sec
3	Co-efficient of uniformity, c_u	2.023
4	Co-efficient of curvature, c_c	1.295
5	Fineness Moduli	2.2-2.3
6	Sand classification	SW

Table-3 Properties of Prefabricated Vertical Circular Jute wrapped with Polyamide Polyster Geosynthetics

Sr. No.	Prperties of newly developed Jute wrapped with Polyamide Polyster drain	Unit	Value
1	Weight	gms	5
2	Diameter	cm	2.1
3	Height	cm	2.1
4	Peak unconfined stress for flexibility measurement	kG/cm^2	0.8
5	AOS (095) - Filter	µm	<75

6	Permeability - Filter	Cm/sec	1.1×10^{-1}
7	Grab Tensile Strength - Filter	KN	0.6
8	Elongation at break - Filter	%	25

1.5. Experimental Set up:

The Experimental setup used in the present investigation consists of:

(i) Hydraulic Pressure System
(ii) Oedometer
(iii) Pore pressure measurement system.
(iv) Settlement measurement system

The hydraulically pressurized Oedometer (Rowe Type) with central Geodrain as shown in Fig.3 is employed in present investigation and Cvr value is determined for both settlement and pore pressure readings. Measurements of pore pressures are planned to carry out by conventional Bishop's pore pressure setup.

Fig. 3: Rowe Type Oedometer and self-compensating Mercury

1.6. Method of Soil Preparation

The soil used for this investigation was clay mineral Kaolinite obtained commercially in the form of powder. To ensure full maturation of the sample the clay was mixed to form slurry with twice the liquid limit using de-aired distilled water. Density was sufficiently low to allow the removal of entrapped air when the sample in the consolidation cell was vibrated. The slurry was transferred into the Oedometer after the cell body had been lightly coated with a thin layer of silicon grease to minimize side friction; the Oedometer was then placed on a handle operated vibrator and vibrated for approximately one hour after which only occasional air bubbles could be seen on the surface. The clay was then scribed level. And a filter paper followed by a porous stone was placed at the top. The sample was then preconsolidated under gradually applied static dead load of 10KPa, with $\Delta p/p$ = 1kPa, 2kPa, 4kPa, 8kPa, 10 kPa) so that the consolidation occurring is normal. (Where 1kPa = 0.01kg/cm^2).These increments are given by means of dead load with porous stone on the top of the clay sample topped by filter paper so that sample during consolidation water gets re- moved through porous stone. These increments of Dead loads are to be kept for a longer period of time (at least 48 hours). Representative sample for determination of water content was taken and measured. Initial strength was measured by Vane shear in separate crucible of vane shear apparatus by dead loading in same fashion by small weights.To avoid the soil structure disturbance of soil cake prepared. Initial Height was measured with pointer arrangement with stand and the extra soil was trimmed. Filter paper is kept on the surface of the trimmed soil and rigid Perspex plate was laid on it to create the equal strain condition while loading.

1.7. Installation stages of Vertical Drain

The axial hole was formed with a thin walled mandrel, having area ratio of 0.8 to 1.6 attached with template and guide frame. Circular Filter paper of size PVD is lowered in the bore hole. A drain hole was then flooded with water from the central connection to the reservoir. The drain was filled with de-aired saturated PVD drain material with the aid of small diameter flexible tubing by syphoning

action without any smear and without any intrusion of clay in PVD to avoid blockage. The top cover is then seated into position.

Fig. 4: Soil Sample in Oedometer- Before & After Test

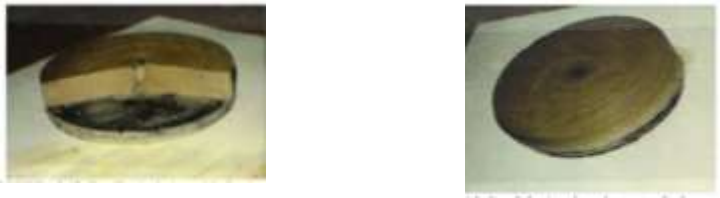


Fig. 5: Typical view of Soil cake with Sand drain after the test Oedometer

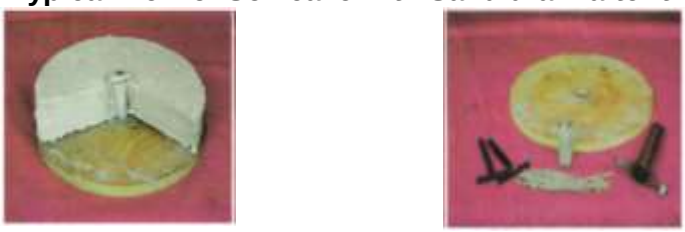


Fig. 6: Typical view of Soil cake with JPPG drain after the test Oedometer

1.8. Test Procedure

After the cell is sealed, settlement dial gauge and Bishop Pore pressure measuring apparatus were connected at their respective location. The first pressure increment is applied through the flexible convoluted jacket after closing the drainage control valve and settlement gauge reading is recorded. After completion of consolidation process drain- age control valve was closed then the next increment of load is applied and the same process is repeated for a series of various pressure increments, with P/P = 1.0. The loading was done in the increment of 20, 40, 80, 160 and 320 kPa keeping $\Delta P/P = 1$.

After the completion of the test, vane shear test was performed at 3 locations to determine the gain in strength due to dissipation of water. The value of settlement and pore water pressure is measured with respect to time and co-efficient of radial consoli dation is calculated. The typical view of soil cake before and after the test for Sand drain and JPPG as shown in Fig.4, Fig. 5 and Fig.6.

1.9. Result Analysis:

Results of following graphs are presented and discussed:

(a) Settlement:

1) Degree of consolidation against log of time (Ur % v/s log t).
2) Coefficient of consolidation against degree of consolidation (Cvr v/s Ur %).
3) Coefficient of consolidation against Pressure
4) Void ratio against log of consolidation pressure (e v/s log p).

(b) Pore Pressure:

1) Degree of consolidation against log of time ((1-Ur/Uo) % v/s log t),
2) Comparison of theoretical time factor curve with experimental

(C) Shear Strength of Soil with respect to various drains:

2.0 Results and Discussion*:*

1) Degree of consolidation against log of time (Ur % v/s log t) - Rate of settlement:

General trend of the curve is inverted S- shape having initial flange prominent and end flange inconspicuous i.e. not fully developed.

These curves are comprised of three zones: first zone - Initial resistance to compression (rate of consolidation is very slow); second zone - gradual rate of consolidation and third zone – rapid rate of consolidation ending to 100% consolidation for both material of central drain. In sand drain 1st zone is more compared to JPPG which shows 15%, for 2nd zone where gradual rate of consolidation persists is 55% in sand and merely 30% in JPPG. 3rd zone in each drain follows with a rapid rate of consolidation immediately after 2nd zone ending to 100% consolidation as shown in Fig.7

Degree of consolidation and log time graphical relationship curve of JPPG exhibits least resistance to initial consolidation rate as well as minimal resistance at 50% consolidation showing 190 minutes only compared to other drains Rate of consolidation with respect to settlement at 50 % consolidation is faster in JPPG, as compared to sand. The hydrodynamic lag for sand is more as compared to Jute wrapped with PP Geosynthetics.

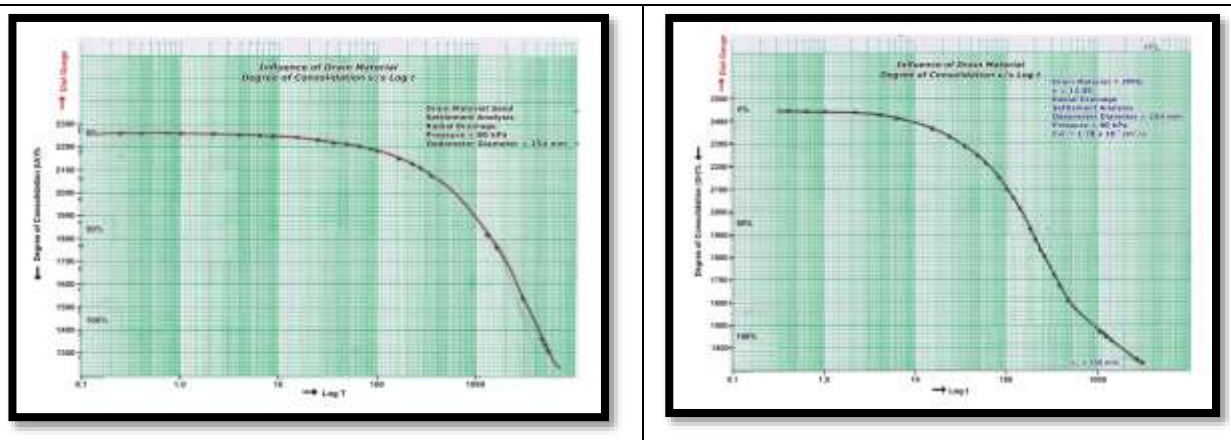


Fig. 7: Degree of consolidation and log time graphical relationship curve of JPPG & sand Drain

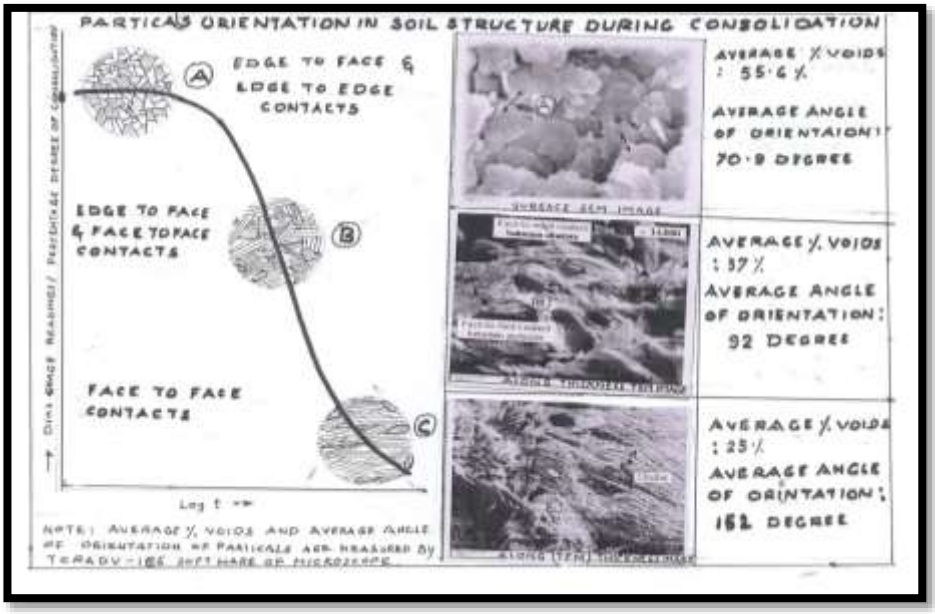


Fig. 8: Particle orientation in soil structure during consolidation

Quantum of above structural arrangement Initial resistance & gradual resistance to consolidation is decreased while the rate is increased with increase of pressure.

It means under higher pressure more or less all the particles reorient to “face to face” configuration indicating least magnitude of voids as shown in Fig.8. Irrespective of any loading, JPPG exhibits faster rate of consolidation at all percentage of consolidation.

2) Co-efficient of Consolidation (Cvr) Versus Degree of Consolidation (Ur %): The curve of Cvr vs. Ur shows exponential curve for all the pressures. The characterization of the curve consists of 3 zones: I zone belongs to sudden drop of Cvr value with degree of consolidation , II zone comprises of gradual decrease from point of deviation to point of asymptotic to horizontal axis and III zone consists of constant value of Cvr with horizontal x-axis with progress of consolidation as shown in Fig.9.

General trend for both drain materials indicates that rate of degree of consolidation is faster up to 5 to 20% and thereafter the rate decreases gradually upto 60 to 80% leading to asymptotic parallel to x-axis showing same value of Cvr.

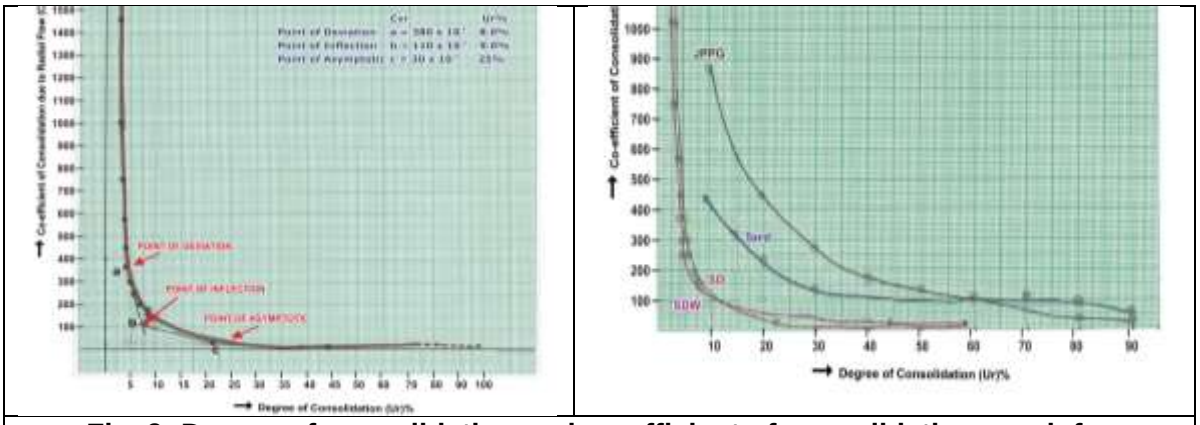


Fig. 9: Degree of consolidation and co-efficient of consolidation graph for JPPG & sand Drain

In both the drains Cvr at 40% consolidation are 6.78 & 4.35 x 10-4 cm2/sec for JPPG, Sand respectively.

Faster rate of dissipation of pore water pressure and rearrangement of plate shape particles from face to edge to face to face randomly constitute faster rate of consolidation.
The microfibers of JPPG is such that micro pores of it facilitate easy outflow and inflow of pore water through this vertical drain from soft soil mass to be treated exhibiting good hydraulic conductivity.

3) Cvr versus Pressure:

Coefficient of consolidation due to radial drainage remains more or less constant for all effective pressures and drain materials except in sand drain i.e. Cvr is independent of all effective pressure as shown in fig. 10. In sand drain arching effect during the formation of the drain in the bore hole might have affected the ‘continuity of the drain’. This might have caused variation at effective higher pressure.

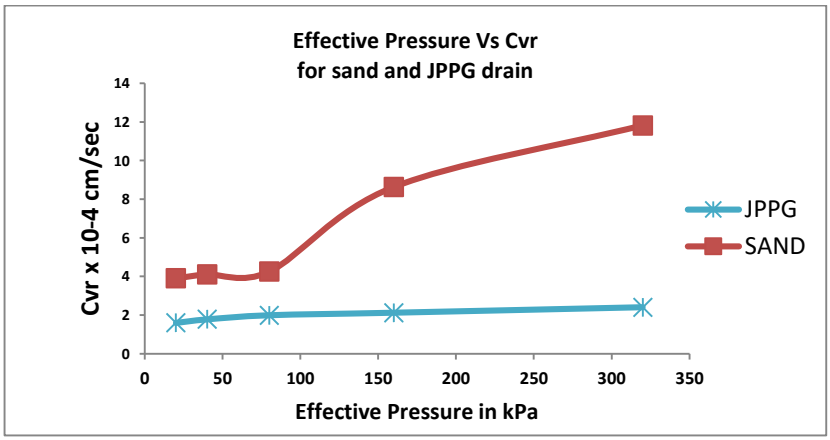
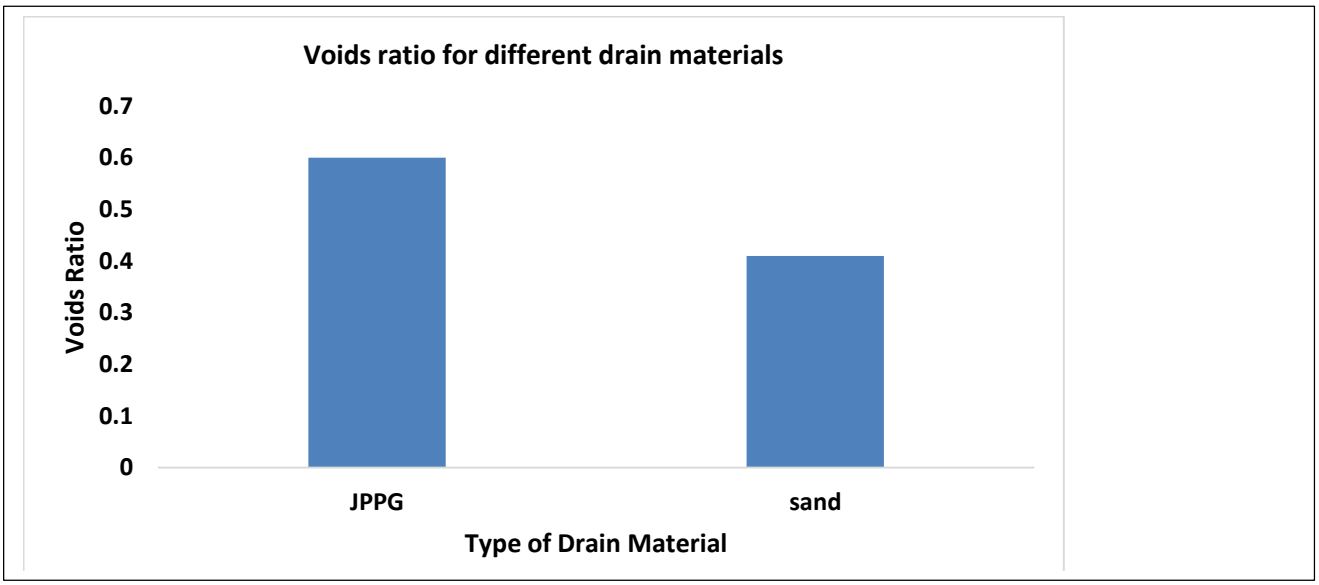


Fig. 10: Degree of consolidation and effective pressure curve of JPPG & sand Drain

4) Void ratio against log of consolidation pressure (e v/s log p).



Amongst sand and JPPG drain, JPPG is more effective in exhibiting more rate of compressibility of the soil compared to sand drains. Primitive material sand also shows their competency in making good rate of compressibility of the soil. Higher the Ccr value indicates higher rate of compressibility of the soil as shown in the fig.11. Higher rate of compressibility of the soil reveals that settlement will complete in less time i.e. the goal of accelerating the rate of consolidation by radial flow employing central drain.

(b) Pore Pressure:

1) Degree of consolidation against log of time ((1-Ur/Uo) % v/s log t):

In general, rate of dissipation at r/4 (where r is distance from central drain to periphery) requires least time for any percentage of Consolidation compared to mid plane i.e. radial point of r/2 and farthest radial point of 3r/4 from central drain for any degree of consolidation and at any pressure. As the distance increases from central drain, development of pore water takes more time compared to Mid-plane and near to the drain. As the pressure increases this delay in development of pore pressure is reduced. Therefore mid-Plane pore water pressure is considered to compute degree of consolidation as shown in Fig.12.

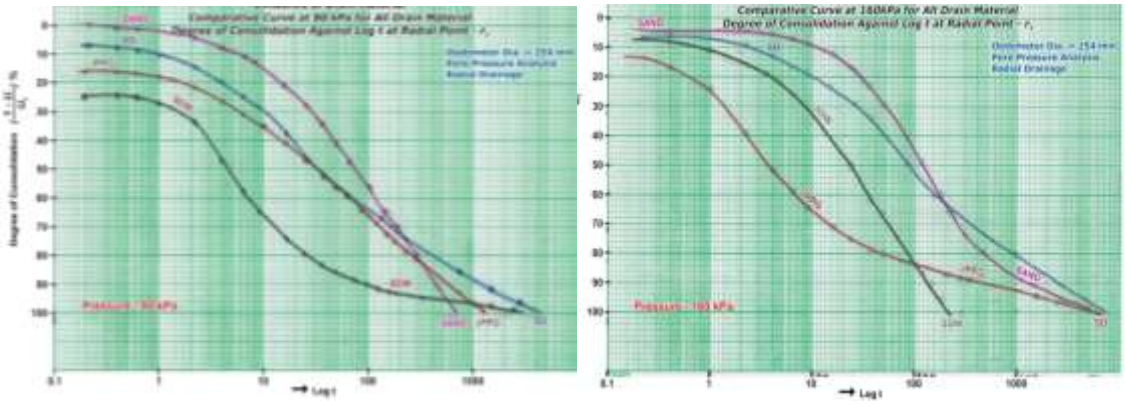


Fig. 12: Degree of consolidation and log time graphical relationship curve of JPPG & sand Drain for pore pressure

Both the drains sand & JPPG bears ideal curve of Pore pressure dissipation v/s degree of consolidation bearing “S”- Shape of curve as shown in Fig .12 composed of early development of Pore water pressure with sudden decrease of pore water pressure tending to constant value there after gradual rate of dissipation reaching to zero. Pore water pressure at mid plane is low for Sand drain as compared to JPPG at all effective stresses.

2) Comparison of theoretical time factor curve with experimental time factor:-

In case of JPPG drain, the experimental and theoretical curve adhere well from 5% to 90% degree of consolidation. During initial 5% degree of consolidation, 2 to 3 % variation and at end portion 5 to 7% variation is observed.

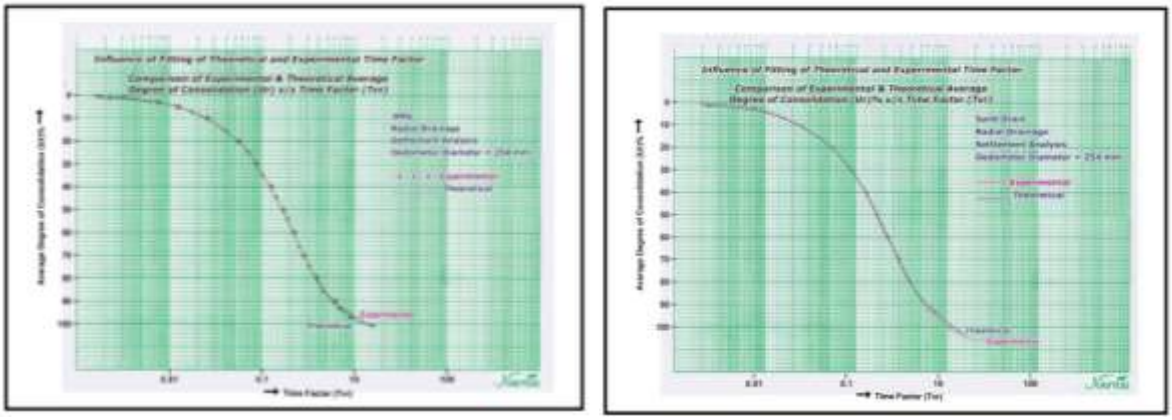


Fig. 13: Comparison of theoretical time factor curve with experimental curve of JPPG & sand Drain for pore pressure

In Sand drain theoretical curves deviate below the experimental Tvr curve as shown in Fig.13

(C) Shear Strength of Soil with respect to various drains:

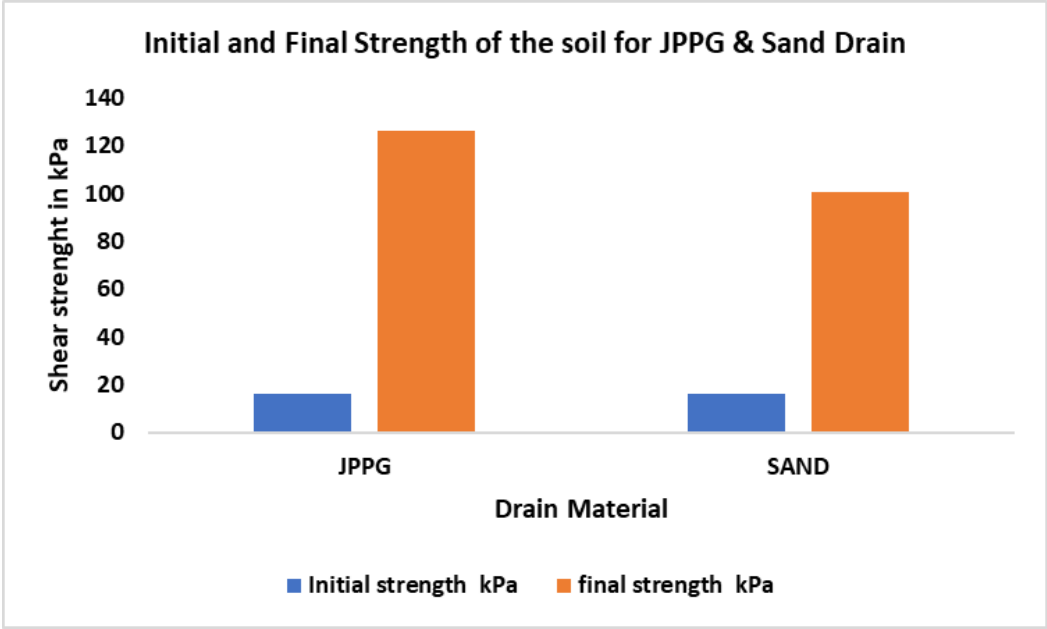


Fig.14: Shear strength of soil for JPPG and Sand drain

It is observed that mid plane strength at r2 in every material is higher compared to near the Drain and at farthest radial point at r3. For any radial distance from central drain, JPPG exhibits higher strength i.e. 126kPa at r2 compared to sand drain i.e.100.6 kPa at r2 as shown in Fig.14. This is due to lengthy radial drainage path at farthest radial point while at near the drain higher water gradient mobilizes the shear strength to some extent.

2.1. Conclusion*:*

Coastal areas with soft clay soil pose unique challenges for road construction due to the heavy compressibility of saline water present in the soil pores. This not only hinders the bearing capacity of the roads but also leads to increased maintenance costs over time. Our research focuses on optimizing ground improvement methods, particularly the acceleration of consolidation through the use of Primitive Sand Drains extended to Prefabricated Vertical Geodrains.

The research findings suggest that by implementing the suggested guidelines and utilizing cost-effective materials for drain preparation, MOST can achieve substantial cost savings of 20% to 30% in the construction of roads within coastal areas with soft clay soil.

In practical terms, a relative cost of 380 suggests that for a given area, using 15-meter deep sand drains is 380 times more costly than using JPPG drains, which are considered the standard or reference in this context. This information can be valuable for project planning and budgeting, helping decision-makers understand the cost implications of choosing one method over another for ground improvement in the specified conditions.

Table: 4 Relative Cost per Unit Area Treated by 15M Deep Drain

Drain type	Equivalent diameter	Square spacing* in Meter	Relative cost per 15 M deep +, ++	Relative cost per unit area treated+, ++	Equivalent diameter
Sand drain	30	4.93	790	380	30
Band drain shape of JPPG	50	2.40	100	100	50

*assuming t=6months and C_{vr} = 5 M^2/year, +assuming JPPG drains for 100% cost; ++excluding all on site establishment cost, above table 4 justifies JPPG drain may be cost effective to be adopted

While both methods aim to improve soil stability and reduce settlement, PVDs offer several advantages over traditional sand drains like Installation Speed, Uniformity and Consistency, Reduced Soil Displacement, Greater Vertical Reach, Environmental Considerations, Cost-Effectiveness and Flexibility

It's essential to note that the choice between PVDs and sand drains should be made based on a thorough geotechnical investigation and project-specific considerations. While PVDs offer many advantages, there may be situations where sand drains are a more suitable or cost-effective solution, depending on factors such as soil composition, project requirements, and budget constraints.

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NON DESTRUCTIVE TESTING OF STEEL STRUCTURES AND FOUNDATIONS OF SWITCHYARD – A CASE STUDY

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1.0 INTRODUCTION

1.1 As the asset starts aging, it becomes necessary to assess the health and the integrity of it.

1.2 Non-destructive testing is being done by most of the power utilities on such assets or equipment.

1.3 When the asset is in the form of switchyard structures made out of mild steel and is found to be aging, specific non- destructive tests have to be performed. Similarly, the foundations on which the structure is resting also need to be tested for their present condition.

1.4 The non-destructive test may be invasive or non-invasive.

1.5 The presentation below covers a case study of a 132kV switchyard on which the team of Author's company carried out non-destructive testing.

2.0 REASONS FOR NDT

2.1 The 132kV switchyard is evacuating power from the thermal power station. The switchyard is located close to the power station which is more than 50 years old. As the power station augmented its capacity, it had become necessary to extend the switchyard periodically.

2.2 Due to the presence of coal particles (Fly ash) and the condensation from the cooling towers of the thermal power station, the steel structures suffered from rusting and pitting.

2.3 Due to periodical extension of the switchyard and replacement of damaged/unserviceable equipment, lot many structural changes were required to be made over a period of time. There was a structural mismatch at many places.

2.4 Most of the structures were painted (not galvanized). Repeated painting created layers on the structure but could not prevent rusting and pitting.

2.5 The number of welded joints had become weak and strength of these joints was doubtful.

2.6 Verification of structure was also doubtful.

3.0 ESTMATION OF WORK INVOLVED & TIME FRAME

3.1 In order to organize the NDT work, it was necessary to list out the scope of work. The following broad scope of work was chalked out.

3.2 There are 36 columns in 132kV switchyard, and each leg has one raft foundation with four columns (one per leg). Thus, in total there are 144 numbers of foundations in the switchyard. Excavation work for carrying out NDT on foundation was in the scope of work. During the inspection in the switchyard, it was found that the foundations of all the columns and equipment support structures are well maintained and robust. Some surface cracks were observed on the top portion of the concrete columns. Therefore, it was jointly decided to excavate some critical looking foundations and carry out NDT on them. It was further decided that if the results of such NDT on foundation are found convincing, it may not be necessary to investigate all the foundations through NDT. Ultimately, 25 foundations were selected for inspection and carrying out NDT. Samples were taken from all these 25 foundations. These foundations were constructed around 50 years before and at that time M15 grade of concrete was prevalent. The surface looked smooth and there were no honeycombing or other major faults found in the foundation visually. Some photographs related to the inspection of foundation are exhibited below.

Looking to the test results, 94% foundations showed good compressive strength (i.e., more than 15 N/mm^2), on the other hand 6% foundations showed doubtful results (less than 15 N/mm^2). For these weak foundations, rectification plan was given in the report.

3.3 NDT on foundations included the following.

3.1.1 Conducting Rebound Hammer Test in a grid of 30cm x 30 cm including surface preparation as per IS: 13311 PART 1 & 2.

- Extraction of concrete cores from structural elements and testing for density, absorption, compressive strength, and carbonation as per requirement.
- Half-cell potentiometer Test as per IRC-SP-40 in a grid of 300mm x 300mm in a $1M^2$ area.
- Photometer test, Cover meter test for determination of location and cover of rebars.
- Chemical test, Chloride, Sulphate & Ph value

3.4 Mechanical Test for Steel structures included the following.

- Visual inspection and studying the condition of corrosion, painting condition, sagging of members.
- Thickness survey using ultrasonic Thickness gauge.
- Inspection of weld joints by DPT test
- Condition of bolted connection

4.0 Challenges in the assignment

4.1 The switchyard is meant to evacuate about 360MW of power generated from the thermal power plant located at a distance of 100M and above. The switchyard is supplying power at 132kV & 66kV to various step down substations of a metropolitan city withing a radius of 75KM. For this fact it was not possible for the authorities to give continuous outage on any section of the switchyard. The authorities allowed outage on any one circuit at a time and the work had to be done within 6–8 hours of outage. Thus, the safety of the crew members was always a concern.

4.2 Due to the availability of shutdown (outage) for only one circuit, induction from the live circuits had to be contained. The earthing system for the safety of crew members had to be specially devised.

4.3 The structures of the switchyard are very old and there was visible damage to them. The work had to be done with proper precautions. Fortunately, the client provided a collapsible platform to perform certain activities. However, these platforms did not help to work on the gantry beams. This work had to be done by monkey climbing only.

4.4 It was mandated to depute a safety officer continuously during the period of field work of the assignment. The safety officer was expected to supervise the electrical and mechanical safety of working crew members during the tenure of actual work in switchyard.

4.5 The foundations of gantry columns and equipment support structures were constructed many years ago and it was not possible to foresee the condition from the ground level. The visible cracks were not extending much below the ground level.

4.6 Since the work was to be done in the premises of a Thermal Power Plant, security formalities had to be done in advance. Besides daily routine entry and existing formalities were also essential for security. In case a working crew member is changed the security formalities have to be repeated. These formalities needed additional time allowance.

4.7 Since the client is a big corporate company, H.R. and labour related formalities had to be done and all the compliances had to be ensured. This was time consuming and expensive / proposition. Besides, a lot of preparation had to be done for the compliance.

5.0 Details of Actual Work on Foundations

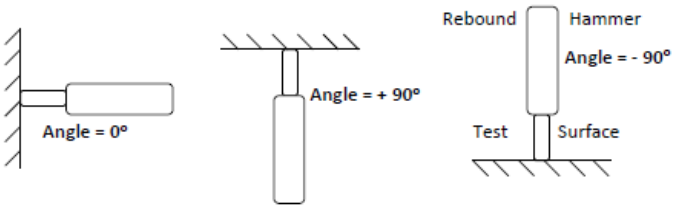
5.1 Conducting Rebound Hammer Test in a grid of 30cm x 30 cm including surface preparation as per IS: 13311 PART 1 & 2.

5.1.1 Rebound Hammer Tests are carried out in accordance with I.S. 13311 (Part 2) – 1992 for assessing the likely compressive strength of concrete, the uniformity of concrete and the Quality of concrete with respect to standard requirements.

5.1.2 The apparatus used consists of a spring-controlled mass that slides on a plunger within a tubular housing. The impact energy required for rebound hammers is 2.25 Nm.

Rebound Hammer Testing

5.1.3 For rebound hammer testing smooth, clean, and dry surface was selected. If loosely adhering scale is present, this should be rubbed off with a grinding wheel or stone. Rough surfaces resulting from incomplete compaction, loss of grout, palled or tooled surfaces do not give reliable results and should be avoided.



Schematic Diagram of Rebound Hammer Positions

5.1.4 For taking measurements, the rebound hammer is held at right angles to the surface of the concrete member. The test can thus be conducted horizontally on vertical surfaces or vertically on horizontal surfaces. If the situation demands, the rebound hammer can be held at intermediate angles also, but in each case, the rebound number will be different for the same concrete.

5.1.5 The logic of using rebound hammer for NDT is that it is very handy and simple to perform needing less manpower. It gives preliminary idea about the structural status of the cast concrete. It is possible to categorize various structures in a broad way like very good concrete, good concrete, and average/poor concrete. Large numbers of tests were done on concrete structures which lead to good precision regarding the homogeneity of the concrete. The detailed analysis of rebound hammer test leads to the decision regarding other types of NDT to be performed on the concrete or otherwise.

5.1.6 The Rebound hammer test result of some of the gantry columns, is given in the table below.

Specimen of Rebound Hammer Test Result

Sr. No.	Column no./ leg no.	Rebound Number					Average	Std Dev	Sigma (Avg. Strength) N/mm^2 (Min. 15 N/mm^2 required)
1	C-1/1A	51.50	80.50	66.00	50.00	55.00	57.3	9.5	73.9
		51.50	58.00	49.50	56.00	54.50			
2	C-1/1B	53.50	53.50	55.00	66.50	54.50	56.0	5.5	68.2
		49.50	50.50	64.50	58.00	54.50			
3	C-1/3A	48.50	50.50	54.00	45.00	34.50	53.1	11.0	56.7
		53.50	50.00	73.00	53.00	69.00			

4	C-1/3B	61.50	48.50	51.50	57.00	69.00	57.9	9.3	76.8
		49.50	49.50	56.50	58.50	77.00			
5	B-2/2A	48.50	46.00	71.50	73.00	55.00	58.6	13.8	80.6
		86.50	56.00	54.00	44.00	51.50			
6	B-6/2B	42.50	37.00	46.00	41.50	17.50	31.4	10.0	14.1
		21.50	28.00	30.50	20.50	28.50			
7	A-7/1A	24.50	32.00	34.00	38.50	32.00	34.4	6.0	17.1
		33.00	36.00	29.50	47.00	37.50			
8	A-7/1B	41.00	29.00	39.00	38.50	33.50	33.3	5.1	16.0
		32.00	36.00	29.00	28.00	27.00			
9	A-7/3A	37.50	27.50	37.50	40.50	39.00	34.0	5.0	16.6
		28.00	30.50	33.00	28.50	37.50			
10	A-7/3B	35.00	36.50	27.50	36.50	39.50	35.7	4.4	18.6
		44.00	31.50	35.50	34.00	37.00			
11	C-7/1A	25.00	23.50	19.00	16.50	20.50	23.0	6.8	8.2
		15.50	18.00	37.00	31.50	23.00			
12	C-7/1B	19.50	19.00	25.00	15.50	23.00	19.6	3.4	6.6
		14.50	17.50	21.50	22.50	17.50			
13	C-5/1A	38.00	45.50	39.50	47.00	40.50	43.2	10.0	30.1
		41.00	51.50	50.00	58.00	21.00			
14	C-5/3A	55.50	61.00	45.50	60.00	53.50	55.4	4.5	65.4
		58.00	51.50	55.00	55.50	58.00			
15	D-6/4A	33.00	46.50	32.50	40.00	39.00	33.9	6.1	16.6
		27.50	32.50	27.00	29.50	31.50			
16	D-6/4B	30.50	34.50	43.50	51.50	35.00	35.6	7.4	18.5
		34.50	24.00	34.50	31.50	36.50			
17	D-8/2A	39.50	43.50	42.50	34.00	53.50	43.8	6.4	31.3
		35.00	47.00	51.50	47.50	44.00			
18	D-8/2B	40.00	45.50	44.00	50.50	37.00	43.5	6.1	30.7
		44.00	35.50	48.50	37.00	53.00			

5.1.7 In the above table results of serial number 6, 11 & 12 are not satisfactory. Results of sr. 7, 8, 9, 10, 15 & 16 just meeting the requirement of M15. The results of the remaining serial number are satisfactory or more than satisfactory. As a matter of fact, 52 rebound hammer tests were conducted but only 3 results are found to be unsatisfactory.

5.2 Extraction of concrete cores from structural elements and testing for density, absorption, compressive strength, and carbonation as per requirement.

5.2.1. The Extraction of concrete cores is a kind of invasive test. This test is used for testing of actual properties of concrete in existing structures such as strength, chemical analysis, carbonation etc. Concrete cores are usually cut by means of a rotary cutting tool with diamond bits. In this manner, a cylindrical specimen is obtained usually with its ends being uneven, parallel, and square and sometimes with embedded pieces of reinforcement. The core samples can also be used for determining the strength and density of concrete, depth of carbonation of concrete & chemical analysis.

5.2.2. This test has not been performed in the present assignment because looking to the RCC column condition visually and by referring to the rebound hammer test results, it was concluded that foundations are very good in condition despite some minor external cracks. Moreover, the substation is very much live and it is not required to take any undue risk as the civil work done 45 years before is very sound. When 25 numbers of foundations were excavated and concrete work was exposed, it was observed that the concrete work is absolutely good.

5.3 Half-cell potentiometer Test as per IRC-SP-40 in a grid of 300mm x 300mm in a 1M2 area.

5.3.1. The method of half-cell potential measurements normally involves measuring the potential of an embedded reinforcing bar relative to a reference half-cell placed on the concrete

surface. The half-cell is usually a copper/copper sulphate or silver/silver chloride cell, but other combinations are also sometimes used. The concrete functions as an electrolyte and the risk of corrosion of the reinforcement in the immediate region of the test location, may be related empirically to the measured potential difference.

5.3.2. This technique is used for assessment of the durability of reinforced concrete members where reinforcement corrosion is suspected.

5.3.3. This is an invasive test. For performing this test, the RCC column are required to be damaged to expose a reinforcement bar so that both probes could be attached, and readings of voltage could be taken.

5.3.4. During co-ordination meeting it was jointly decided to drop this item from scope of work. This was for the reason that the very healthy concrete work may unnecessarily be damaged and may cause deterioration of rest of the steel reinforcement of the concrete either through capillary or direct ingress of water.

5.4 Photometer test, Cover meter test for determination of location and cover of rebars.

5.4.1 Cover-meter test is a non-destructive test which is used to identify the **location of reinforcement bars** in the as built concrete work and to determine the exact **concrete cover available and then compare with the specified cover**. The magnetic rebar locator test plays a significant role in construction audit works because the information about the location of steel bars, concrete cover, and bar sizes is essential before the work is completed and handed over. It is also essential to investigate the failure of concrete work and health monitoring of concrete work after a passage of long time (such as the present assignment).

5.4.2 A cover-meter is a device that gives information about concrete covers and steel reinforcement in concrete using magnetic fields. Magnetic instruments for locating reinforcement steel (present within the concrete) work on the principle that the steel affects the alternating magnetic field.

5.4.3 When a hand-held search unit is moved along the concrete surface, a beep indicates that the unit is located directly above a reinforcing bar. These meters can also be used to estimate the depth of a bar if its size is known or estimate the size of the bars if the depth of cover is known.

Figure-1 Testing of cover by a meter

5.4.4 During the site co-ordination meeting it was decided to do some sample testing of concrete cover over the reinforcement. This decision was taken on the strength of visual observations of the concrete work in the switchyard which presented a good picture of the concrete work done 50 years back.

5.4.5 The Cover meter test result is given in table below.

Table 1 Cover meter results

Sr. No.	Testing location (column number)	Main steel depth (mm) (Min 50mm)	Distribution steel depth (mm) (Min 40 mm)
1	**C1-1**	**60.0**	**40.0**
2	**C1-3**	**70.0**	**60.0**
3	**B2-2**	**60.0**	**40.0**
4	**B2-4**	**60.0**	**45.0**
5	**D2-2**	**70.0**	**60.0**

6	**D2-4**	**60.0**	**50.0**
7	**A3-1**	**80.0**	**70.0**
8	**A3-3**	**100.0**	**90.0**
9	**C3-1**	**50.0**	**40.0**
10	**C3-3**	**70.0**	**60.0**
11	**D4-2**	**70.0**	**60.0**
12	**D4-4**	**60.0**	**55.0**
13	**A5-1**	**100.0**	**90.0**
14	**A5-3**	**110.0**	**100.0**
15	**C5-1**	**60.0**	**50.0**
16	**C5-3**	**70.0**	**60.0**
17	**B6-2**	**70.0**	**60.0**
18	**B6-4**	**70.0**	**60.0**
19	**D6-2**	**70.0**	**60.0**
20	**D6-4**	**90.0**	**80.0**
21	**A7-1**	**90.0**	**80.0**
22	**A7-3**	**70.0**	**60.0**

5.4.6 Interpretation: Looking to the result of cover-meter test, the average cover value of the main reinforcement bar is 73 mm and minimum value of the same is 50 mm. The average cover value of the stirrups is 62 mm and minimum value of the same is 40 mm. Thus, these values shows that cover is well maintained and no reinforcement is exposed or susceptible to corrosion.

5.4.7 It may be important to note that in earlier days recommended cover used to be 25 mm only (from the main bar). Thus, the concrete cover available is much more than required.

5.5 Carbonation test

5.5.1 Generally, chloride and sulphate ions content tests should be carried out on powdered samples of concrete taken during drilling or from core drilling or other concrete fragments obtained from the structure. This is an invasive test.

5.5.2 The test for chloride content in concrete is very significant as when chloride is present in reinforced concrete it can cause corrosion of the steel reinforcement. Chlorides can originate added to the concrete at the time of mixing. This includes calcium chloride accelerating admixtures, contamination of aggregates and the use of sea water or other saline contaminated water. b) “External” chloride, i.e., chloride entering the concrete post-hardening by sea water or saline water nearby. The test involves crushing a sample of the concrete to a fine dust, extracting the chloride with hot dilute nitric acid, and then adding silver nitrate solution to precipitate any chloride present.

5.5.3 Exposure of concrete made with Portland cement to sulphate salts can cause damage due to an expansive reaction between the cement and the sulphate salt to form crystals of ettringite (product of the hydration reaction). Given adequate space to form, the ettringite forms needle like crystals, but in confined space causes an expansive reaction. Due to this expansion cracking, corrosion and deterioration may develop in the concrete structure. Sulphate Testing involves an acid extraction and precipitation of the sulphate as barium sulphate with barium chloride solution. The resulting barium sulphate is filtered and weighed to determine sulphate gravimetrically.

5.5.4 Concrete pH is equally significant because it conditions effective passivation of the reinforcement. In the case of relatively new concrete with correct composition and structure (high pH), the reinforcing steel surface is passivated and effectively protected against corrosion. However, concrete surface pH decreases in time, which makes the concrete gradually lose its protective properties. Simultaneously, the threat of concrete contamination with aggressive chemical agents, increases as they penetrate the concrete near the reinforcement.

5.5.5 The test values, result and interpretation of the chemical test is as follows.

Table 2 Chemical Test results

Sr. No.	Location	pH (Min 11-14)	SO4 (PPM)	SO4 (%) (Less than 4%)	CL (PPM) (Less than 600 ppm)	CL (%)
1	C1-1	11.67	140	0.070	700	0.350
2	C1-3	11.66	144	0.072	720	0.360
3	B2-2	12.15	700	0.350	3300	1.650
4	B2-4	12.17	680	0.340	3280	1.640
5	D2-2	11.80	236	0.118	912	0.456
6	D2-4	11.70	230	0.115	900	0.450
7	A3-1	11.73	190	0.095	750	0.375
8	A3-3	11.71	182	0.091	720	0.360
9	C3-1	11.46	42	0.021	272	0.136
10	C3-3	11.50	46	0.023	276	0.138
11	D4-2	11.42	76	0.038	386	0.193
12	D4-4	11.47	80	0.040	400	0.200
13	A5-1	11.53	130	0.065	576	0.288
14	A5-3	11.70	150	0.075	600	0.300
15	C5-1	11.30	38	0.019	416	0.208
16	C5-3	11.25	48	0.024	444	0.222
17	B6-2	11.65	160	0.080	592	0.296
18	B6-4	11.60	175	0.087	612	0.306
19	D6-2	10.48	16	0.008	44	0.022
20	D6-4	10.45	18	0.009	52	0.026
21	A7-1	11.43	48	0.024	252	0.126
22	A7-3	11.41	50	0.025	248	0.124
23	B8-2	11.16	40	0.020	288	0.144
24	B8-4	11.46	82	0.041	348	0.174
25	D8-2	11.30	72	0.036	328	0.164
26	C7-1	11.60	28	0.014	106	0.053
27	C7-3	11.50	20	0.010	96	0.048
28	A9-3	11.77	242	0.121	960	0.480
29	C9-1999	11.82	232	0.116	928	0.464

5.5.6 As per the theory of concrete, the concrete must be alkaline and its pH ranges from 11 to 14. If it reduces, the reinforcement is susceptible to corrosion. Looking to the values of pH, the concrete seemed highly alkaline side seemingly and hence it is safe.

5.5.7 The values of sulphates should not exceed 4% as per IS456-2000 clause no 8.5.2. Looking to the values of sulphate in concrete, all the values are within permissible limit, hence, it is safe.

5.5.8 The value of chloride in the concrete should not be greater than 600ppm as per IS456-2000 clause no. 8.5.2. Looking to the above values of chloride in concrete, more than 60% samples are safe and show chloride level within permissible values. However, for remaining values, they are at higher side and vulnerable which could further cause corrosion to steel bars. Despite having these higher values, no visual cracking, damage to concrete or any exposed reinforcement was found. To be on safer side, for these foundations the concrete repairing is suggested to strengthen them, and it is recommended that after every 5 years, these foundations should chemically be checked again for further assessment of integrity and effectivity of the repairing work.

5.6 Visual inspection and studying the condition of corrosion, painting condition, sagging of members etc.

5.6.1 Thickness survey of corroded members using ultrasonic Thickness gauge.

5.6.1.1 An ultrasonic thickness gauge is a measuring instrument for non-destructive investigation

of a material's thickness using ultrasonic waves. Ultrasonic thickness gauges are designed to improve safety and ensure reliability of materials which are subjected to corrosion or damage.

5.6.1.2 In this gauge, the thickness of a sample is evaluated using ultrasonic pulse echo method as a product of ultrasonic velocity in the sample and time of travel of waves. The gauge evaluates the time of flight basically and then multiplies it with some value of velocity. A timer or flip-flop circuit measures the time interval between the pulse that triggers the circuit on and the pulse that puts the circuit off. The method and equipment are exhibited below.

5.6.1.3 The recommended values for the thickness of any member are given in Indian Standard 802 (part 1) -1977 clause no. 11.

Ultrasonic thickness gauge survey at Leg of column

5.6.1.4 The specimen of UTG test result is given in the table below.

Ultrasonic thickness gauge survey

Location: 132 kV S.S. (D, E and F) (up to 6 feet height)		**Date of inspection: 14.7.21 to 15.7.21**	
Temperature: Ambient		**Surface Condition: Smooth**	
Reference code: IS 15435-2003 RA 2020		**Couplant: Greece**	
Thickness Tower A3 Leg 1 East Side			
Location	Leg (design thickness 10 mm)	Gusset Plate (design thickness 8 mm)	Lattice (design thickness 5 mm)
1	10.32	8.32	4.32
2	10.21	8.49	4.51
3	10.39	8.72	4.44
4	10.61	8.31	4.61
5	10.57	8.90	4.39
Thickness Tower A3 Leg 1 North Side			
Location	Leg	Gusset Plate	Lattice
1	10.32	8.04	4.35
2	10.51	7.85	4.58
3	10.37	7.23	4.59
Thickness Tower A3 Leg 2 East Side			
Location	Leg	Gusset Plate	Lattice
1	10.58	5.95	4.59
2	10.53	6.70	4.71
3	10.71	6.83	4.78
Thickness Tower A3 Leg 2 South Side			
Location	Leg	Gusset Plate	Lattice
1	10.59	9.05	4.53
2	10.71	8.79	4.71
3	10.83	9.32	4.93
Thickness Tower A3 Leg 3 South Side			
Location	Leg	Gusset Plate	Lattice
1	10.93	6.90	4.78
2	10.81	5.97	4.90
3	10.73	6.73	4.58

Thickness Tower A3 Leg 3 West Side			
Location	Leg	Gusset Plate	Lattice
1	10.71	8.32	4.93
2	10.81	8.07	4.90
3	10.73	8.52	4.78
Thickness Tower A3 Leg 4 West Side			
Location	Leg	Gusset Plate	Lattice
1	10.98	9.05	4.85
2	10.91	9.08	4.93
3	10.97	9.21	4.75

5.6.1.5 Interpretation of the result:

Based on the thickness survey, it is found that thickness of most of the members is as per the actual section of design, barring some legs and lattices, especially near the cooling towers in 132kV (D, E and F station) where thicknesses of the members are less than acceptable. These legs and members of the columns which are in severe condition near the cooling towers are needed to be replaced. Further recommendations are given below.

5.6.2 Inspection of weld joints of bracing, tie and gusset plate, base plate etc. using by DPT test.

5.6.2.1 Liquid (dye) penetrant examination is a non-destructive testing method in which Liquid penetrant (dye) examines welding material flaws which are open to the surface by flowing very thin liquid into the suspected flaw and then drawing the liquid out with a chalk-like developer. This test is widely used for checking welded connection. It can detect any surface discontinuity (or irregularity) such as surface cracks, porosity, pinholes, etc.

5.6.2.2 DPT is based on the principles of capillary action, by virtue of which, liquid tends to flow or seep into any narrow opening, even against external forces like gravity. This phenomenon occurs due to molecular attraction.

5.6.2.3 The surface must be thoroughly cleaned, to make it free from dirt, oil, paint, grease, water, or other contaminants. For cleaning one can use dry cloths, solvents, cleaner, rust remover, etc. depending on the condition of the surface to be inspected. After a thorough cleaning, Penetrant is applied. Penetrant is a red coloured or dark pink coloured Liquid. It can be applied on the surface by spraying (most common) or by brushing or by immersing the entire surface in a penetrant bath (in our examination we have used spray and brush).

5.6.2.4 Leave the penetrant, as it is, on the surface for a minimum period (Known as Dwell Time or Dwell Period). During the dwell period, the penetrant seeps into the flaws (if present on the surface being inspected), **due to Capillary action.** The dwell period varies from 5 minutes to 60 minutes or even more than that depending upon the material and its service condition. In our examination we have considered 10 minutes as a dwell period which is more than reasonable for steel sections.

5.6.2.5 After leaving the surface for the recommended dwell period, the penetrant shall be cleaned, for cleaning the excess penetrant cloth and penetrant removal shall be used.

5.6.2.6 After a thorough cleaning, a thin layer of developer is applied. The developer sucks the trapped penetrant on the surface, from the flaws (if present). That is the penetrant bleeds out on the surface, and it appears in sharp red/dark pink colour. The developer is a white colour liquid and hence the penetrant appears in a sharp red colour, hence we can easily identify the flaws. The process is exhibited in the photograph below.

Tower A3 Dud. Feeder 1

Tower A3 Dud. Feeder 1

5.6.2.7 The DPT test result is given in forthcoming table.

Table 3 DPT test results

Location: 132 kV S.S. (D, E and F) (up to 6 feet height)		Date of inspection: 14.7.21 to 15.7.21
Temperature: Ambient		Surface Condition: Smooth
Reference code: IS 3658-1999 RA2020		Penetrant Application: By Brush
Sr. No.	Job Description	Observation
1	Tower A3 Leg 4 to gusset plate weld joints	No relevant indication observed with respect to area tested.
2	Tower A1, A2, A5, A6, A7, A8, A9 all joints	No relevant indication observed with respect to area tested.
3	Tower B9 Leg 1 & 3, Tower B5 Leg 1-3, Tower B1, B2, B3, B4, B6, B7, B8 all joints	No relevant indication observed with respect to area tested.
4	Tower C1, C2, C3, C4, C5, C6, C7, C8 all joints	No relevant indication observed with respect to area tested.
5	Tower D8 Leg 1-3 to gusset plate weld joints	No relevant indication observed with respect to area tested.
6	Tower D3, D4, D5, D6, D7 all joints	No relevant indication observed with respect to area tested.
7	Tower A4	Crack between gusset plate observed

Figure -2 DPT test

DPT Photographs		
 Column A4 bay D station Generator	 Column A4 bay D station Generator	 Column A4 bay D station Generator
 Column A9 bay IT feeder 2	 Column A9 bay IT feeder 2	 Column B9 bay IT feeder 2
 Column A2 bay DT-1 feeder	 Column B2 bay DT-1 feeder	

Visual Photographs	
 Column D8 bay GEB1	 Column D8 bay GEB1
 Column C8 bay GEB1	 Column C8 bay GEB1
Observation: Heavily corrosion with dis-bonding observed on gusset plat to leg weld joints. Paint peeled off was found on gusset plate, fasteners & members.	

Location: 132 kV S.S. (D, E & F) (above 6 feet height)		**Date of inspection: 11.8.21 to 12.8.21**
Temperature: Ambient		**Surface Condition: Smooth**
Reference code: IS 3658-1999 RA2020		**Penetrant Application: By Brush**
1	**Randomly two legs to gusset plate, Leg to support plate weld joints in each tower of 132kV Airport switchyard (Total 72 nos. weld joints)**	**No relevant indication observed with respect to area tested.**

5.6.2.8 **Interpretation of the result:**

1.. Firstly, Joint with welding is not recommended for transmission tower or gantry structure like this because it forms a rigid joint, and these steel structures are supposed to be flexible. In 132 kV substation (D, E and F), because of this rigid joints and heavy loading, most of welded connections have dislodged form the position and welding are torn off from inside of leg. The gusset plates are buckled at joints. Moreover, it is found during the test that many welded connections have shown cracks and failure (shown above). Thus, it is desirable to have bolted connection for the structure. This is mainly because the joints are welded and thus it is fully restrained. Due to temperature stresses developed in the section which further caused cracking in weld and dislodged of the gusset plate form the leg.

5.6.3 **Condition of Bolted connection**

5.6.3.1 ***132 kV sub-station (D, E and F)*: Some observations of bolted connection are given below.**

a. At lattices intersection and at joints between gusset plate and leg, bolts are missing. Also, these joints are connected by welding instead of bolts.

Figure -3 missing bolts at Column C4 bay D station

b. At some lattice intersections, even the bolt holes are not provided and therefore bolts are not inserted, and the intersection is unfastened. In absence of the intersection bolt the concern bracings will not be able to transfer the load effectively and may be prone to buckling. It is necessary to make the holes and insert the bolts.

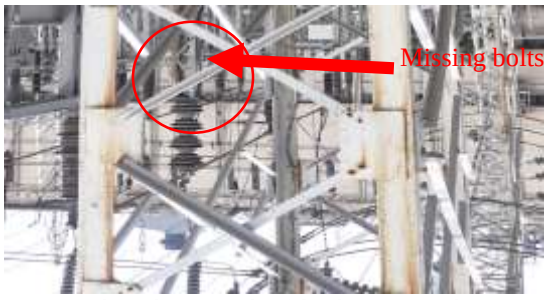
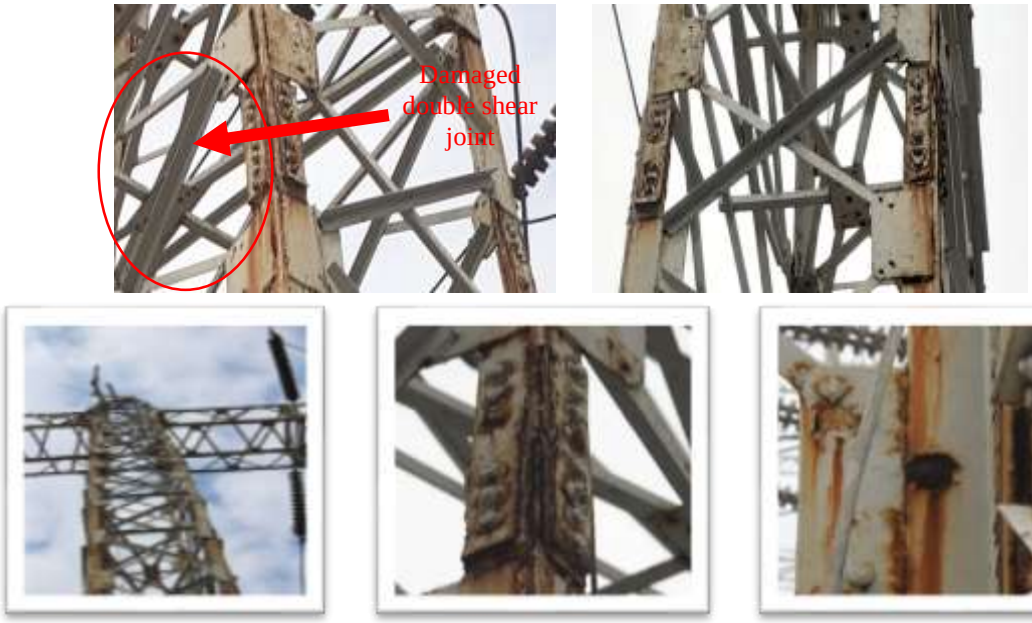


Figure -4 welded connection instead of bolts at Column A2 bay DT-1 feeder

c. The connection between redundant and the main members is done by welding extra plate. This connection will act as a rigid joint, and it may lead to buckling in lattice member under severe load condition.

Figure -5 Joints at Column C9 bay IT feeder 2

d. Some of the bolts in the structure in the vicinity of cooling tower are corroded and needed to be replaced. In some double shear joints, bolts and plates are heavily corroded and deformed. They need to be replaced all together.



Totally damaged tower near cooling station (D8 GEB 2)

Heavy rusting cover plate and double shear joint (A4 D-stn)

Damage in welding inside joint (A4 D-stn)

Figure -6 Heavy rusting in bolted joints at Column A4 bay D station and C8 GT -2

e. Beams in this switchyard are having two types of shortcomings with reference to the bolted connections.
   - Some joints have bolts, but they are heavily rusted and cannot be relied for reliability.
   - Many joints have no bolts at all. This is very serious as the total load of conductor tension I carried by the beam ad missing bolts /members will cause damage.

5.6.3.2 ***132 kV sub-station (Airport)***: Some observations of bolted connection are given below. The bolted connections are in good condition and found in order. All the bolts are galvanized. Although the structure's condition is in order, the are some places in columns and beam where there are no bolts at lattice's intersection point, some bolts are loose and rusted which has been shown in above photographs.

5.6.4 **Condition of Foundation (Anchor) Bolt connection**

5.6.4.1 ***132 kV sub-station (D, E & F)***: Some observations of foundation (anchor) bolted connection are given below.

5.6.4.2 The condition of foundation bolts appears to be good. However, there is rusting on most of them. This may be due to non-use of galvanized anchor bolts coupled with vicinity of cooling towers and deposition of fly ash.

a. During inspection of foundation bolts, some nuts were missing and/or loose. This is undesirable. It is necessary that nuts may be provided along with plane washers and tightened after proper servicing of bolt-nut threads.

Figure 7 Missing nuts in foundation bolts in Column B4, B9 and C9

5.6.4.3 ***132 kV sub-station (Airport)***: Some observations of bolted connection are given below.

a. Some foundation bolts are embedded in concrete along with the base plate and are not visible. So far as the foundation bolts which are protruding above the concrete are concerned, their condition is very good, and no rectification is required.

Figure 8 foundations in 132kV (Airport)

6.0 Summary and Recommendations

6.1 Rebound hammer test performed on various foundations indicates that the compressive strength of most of the foundations is more than required. However, in the case of column B-6 and C-7, compressive strength of the foundation appears to be unsatisfactory. Hence, these results can be ignored.

6.1.1 Chemical tests performed on various foundations indicate that the results are in line with the requirement. However, the results of some foundations (of columns B2, C1, C2, A3, B6, A9, and C7) appear to be unsatisfactory. These results can be ignored, terming them as a local effect.

6.1.2 Examination of concrete work after removal of soil around it, has revealed that the structural concrete is smooth from all the sides without any honeycomb or protruding reinforcement bar.

6.1.3 Most of the joints of the gantry structure are welded and hence they have become rigid and susceptible to temperature stresses. The deformation of large number of gusset plates, cover plates and angle members is testimony to it. The Indian standard recommends bolted connection (i.e., flexible connection) for the transmission line towers and the structures like gantry and equipment support. Due to this rigid connection, many gusset plates have deformed, and welding joints have peeled off from the connection. There are two alternatives to repair these: (1) to reweld the connection or (2) to make fresh connection by nuts and bolts. Additional gussets for bolting can be provided.

6.1.4 At many lattices cross joint connection, the bolts are missing which have affected the slenderness ratio of lattices. Thus, these lattices have crippled which may further lead to structure collapse over a period of time (particularly under heavy storm). Also, due to excess loading, few members of the beam are crippled. These all members are needed to be replaced with new members of same size and connected with bolts only.

6.1.5 The gusset plates have crippled / deformed due to uneven loading and temperature stresses (due to rigid connection). There is severe rusting at number of joints too. All these gusset plates are required to be replaced with new ones having same properties and connecting them by bolts.

6.1.6 Few lattices are missing which are required to be replaced with new members. Failing this there are chances of catastrophic failure under stringent weather conditions and/or short circuit conditions.

6.1.7 At some locations, it is found that bolt holes are not matching with each other, and it is connected with weld. At these places, bolt holes are required to be drilled at site and connected with new bolts.

6.1.8 Few towers and beams near cooling station are heavily rusted/pitted and are in grave condition. It is strongly recommended to replace such towers and beams with new ones.

6.1.9 In ICT-1 bay and Dudheshwar bay, the main angle members of two beams are crippled which caused deflection in whole beam. Moreover, in D-station Generator Bay, diamond pattern in the body of the beam at the conductor hanging/fixing position is damaged. Thus, these members are to be replaced. Failing this there are chances of damage to the beam under stringent weather conditions and/or short circuit conditions.

6.1.10 So far as the thickness survey through Ultrasonic thickness gauge is concerned, the thicknesses of few members in some towers are doubtful. Hence those members are needed to be replaced. However, there is no immediate threat.

6.1.11 Almost all the structures require removal of corrosion and repaint. For this the existing paints need to be removed with appropriate means, and then new oil paint with primer is required to be applied. Use of zinc rich paint will give better longevity.

6.1.12 Many foundations are having cracks in the top portion of the column. However, they are not extending below the ground and can be termed as plaster cracks. Resurfacing and plastering is suggested. There is no threat to the stability of foundations.

6.1.13 Nuts are missing on few anchor bolts. New nuts are to be fixed. Also, installation of lock nut should be done.

6.1.14 It is observed that the ground wire is missing in almost entire substation. The installation is of new ground wire is recommended for providing direct stroke lightening protection to equipment and conductors in the switch yard.

7.0 Refurbishment plan:

7.1 5 years durability plan for the existing structure

7.1.1 In 132kV substation (Airport), all the missing and crippled members are required to be replaced with new members of same size. Also, new bolts with proper grip length (with pack washers and spring washers) are required to be installed in all concerned junctions. If bolt holes are not matching, the new holes will have to be drilled to fix the bolt.

7.1.2 In 132kV substation (D, E and F), as per the recommendation given above, some new columns and beams will have to be erected with fresh bolted joints, members, and gusset plates. To add to this, existing missing or rusted bolted connections will have to be replaced with new nut-bolts with pack washers and spring washers. Existing crippled and missing lattices will also have to be replaced. New nuts are to be provided at anchor bolt where nuts are missing.

7.1.3 For existing rusted and damaged welded joints, new bolted gusset plates should be provided as a temporary strengthening measure.

7.1.4 The existing paints need to be removed with appropriate means, and then new oil paint with primer is required to be applied. Use of zinc rich paint will give better longevity.

7.1.5 Re-plastering is required to be done above ground level where existing plaster is damaged.

7.1.6 BOQ and estimated cost of 5 years' durability is attached herewith as annexure 3.

7.2 10 years durability plan for the existing structure

7.2.1 In 132kV substation (Airport), all the missing and crippled members are required to be replaced with new members of same size. Also, new bolts with proper grip length (with pack washers and spring washers) are required to be installed in all concerned junctions. If bolt holes are not matching, the new holes will have to be drilled to fix the bolt.

7.2.2 In 132kV substation (D, E and F), as per the recommendation given above, some new columns and beams will have to be erected with fresh bolted joints, members, and gusset plates. To add to this, existing missing or rusted bolted connections will have to be replaced with new nut-bolts with pack washers and spring washers. Existing crippled and missing lattices will also have to be replaced. New nuts are to be provided at anchor bolt where nuts are missing.

7.2.3 All the joints must be converted to bolted connection for the flexibility of the structure. Thus, not only existing rusted and damaged welded joints, but also other bolted connection in good condition, will have to be connected with new gusset plates and nut-bolts with proper grip length. (With pack washers and spring washers)

7.2.4 The existing paints need to be removed with appropriate means, and then new oil paint with primer is required to be applied. Use of zinc rich paint will give better longevity.

7.2.5 All the Double shear joints are to be replaced with new cover plates and new nut bolts.

7.2.6 Re-plastering is required to be done to all foundations above ground level.

7.2.7 Column foundation repairing work is required to strengthen, where weak compressive strength is observed in rebound hammer test and chemical test.

7.2.8 BOQ and estimated cost of 10 years' durability is attached herewith as annexure 4.

7.3 More than 10 years durability plan for the existing structure

7.3.1 132kV substation (Airport): These structures are recently erected and appear to be in good condition despite having few shortcomings like missing members and bolts. Installation of new members in place of missing ones and all bolts with proper grip length (with pack washers and spring washers) will have to be done. If bolt holes are not matching, the new holes will have to be drilled to fix the bolts in proper position. In addition, new zinc rich paint coating is suggested to make this structure more durable.

7.3.2 132kV substation (D, E and F): These structures are very old (built in 1974 according to TPL) and any plan or structural drawing is not available. Moreover, maximum refurbishment is required to strengthen this. In visual inspection many columns and beams are damaged and are on the verge of collapse. Many members are heavily corroded, and joints are rusted and peeled off. Thus, for more durability, the gantry structure must be constructed a fresh with latest codes of practice for the design of switchyard structures so that new designed structure can be erected to make this structure durable for more time-period. Since foundations are found to be robust, they need not to be redesigned or cast again. However, validation of foundation design may be needed as the structure will be redesigned as per the latest Indian standards resulting in to increase in foundation reactions. As such no BOQ or estimation can be done as of now.

8.0 Conclusion:

8.1 The NDT carried out on foundation showed reasonably good results. Some minor rectification is sufficient.

8.2 The foundation bolts are generally in order. However, at some places rectification is required.

8.3 The condition of steel structures is alarming and immediate steps are required

About Author

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Born on 30th March 1948 and graduated in Electrical Engineering from the MS University of Baroda, Vadodara in 1971. He has more than 40 years' experience in Transmission and Distribution of Power as well as Hydro Power. He retired from Gujarat Energy Transmission Corporation in the cadre of Chief Engineer in March 2006. Presently he is a Managing Director of consultancy firm named "Takalkar Power Engineers & Consultants Pvt. Ltd." in Vadodara. The firm is engaged in Design, Engineering and Construction Supervision of Transmission lines and Substation up to 765kV. The firm is also actively involved in industrial electrical design and hydro power designs.

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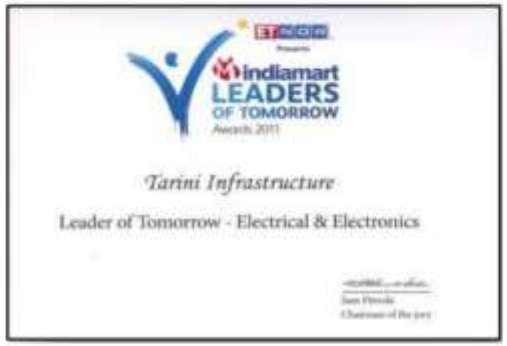

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Benefits of Exposure to the Practices used in International Transmission Line Projects

Dr. Deepak Lakhapati
Founder CEO Transs Energy

A. INTRODUCTION

The Author has been working in the power transmission line industry for 49 years. He has been instrumental in revolutionizing the Design, Engineering & Construction practices in India and many other countries. The case studies given below are few of his challenging assignments globally. Each case study is concluded by the takeaways. Every assignment during last 49 years has been challenging and time bound which was executed to the entire satisfaction of clients and has been included in his book titled "POWERING THE WORLD".

B. Restoration of Transmission Network in Iraq

Post the Iraq war in 2003, most of the electricity network in the country was destroyed. There was an immediate need for rehabilitation of Iraq's power infrastructure. The winter was almost there, while Southern Iraq stood in darkness.

The US Army outsourced the project of rebuilding the Transmission line and Substations to M/s Perini, USA, 360 kMs long, 400kV S/C line with 750 lattice steel towers from Al Hartha to Al Kut in Southern Iraq including new substation. The entire contract for security was awarded to M/s Hart Security Company. M/s Power Engineers, USA were the engineering consultants for the project.

M/s Perini, an American company, contacted KEC International as EPC contractors to rebuild the 400kV transmission line. They gave us some preliminary assessment data. There were however no drawings and all the towers had been destroyed in the war. The substation was also burnt to ashes.

I was one of the first to be deputed to Iraq for the detailed assessment of the condition of Iraq. Inspection of every location was completed, location wise BOQ was generated with few surveyors and the field staff. After the formal introduction, I started my first site assessment visit.

Figure-1: Orientation Map of Iraq

We were escorted in a convoy of 5 cars with 5 guards in each car with assault rifles and safety gears ready with satellite phones for communication. Each car had a neon flag on the roof, which the helicopters could watch and guide for any possible danger on the route. I was asked to wear a bullet proof vest and sit in the third car, in the middle of the back seat, well surrounded by the guards.

Figure-2: Our Team going for Assessment of every location along with security guards

As we started for our site visit, I could look at the surrounding area which was quite horrifying. War was over and all the burnt military tanks,

Figure 3: Vehicles used for site visit

Figure 4: Military Tanks & Land Mines

vehicles, shells, rockets were all around. Our route was pre-decided after clearing all the land mines in the route. When we reached our first tower location, I saw that the tower was completely damaged, including the foundation. I was warned to not to go outside the security ring of the armed guards, and take all the required measurements, back-to-back base dimensions, angle flanges, thickness, pictures of all insulators, hardware fittings and any relevant information for the as-is assessment of the tower location. Completing the first day of the visit, I returned to the camp and reported the progress at 6 pm with an updated 3 days rolling plan.

Figure 5: Damaged Transmission Line Tower

Since we did not receive any drawings of the old towers, we resorted to “Reverse Engineering“ by measuring the dimensions of foundations, stub, tower parts etc. We created structural drawings and made a full-scale prototype in our factory in India.

Towers were then mass produced and airlifted to Basrah Airport. Conductor drums were airlifted from Bahrain and Insulators were airlifted from China.

Meanwhile, a team of thousand labourers, climbed aboard the plane to Basrah to head out for the mission.

Figure 6: Tower Parts Air lifted from India on the Basrah Airport

Figure 7: Air craft used for airlifting tower parts, conductors, Insulators, heavy

Two working camps / stores were created so that the work could simultaneously start from both ends of the line. The entire rebuilding plan was ready with work instructions and working drawings of foundations, tower erections, insulator hoisting, stringing, final sagging, and clipping.

The transmission line work started in full swing, and materials were lined up, foundation repair works were completed, towers were erected

manually, stringing machines were also in place for stringing through cut points. Local authorities would also visit the sites regularly. We used to carry cookies for the curious local children who would visit our sites.

Figure 8: Inspection of damaged tower during an assessment study

"Take away from this project was a proof that nothing is impossible. Working in War-torn countries in most risky situation. Without the availability of tower drawings, reverse engineering was done by the actual measurement of tower members. All materials were airlifted from country of manufacturing. Secured camps were created. Entire workforce was split for both sided construction activities. Line was commissioned in a record period of 4 months. Our mind-set is that the transmission line construction cannot be done in such a brief time, but with dedicated team of people, it was made possible."

Figure 9: Fully damaged sub-station during the war

Figure 90: Final restoration work in progress

Figure 11: Local girl and her brother liked to visit site for Cookies & Football

C. Transmission line construction in war torn Afghanistan

As planned, we had a set out for the meeting with consulting engineers in their office at 9:30 AM. On our way as we were about to reach the central market in Kabul, our car was stopped by the security forces and told to go back to our hotel. There had been a bomb blast on the previous night, and the whole area was under curfew.

There was a huge mountain, at the foot of which the substation was planned to be constructed. Transmission lines were supposed to be drawn straight from the mountain into the city, which wasn't a hassle. However, during the execution of the project, the terms changed. We had to now set up the substation on the other side of the mountain and cross the Transmission Lines from over the top of the mountain. The reason to shift the substation location to the other side of the hill was to facilitate the future planning of transmission lines for the evacuation of power.

Technical challenges increased multi-fold. We surveyed the mountains for a few days. We had

to cross a mountain that was too high, and a few kilometers wide. We found a small gorge that was crossing through the mountain. It was a very narrow road alongside a flowing stream of water. We were afraid that if we installed towers there, the conductor would have touched at least one side of the hill. This could've been a technical hazard. To understand the situation better, we did a 3- dimensional mapping of the place and took the data to India to study it better using PLS-CADD software.

Figure 12: View of the Tosh Gorhan gorge through which TL had to pass

The engineering team had several discussions and ultimately decided that it would best to eliminate the cross arms since there was no space for it. We conceptualized the design with conductors directly attaching to the body of the towers. However, the height wasn't as big of an issue as the width of the towers. The project was funded by ADB for the construction of 220kV double circuit line during the year 2010.

We decided to go ahead for the first time, for a vertical configuration with six conductor attachments directly to the body of the tower for a double circuit tower and a jumper arrangement.

Once the towers were designed, it was time to


Figure 13: View of Narrow Road and water flowing canal where foundations had to be made

set up the foundations for the project. This became another challenge for us as what we thought was just a small stream flowing by the road, turned out to be nine feet in depth. Since there was no space available on the road for any suitable foundation, it was decided to look for the small space in the water body. A single raft foundation was designed to manage the space available. All the four legs of the tower structure were embedded in the block foundation with reinforcement.


Figure 14: Block foundation covering all four legs of tower

Figure 15: Block Foundation in progress along with Template

Figure 16: Deep excavated foundation

Figure 17: Finally executed executed transmission line along with the road

"Take away from the project is to think Out of Box whenever a situation comes where conventional approach do not work. Above unique solution received an international design award". The concept was then implemented in India for the first time on 765 kV S/c line on a delta configuration. Top phase conductor was attached to the body of the tower."

D. Transmission line construction in cryogenic temperatures of Canada.

Working in negative temperature (-40^{0}C) is a unique challenge. Entire right of way is first purchased from the landowners after various court hearing. Trees are cut, cleared and in the meantime all towers are assembled in gas heated sheds. All foundation are steel grillages, and towers are guyed structures. Mobile workers camps are created. Once the snow sets in, all heavy machines are moved in to start the construction. Only six months period is available to finish the work at site.

Figure 18: Tree cutting & cleaning in transmission line corridor

Figure 19: Heavy Snow causing difficulties in moving of heavy vehicles

Figure 100: Workers mobile camp

Figure 21: Steel grillage lowered in foundation pit.

Figure 22: Tower assembly in gas heated tents and assembled structures at site

"Takeaway for Canda project is an experience to work under cryogenic temperatures using gas heated tents and assembly of guyed towers erected by cranes in working period of 6 months.

Leveraged prefabricated grillage foundation as traditional concrete foundation are not suitable."

E. Transmission line construction using prefabricated towers and foundations in Kazakhstan

We ordered manufacturing companies for the supply of towers, foundations were all prefabricated, all guy wires were typically cut to size with end attachments, and all insulators and hardware fittings were in place. Local Construction contractors were to execute the field work.

After the winter was over, the construction season started with the target to complete before the end of summer. So, the working period was only six months.

The assembled towers, foundations, guy wires, turnbuckle and all fixtures were delivered at tower locations. Cranes came into the location and the erection was completed in only a few hours.

The way that Kazaks constructed their towers was one of the best things that I had witnessed in my years of service in the industry.

A stretch is taken at a time, and a set number of construction workers are allotted for it. They precisely take 21 days to construct the whole stretch. A team is designated only to look after setting up camps and take care of the necessities that a person would require while working. Once they complete the stretch in 21 days, this team relocates the camps and necessities to the next stretch. This relocation is conducted in a period of 9 days. The construc-- ction workers would get a break for these 9 days.

This whole procedure speeds up the construction multi-fold. Our project was carried out in 6 months, as compared to 2-3 years elsewhere.

Figure 23: 500kV Single Circuit double mast guyed tower under construction

Workers were allotted for stretches of 200kM each. Even during the bidding process, we were in India, estimating timeline in years and months. On the other hand, they were anticipating days and weeks, despite having a much challenging terrain.

Figure 24: Form-Box for pre-cast foundation (base & chimney)

Figure 25: Reinforcement used in the base and chimney of foundation

Workers were allotted for stretches of 200kM each. Even during the bidding process, we were in India, estimating timeline in years and months. On the other hand, they were anticipating days and weeks, despite having a much challenging terrain.

This first project that we had undertaken was completed successfully and got us best experience working in low temperatures.

In the next project, we bid smarter since we were now well versed with the costs that we could incur when purchasing the designs and subcontracting manufacturing and construction.

They also wanted a knowledge transfer as most of the population in the utility was either too old, i.e., 70 and above, or were too young. Hence, it came up on KEC's shoulders to train the young in the utility.

I was supported by talented young girl from KEC Kazakhstan, who assisted me by translating my lectures. She was fluent in English, Russian and Kazakh. I prepared my lecture notes and presentations in four modules in English, which were then translated into Kazakh.

Figure 26 Electric well for setting concrete in precast foundation

Figure 27: Mass production of precast foundations in plant

Figure 28: Final precast foundations are ready

We travelled to the head office of KECOG in Astana, where the training hall was reserved for us. Training was slow since my speech had to be translated online and any questions had to be answered. By the end of our training period, I was friends with everyone.

"***Take away from this project was a unique experience of use of pre-cast foundations. As we mass produce guyed towers, smaller in weight, we can also mass produce pre-cast foundations, Vertical hot dip galvanizing was a new concept.***

This is where I felt that India still has a long way to go when it came to execution of infrastructural projects in shorter time frames."

F: Conclusion

While Indian Power sector has done remarkable progress in terms of Power Generation capacities during last five decades along with Design to execution of multiple thousands of kMs of transmission lines from 132kV to 1,000kV that I have witnessed. With a tremendous push on the renewable power which takes much shorter time to build, evacuation can become a bottleneck unless the construction of trans-mission lines is also expedited to match with

About Author

Dr. Deepak Lakhapati

Excellent academic achievements, BE (Civil) Gold Medalist, ME, Ph.D. degree holder. Trans-mission & Distribution industry expert with 49 years. of experience in providing innovative technology solutions.

Worked with EPC companies (ABB-SAE, KEC, GAMMON, Jyoti Americas) in Transmission and Distribution projects in 45 + countries in the world and with a transmission line developer company since last 7 years. Member of CIGRE, Paris and IEEE, USA, Member of National committees. On the panel of expert committee formed by CEA (Ministry of power, Govt. of India) to advice on tower failures and solutions.

Civil & Structural Engineering for establishing Solar Projects by Gujarat State Electricity Corporation Limited

SS Sheth

1.0 INTODUCTION

1.1. Harnessing Green Energy is now very common world over. The solar and wind energy are most preferred form of Green & renewable energy.

1.2. For establishment a solar power plant (say of 100MW) lot of activities related to civil engineering are required to be taken up.

1.3. GoI has given very stringent targets to all the stake holders in solar power. Gujarat State Electricity Corporation Limited (GSECL) has also given target for larger installed capacity.

1.4. The presentation here under gives brief account of the activities involved in establishing a major solar power project.

2.0 ACTIVITIES INVOLVED

2.1. The establishment of Solar projects on Government waste land includes following major events:

- Identification of Government waste land in various districts and application for acquisition.
- Follow up with Revenue department officials and joint measurements/ visits at site till land order is issued.
- Taking over possession of land.
- Feasibility survey.
- Preparation of EPC tender, invitation of tender and finalization of Work Order to successful bidder.
- Initial surveys like topography, soil investigation and hydrology by the EPC contractor.
- Preparation of drawings and design by the EPC contractor, submission and approval process.
- Execution of works at site and supervision.
- Resolution of site issues and local issues.
- Acceptance of works completed and Finalization of works and supply contract.

Role of Civil engineers and measures to be taken are narrated as under for above 10 points:

3.0 IDENTIFICATION OF GOVERNMENT WASTE LAND IN VARIOUS DISTRICTS AND APPLICATION FOR ACQUISITION:

3.1. This is an important initiative and very cumbersome activity in some cases. Waste land is identified by visiting Mamlatdar office of concerned taluka and after due verification of records and sometimes site visit if the land is suspected to have conditions like encroachment etc. which are beyond resolution. Thereafter, an application is submitted to Hon Collector of the district under the provisions of GR referred in first para. The land acquisition is very lengthy process, involving many steps and many authorities from Hon. Collector, District Inspector of Land Records to Sarpanch and leading residents of the adjoining villages. The land such acquired will enable execution of Solar projects at comparatively low cost, as the land is at lease with a token rent of Rs 1=00 per Ha for a period of 30 years, i.e. execution time and life span of the Solar project. However, if the company prefers to be Project Developer and allots land to various developers, then the rent will be Rs 15,000=00 per Ha per year. Requirement of land is estimated @ 1.85 Ha per MW and installation cost is 4 to 5 Cr per MW.

4.0 FOLLOW UP WITH REVENUE DEPARTMENT OFFICIALS AND JOINT MEASUREMENTS/ VISITS AT SITE:

4.1. After application of waste land, till issuance of land order and possession, many steps/ procedures are involved. Initially, a pre JMS (Joint Measurement Survey) is arranged by

DILR. Thereafter, the mapni sheet (land measurement sheet with co- ordinates) is routed through Mamlatdar and other authorities as applicable as Forest, Mining etc. to ensure the land is encumbrance free. Intermediate follow up with various Government authorities at personal level clears the status of land acquisition.

4.2. During JMS (Joint Measurement Survey) and other site visits, site needs to be verified in respect to accessibility, profile, availability of water bodies and religious places within the earmarked land, visual soil assessment, village roads passing through the applied land, existence of cremation place due to non-use of land since long etc. These aspects if missed during the initial stage of land acquisition will raise various issues with mass or at individual levels immediately after start of actual work at site; hence identification of such issues before allotment of land order or before possession, is highly essential. It need to be ensured that the land is totally unencumbered.

5.0 TAKING OVER POSSESSION OF LAND:

5.1. After completion of all procedures at various levels and with various authorities, land order is issued by Hon Collector of the district and possession is transferred on lease basis. Before issuance of land orders, now a days a trend is being established to refer the JMS sheet to Forest department to ensure there is no right of Forest department over the land being asked by the company.

5.2. Land order is containing general conditions such as to maintain the village roads, water bodies, shifting of cremation within defined distance etc. Various fees, as applicable from time to time need to be paid to the respective departments by challan.

6.0 FEASIBILITY SURVEY:

6.1. At any stage during land acquisition, if it is found that installation of 10MW or more capacity is not feasible, the application is cancelled or the land is surrendered even after land order is issued. If the land is primarily suitable for installation of Solar projects, possession is accepted and a feasibility survey is carried out to ensure the potential of the site with respect to capacity installation.

6.2. Use of latest technologies is being made here. From the co-ordinates of the land, a KMZ file is prepared which shows outline of the allotted land on Google map. Even while moving to the site, it indicates the location of cell phone and this makes it possible precise observation within the land area allotted with boundaries. With the use of software, available area and other relevant details like Solar radiation, shadow analysis, various financial analysis etc. are finalized by the consultant and probable capacity that can be installed is decided. Also, evacuation is decided during this stage. This serves as a primary base line for finalization of EPC tender.

Specimen Feasibility survey contents

KMZ File

PREPARATION OF EPC TENDER, INVITATION OF TENDER AND FINALIZATION OF WORK ORDER TO SUCCESSFUL BIDDER:

6.3. After feasibility survey, tender is prepared for the proposed Solar project on EPC basis, with defined MW capacity within the allotted land and with due consideration of situations in existence. From civil point of view, it is specified to leave the water bodies and roads in use within the plot. Boundary/ fencing and internal roads, drains etc. needs to be proposed accordingly. Work order/ Purchase order is issued to successful bidder after observing all formalities.

7.0 INITIAL SURVEYS LIKE TOPOGRAPHY, SOIL INVESTIGATION AND HYDROLOGY BY THE EPC CONTRACTOR:

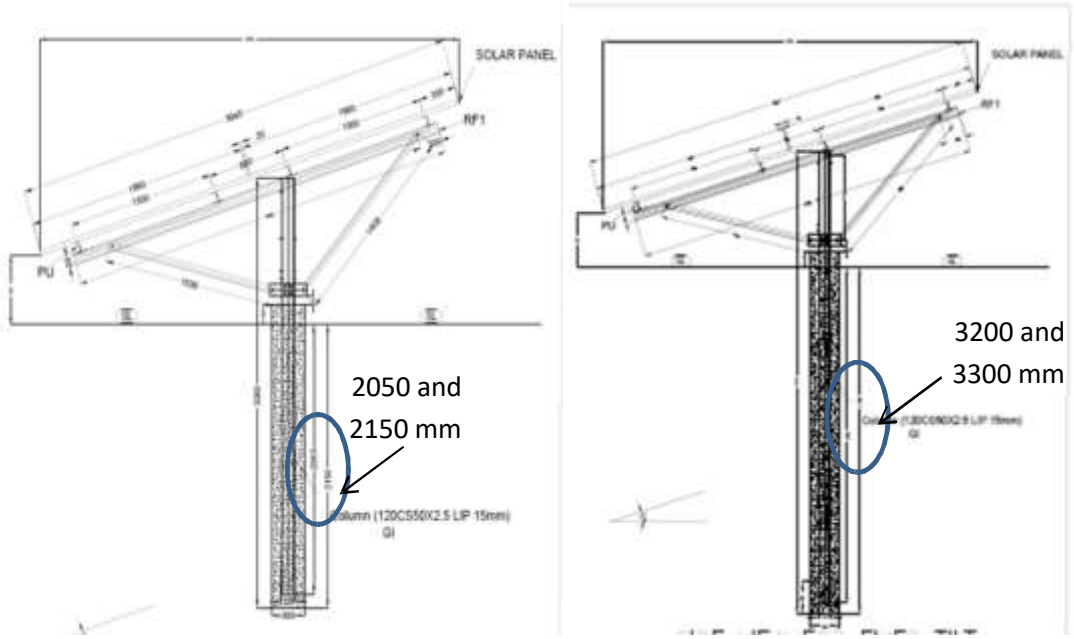
7.1. On issuance of work order, initial survey like contour survey, soil investigation, hydrology study, concrete mix design etc. are being carried out by the EPC contractor. This forms base of design and engineering of the project including PV array layout, drainage system for the area, internal roads etc.

7.2. If there is a major difference in tendered quantity and that surveyed by the EPC contractor appropriate corrections are required in the contract documents.

8.0 PREPARATION OF DRAWINGS AND DESIGN BY THE EPC CONTRACTOR, SUBMISSION AND APPROVAL PROCESS:

8.1. The Government waste land allotted in most of the cases is having various challenges like low permeability, soil with poor SBC (due to less cohesion), highly undulated profile, sandy soil, liquefaction etc. Designing of foundation remains a challenge up to certain extent. Few cases regarding such challenges are narrated as under.

8.2. In one of the Solar projects of 200 MW (02 plots of @ 185 Ha each for 100 MW), situated in Banaskantha; the soil is sandy with very low contents of clay and poor SBC. Due to this reason, piles for supporting MMS are designed with 300 mm dia and 2150 mm to 3300 mm length with embedment of channel section 120 x 50 x 2.50 mm for 2050 mm and 3200 mm. This is comparatively higher side than normal soil conditions and impacting the cost of the project.



8.3. Also, the soil is having low permeability and saline water is available @ 1M below NGL. Here, the land is reasonably levelled with moderate slope. During rainy season, rainwater is accumulated and slowly disposed towards natural gradient. Due to moderate slope and availability of ground water @ 1M below NGL, design of internal drain was not possible with

self-cleaning/ flowing velocity. Alternatively, it was planned to construct sumps at low lying areas to collect the rainwater accumulated and pumping was arranged to dispose the accumulated rainwater. However, against the intake of rainwater, this arrangement failed to give satisfactory results and natural drainage was implemented by construction of culverts with RCC Hume pipes at low lying spots on peripheral bunds; and it is found that this unorthodox method, is effective compared to conventional methods of drain construction.

8.4. In general, following are civil components for Solar project:

8.4.1. Boundary works: Generally, precast compound wall up to 1800 mm height is preferred for Solar projects. However, in case of Khavda Solar park at Kutch, fencing is provided with posts having 65 micron zinc coating and PVC coated fencing (3mm wire zinc coated and 0.5 mm PVC coating) to avoid rusting due to saline atmosphere in longer run.

8.4.2. MMS piling: Piles are now days preferred by EPC contractors for module mounting structures. In case of saline atmosphere and salinity in soil like Khavda Solar Park at Kutch (total capacity 30 GW, GSECL share 3325 MW), Sulphate resistant cement is used/ suggested for all works, below and above GL to avoid accelerated deterioration of the works.

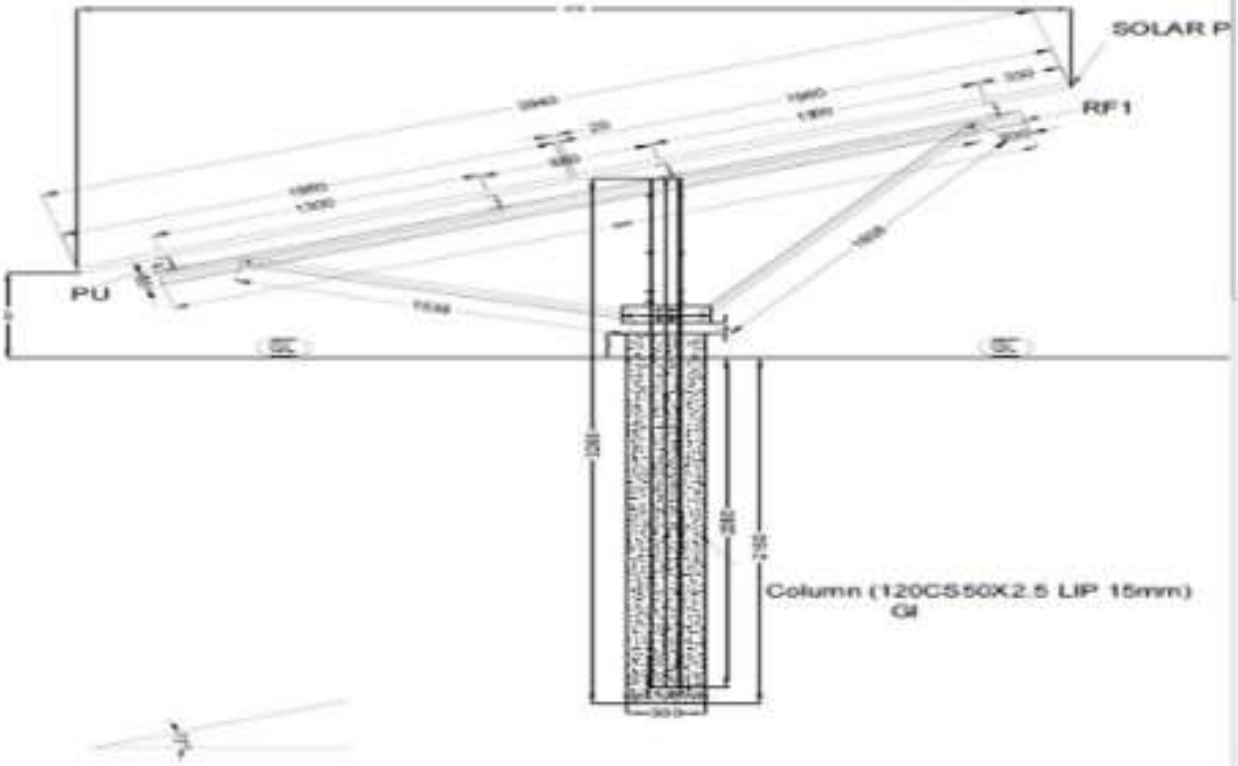
Height of pile above GL is decided in a way that the panel will not submerge during heavy rain and accumulation of rain water for the time being.

8.4.3. Drainage: Conventional drainage system is arranged in most of the projects, keeping in view natural profile revealed after topography survey in the initial stages. However, unorthodox methods are also implemented as mentioned in case of 200MW project at Banaskantha as mentioned above. In case of Khavda Solar Project, area is very wide and soil is low permeable; hence drainage of 6.0 m to 12.0 m wide is proposed and it is also planned to collect the rainwater at sumps proposed on lower elevation for utilizing in panel washing and such works since the ground water is saline requiring desalination.

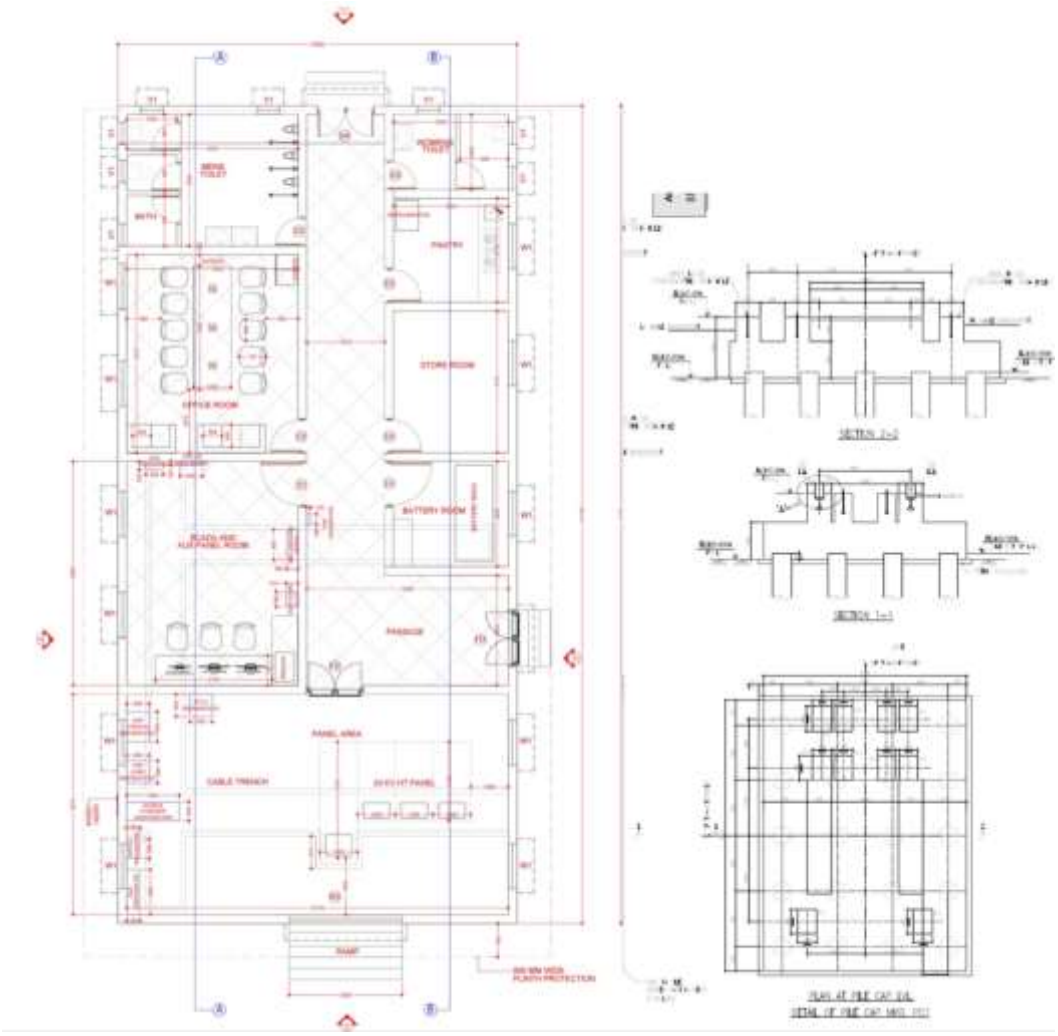
8.4.4. Internal roads: Generally WBM roads are constructed to suit with the maintenance activities within the project area.

8.4.5. Main Control Room: It is a building having battery room, panel room, cable galleries, office for engineers and other amenities like sanitation, pantry etc. mostly, it is a building with ground floor only.

Typical layout of MCR is shown below.



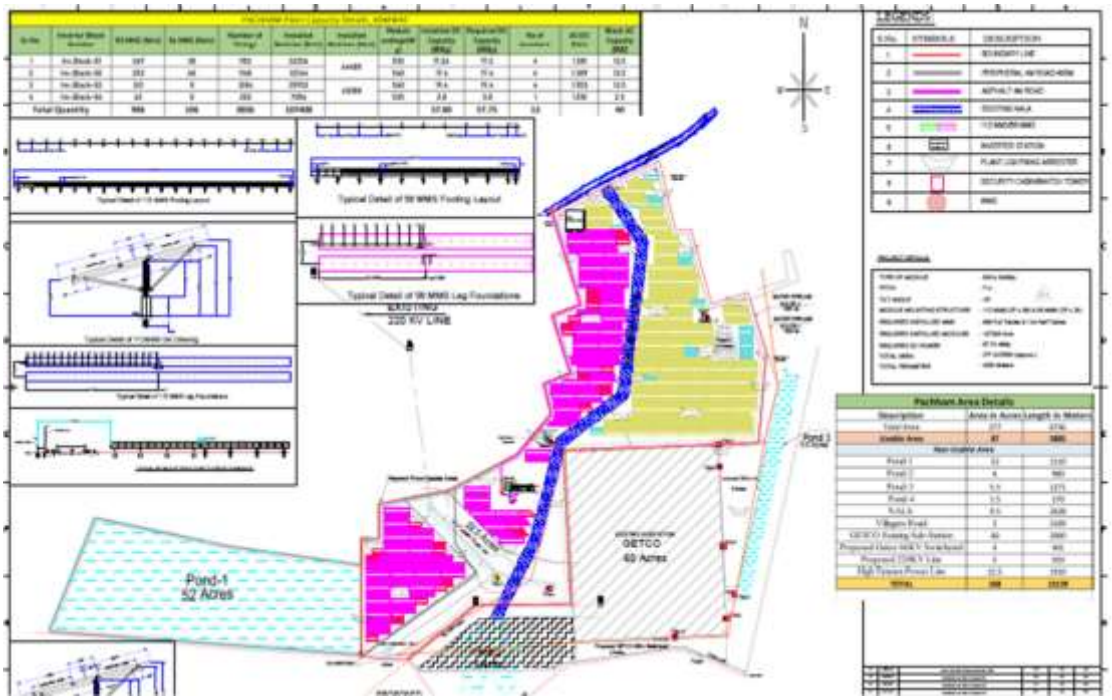
SIDE VIEW FOR FIXED TILT
PV MODULE 1960x992x35
ECONESS



Plan of Main Control Room Invertor Resting Platforms

Invertor resting platforms: Pile cap is constructed above piles for resting Invertor.

Layout for a Solar project of 40 MW is shown here for reference:



9.0 Execution of works at site and supervision:

9.1. The Solar projects are executed on EPC basis and necessary checks, specifically in material approval and invoicing is exercised by GSECL through engineers looking after the project.

10.0 Resolution of site issues and local issues:

10.1. Government waste land, being barren, local issue is being encountered frequently. GSECL officials are coordinating with appropriate authorities up to Hon Collector level. Regarding encroachment in project area, mostly it is resolved by discussions and sometimes police protection is asked by the agency to stop hindrances in execution of the work. GSECL provides support as and when required to resolve the issues at an early date.

11.0 Acceptance of works completed and Finalization of works and supply contract:

11.1. Since the project is carried out on EPC basis and due to prevailing conditions, it is not possible to carry out 100% supervision of the works during execution. Before acceptance of the civil works, over and above visual check, necessary tests are performed like pile integrity test, cube tests in Government approved laboratory for RCC works etc. and a joint protocol is prepared for handing over. All documents collected like material approval, test certificates etc. are compiled and then the work is finalized.

12.0 CONCLUSION

12.1. The construction of solar power project of 10MW and above need many activities related to civil engineering. They include preconstruction activities such as survey, estimation, tenderisation, award of EPC contract etc.

12.2. The execution of work by EPC contractor also involve many civil engineering activities by the utility. This includes supervision, testing, recording of measurements etc.

About Author

Er. SS Sheth, working as SE (Civil) at Gandhinagar TPS since Nov 2022. He started his professional career with M/s Simplex Concrete Piles (I) Ltd. in Dec-1991 and in the year 1994, he joined GEB as Junior Engineer (Civil) at Kadana HEP. Thereafter, he worked at various Thermal Power Stations of GSECL (GEB) viz. Wanakbori, Sikka, Ukai and at Corporate Office of GSECL in different capacities. Most of the time of his career was spent in construction of new units / large structures related to Power Plant. Since 2021, he is involved in Solar Projects by GSECL on Government waste lands in various districts of Gujarat. Construction of major structures of Power Plant is area of his interest.

Significance of Electrical Engineering for Designing of Substation by Civil / Structural Designer

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1.0 INTRODUCTION

This paper provides information to civil design engineers working in/for power sector, regarding importance of interface required with electrical engineer. It also includes basic functional and operational details required particularly for design of civil engineering structure in EHV substations.

A substation is a part of an electrical generation, transmission, and distribution system. Substations transform voltage from high to low, or the reverse, or perform any of several other important functions. Between the generating station and consumer, electric power may flow through several substations at different voltage levels.

A substation may include transformers to change voltage levels between high transmission voltages and lower distribution voltages, or at the interconnection of two different transmission voltages. The word substation comes from the days before the distribution system became a grid.

The main issues facing a power engineer are reliability and cost. A good design attempts to strike a balance between these two, to achieve reliability without excessive cost. The design should also allow expansion of the station, when required.

Selection of the location for Air Insulated Substation (AIS) must consider many factors. Sufficient land area is required for installation of equipment with necessary clearances for electrical safety and for access to maintain large apparatus such as transformers. Wherever land is costly, such as in urban areas, Gas Insulated Switchgear may be viable alternative. The site must have room for expansion due to load growth or planned transmission additions. Environmental effects of the substation must be considered, which may include drainage, noise and road traffic effects. The total ground potential rise, and the gradients in potential during a fault (called "touch" and "step" potentials), must be calculated to protect personnel during a short-circuit in the transmission system. The substation site must be reasonably central to the distribution area to be served. The site must be secured from intrusion by public, to protect people from injury by electric shock or flashover. The site should not be prone to flooding and high water table.

2.0 Importance of interface between Civil Engineer and Electrical Engineer

- Civil Engineering is the foundation stone of any construction activities, and they must know various electrical aspects while carrying out the civil design.
- Civil as well as Electrical activities need to be done in tandem to have cost effective design and trouble free construction.
- It is of paramount importance that an interface between two is identified, defined and provided in each civil drawing to take care of functional, O&M and safety norms as per statute.
- Interface will help to entail a better understanding between the two disciplines and ensure job clarity during execution.
- Maintain & improve the quality through seamless integration.
- Zero error drawings to avoid rework for installing sub-systems.

3.0 Point of Interfaces for Substation design

3.1 Site Selection

The location requirements for substations are site specific, and are determined by practicalities of engineering constraints, connection costs, environmental issues and impacts on social attributes. Generally, the location is dictated by electrical load center and transmission line arrangements for connection of the substation to the grid. Access to the substation site must be suitable under all but the most extreme conditions. Site selection, substation design and layout shall be such that the substation continues to function through this flood event. The site must be proximal to secure road access or be in a position wherein all weather access road can be constructed. This is a must for transporting heavy equipment such as transformer.

- **Input required from Electrical Engineer**
  - Size of substation area
  - Type of substation. I.e. conventional (AIS), GIS, Hybrid
  - Incoming and outgoing lines
  - Future anticipated expansion
  - Weight of equipment and transportation need

3.2 Geo Technical Investigations

The geotechnical investigation for assessing suitable type and size of foundations for Structures and equipment shall include bore holes, plate load tests and trial pits. The Laboratory tests on soil and water samples shall be carried out to establish use of type of cement in the works and suitability of water for construction. For the purpose of grounding (earthmet) design, soil resistivity measurements shall have to be taken.

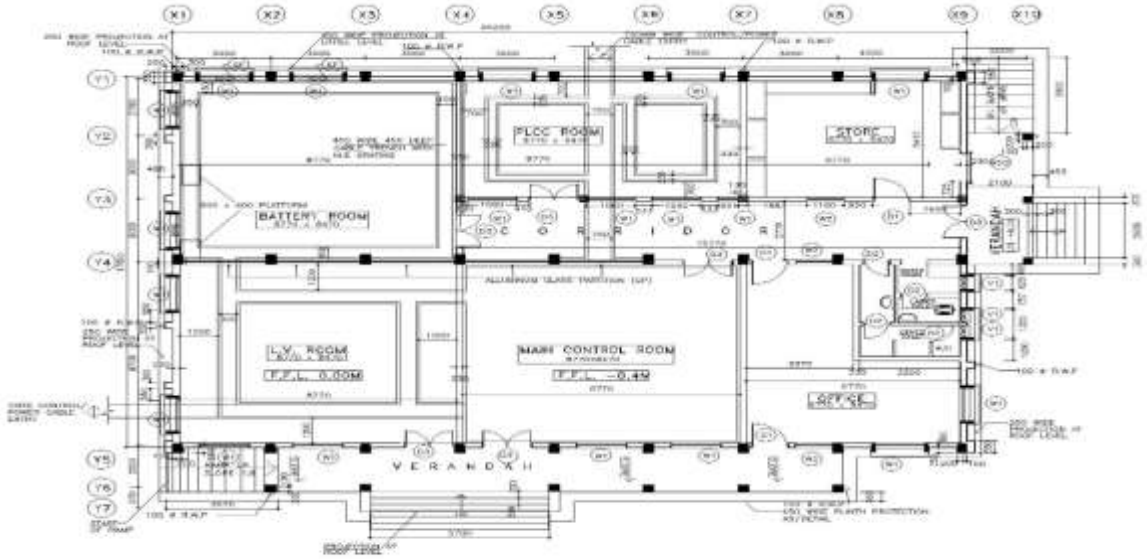
- **Input required from Electrical Engineer**
  - Plan layout showing location of Control Room Building, Transformer & Location of Heaviest load.
  - Soil resistivity mapping

3.3 Substation Layout

There are many different electrical Bus-bar arrangements available but selection of a particular scheme depends upon the system voltage, position of substation in electrical power system, availability of land, operational flexibility needed in system and the budget.

- **Input required from Electrical Engineer**
  - Single bus / two bus / three bus / One & Half breaker scheme
  - Orientation of incoming and outgoing line
  - Arrangement of gantries and equipment as per switchyard layout
  - Cable trench for cables
  - Location of important control centers (bay kiosks)

3.4 Control Room / GIS Room



220KV CONTROL ROOM (SCADA)
(GROUND FLOOR PLAN)

The construction features of control Room/GIS Room may vary depending upon the control panels of the equipment to be installed. Floors may be reinforced to support heavy transformers and switchgear. Walls and ceilings may have to support a heavy cable tray system or bus bars. Additional ventilation or air conditioning may be needed, since electrical apparatus gives off heat but the temperature must not rise beyond the tolerance of equipment. Double doors may be installed to allow for maintenance of large equipment. Fire detection and control systems, such as carbon dioxide or water sprinklers, may be installed.

Control room/GIS room may have extensive provisions for grounding (earthing) and bonding enclosures of electrical equipment to prevent stray voltage and danger of electric shock, even during faults in the electrical system. Lightning protection requires different measures than protection from power-frequency faults.

- **Input required from Electrical Engineer**
  - Location of control room.
  - Arrangements of panels and panel cut outs, Electrical layout, Electrical fittings.
  - Max. size, Nos. & loading of the panels.
  - Type, nos. & loading of batteries i.e. (Lead Acid or Ni-Cd)
  - Requirement and arrangement of Electrical sub systems in building like LTDB, DCDB, special testing room storeroom, battery room, office etc.
  - Entry of cable trench, cable route diagram, Nos. of cable.
  - Type of wiring i.e. Concealed or Open.
  - Type of paints, flooring etc. for battery room
  - Fire Fighting System
  - Air Conditioning and Ventilation System

3.5 Equipment Support Structure Design

They are provided to support the electrical equipment such as, Circuit Breakers, CT & PT, Isolators, surge arresters, insulators and other equipment connected to each other with flexible conductor and rigid busbar. These structures must be designed such that applicable steel design codes and standards are complied. Additionally, structures must be designed to withstand various static and dynamic loads such as the dead weight of conductors/insulators and electrical short circuit forces, wind, snow/ice, and seismic loads.

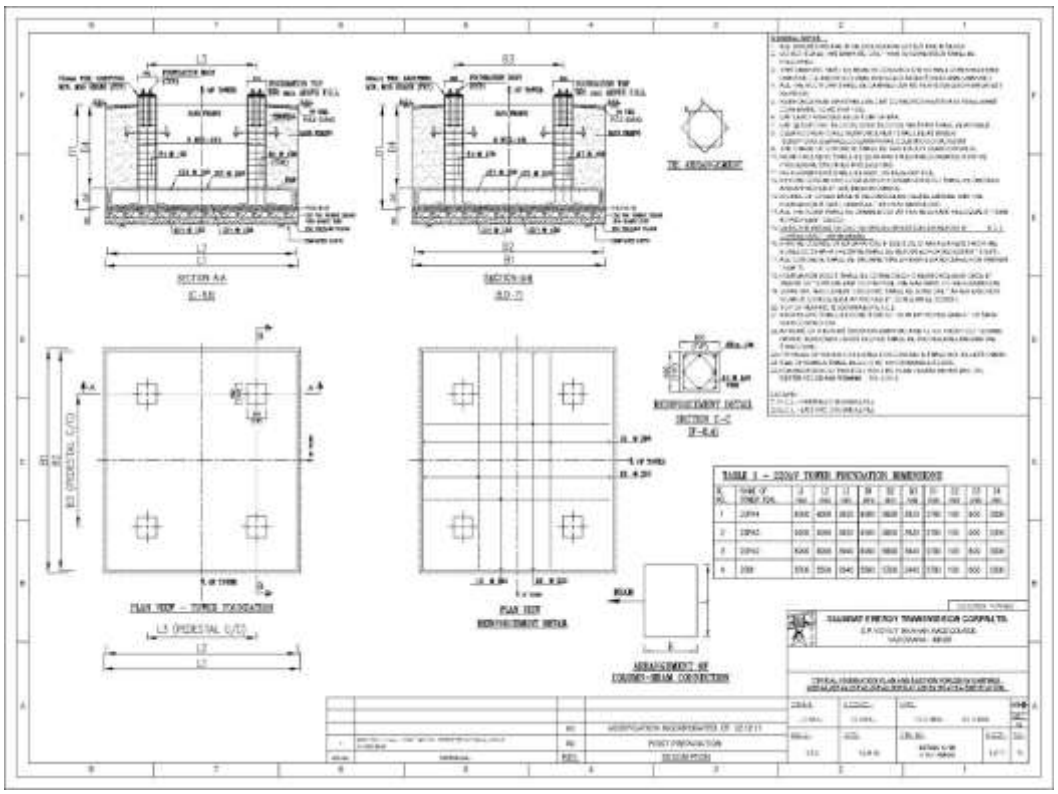
- **Input required from Electrical Engineer**
  - Equipment height and weight
  - Equipment mounting arrangement on structures
  - Required ground clearances
  - Phase-to-phase & phase-to-earth Clearance
  - Loading of equipment, conductor & short circuit force
  - Earthing requirement
  - Mounting arrangement of Marshalling Boxes

3.6 Gantry Structure Design

Substation gantries are required for stringing of overhead conductors from dead end tower of transmission lines to substation gantry. These gantries are also needed to form a bus bar inside a substation. Sometimes a peak is provided on top of the gantry tower for stringing of the lightning protection shield wire. The tower (column) and beam is designed for the project specific load data which is derived from plan layout and section (elevation) details of substation. The design output is then used to select various member sizes and preparation of fabrication drawing for manufacturing.

- **Input required from Electrical Engineer**
  - Bay width, Bus span, Height of upper & lower bus, Height of peak
  - Conductor & Earth wire details
  - Short circuit forces
  - Span of dead end tower to end gantry & Deviation angle (in plan and elevation)
  - Sag tension calculations
  - Length & weight of insulator string & Hardware
  - Length & weight of pilot insulator string & Droppers
  - Ground clearance, phase-to-phase clearance, phase-to-earth clearance
  - Requirement of earthing & maintenance
  - Final layout of gantries as per electrical scheme for foundation design

3.7 Equipment and Gantry Foundation Design

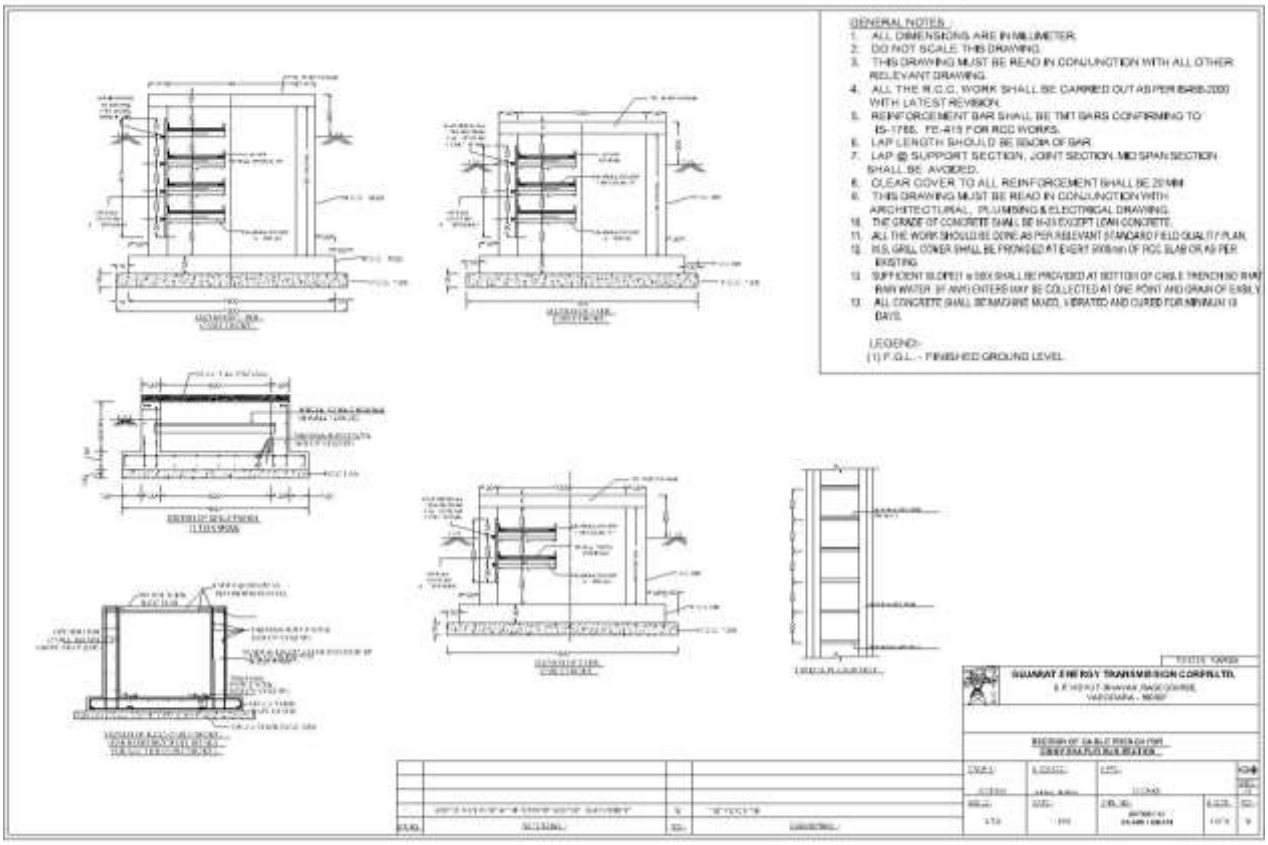


Foundations shall be designed based on the stress resultants obtained from analysis of the worst combinations of loading and on the serviceability requirement of the plant and buildings. The derivation of static and dynamic loading used in the design shall be in line with the recommendations given in the latest codes and standards.

- **Input required from Electrical Engineer**
  - Equipment specifications indicating short circuit forces, loads on equipment
  - Electrical layout of gantries, equipment, Control Room as per electrical switchyard scheme

- Weight and size of heaviest equipment for design of roads
- Plinth height of equipment and gantry foundation as per common reference
- Equipment support structure base
- Clearance between equipment and phases as per CBIP manual
- Foundation loads (to be derived from equipment support structure design)

3.8 Cable Trench



Outdoor cable trenches are required for laying of power and control cables from switch yard equipment to control room. Cables are generally laid on racks mounted inside the cable trenches. PSC covers are provided over the trenches to restrict accidental access to live cables. Cable trenches shall be provided with adequate slopes and their invert levels shall be such that the trenches can drained through appropriately sized pipelines into the nearest point in the peripheral drains where gravity flows can be achieved. In order to prevent flow of water into the control buildings, cable trenches shall be sloped and drained away from the control building. In addition, the design of the cable entry into the control room shall be such that they are as much as possible, water and air tight.

- **Input required from Electrical Engineer**
  - Cable trench layout
  - Nos. of cables and weight of cables in each trench
  - Specific requirement like separation of control and power cables, separate tray for special cables like FO cable, PLCC co-axial cable etc.
  - Movement of equipment over the cable trench to design them accordingly
  - Special provision in cable trench for laying cables in future
  - Special provision for maintenance purpose

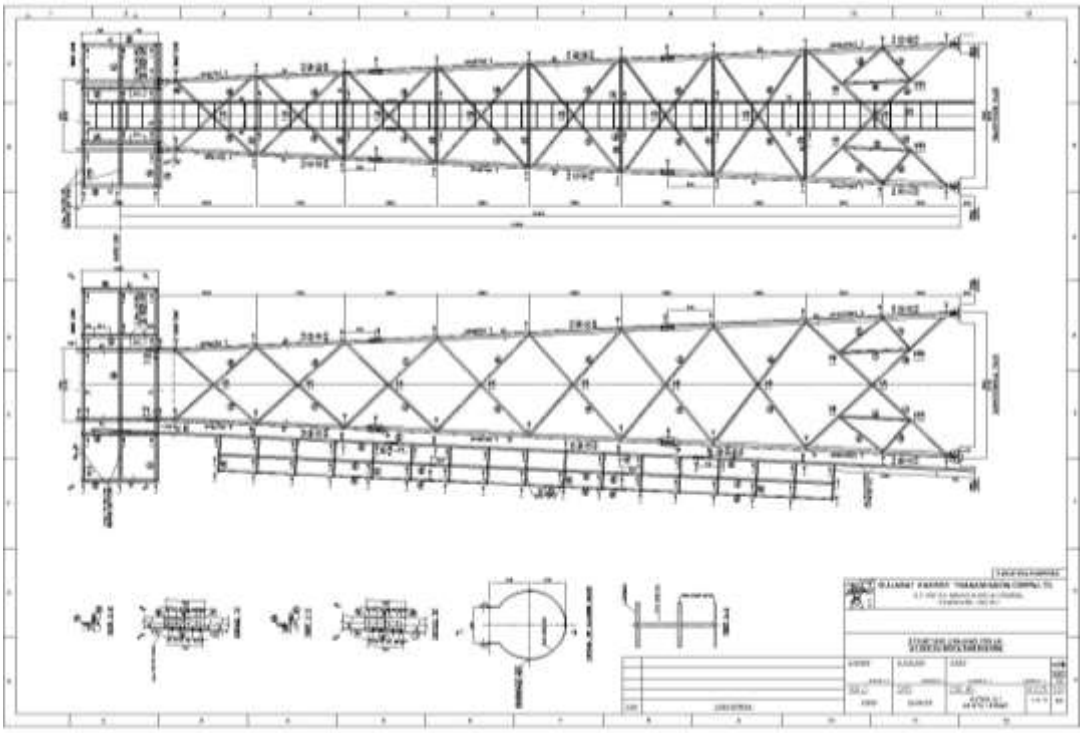
3.9 Foundation of Transformer / Reactor

The designs are to be carried out on the basis of Transformer/Reactor loading data, oil volume and site data such as layout plan of the substation, soil bearing capacity etc. The design and drawings shall be prepared as per specification, national/international standards such as IS, ACI, ASCE, BS etc. In addition to Transformer/Reactor foundation, an oil collection pit is also required for collection of the oil from Transformer/Reactor tank in case of tank damage or fire. In case of large Transformers/Reactors cooling radiators need independent foundations.

Rail-cum-road is required for the shifting of large power transformers and Reactors. Large power transformers are fitted with flanged wheels, which move easily on the rails. The rails are kept flushed with road surface so that there is no hindrance for movement of other vehicles.

- **Input required from Electrical Engineer**
  - GA drawing of transformer / reactor to design foundation for sub systems and adjacent equipment
  - Loading of transformer & reactor
  - Size and location of oil sump, Marshalling Box location and Cooling system
  - Location, length and height of fire protection wall
  - Rail size and gap between rails
  - Any special requirement like anti vibration pads, online equipment and access for maintenance and monitoring

3.10 Lightning Mast



Lightning, an unpredictable, probabilistic phenomenon, can be the cause of severe failures in substations such as insulation flashover and damage of substation devices. Consequently, the power supply is interrupted and economic losses are considerable. Lightning Protection Systems for substations are therefore needed to minimize direct lightning strokes to equipment and buses within substations. These lightning masts can also be used for switchyard lighting purpose.

- **Input required from Electrical Engineering**
  - Type of lightning protection – Mast or Earth Wire protection
  - Height of mast
  - Location of mast
  - Platform arrangement for lighting
  - Dimension of spike for structure design
  - Maintenances requirement
  - Earthing requirement

4.0 Conclusion

Even though the basic requirement of the civil structures, their design cannot be done unless fundamental requirement / input of electrical engineering is taken into consideration.

Every civil engineer engaged in the design and construction of substation need to understand basics of electrical engineering.

About Author

Er. Pradip P Shah is a retired Executive Engineer (Civil) GETCO.

He passed his Diploma in Civil Engineering from the MS University of Baroda, Vadodara in 1994 and joined erstwhile GEB as a Junior Engineer (Civil).

During his tenure as Deputy Engineer, he was in-charge of Design and Engineering of Transmission Lines up to 400kV and sub-station, Towers & Foundations.

After superannuation in 2023, he started working as Consultant providing services of design, engineering & supervision of Transmission Lines & substations up to 765kV. He is also a visiting Faculty at GETRI.

GIS SUBSTATION DESIGN – CASE STUDIES

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1.0 INTRODUCTION

1.1 Civil and Structural Engineering play a pivotal role in the Power Sector & Industries. This paper aims to explore key aspects of Gas Insulated Substation design which has impact on civil and structural engineering through case studies, emphasizing the significance of well-thought-out designs.

1.2 Case studies presented are from the past executed GIS Substation projects designed and engineered by L&T-S&L. The Case Studies aims to shed light on the pivotal role of good engineering practices in substation design and emphasizes the need for optimization to enhance both operational and economic outcomes. We feel the Case studies presented will help in designing of upcoming GIS substations.

2.0 CASE STUDIES

2.1 Gas Insulated Substation Layout Design

2.1.1 For any Substation Project, the design engineering starts from the updation of the conceptual substation layout to the detail design. During the proposal / tendering stage, conceptual layout is generally prepared depicting the major equipment and buildings, roads, etc.

2.1.2 During the detail Design Engineering phase the layout need to be optimised meeting all the technical requirements, keeping sufficient space for the installation of the equipment and its access for operation and maintenance.

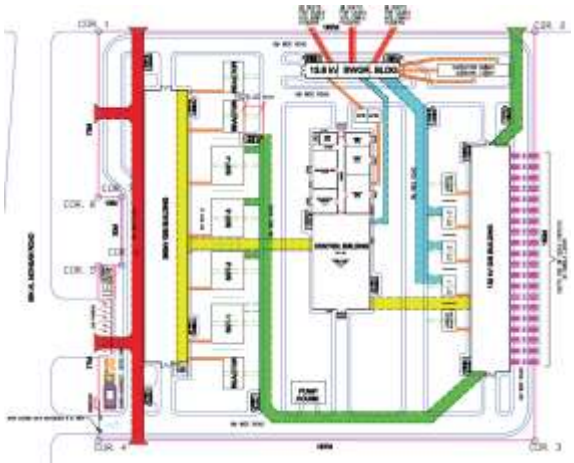
2.1.3 Through a detailed case study, this section illustrates the significance of optimizing the overall layout to reduce civil design costs without compromising operational integrity. The paper will delve into the challenges faced, the innovative solutions applied, and the measurable impact on project costs.

2.1.4 Key Optimization Strategies:

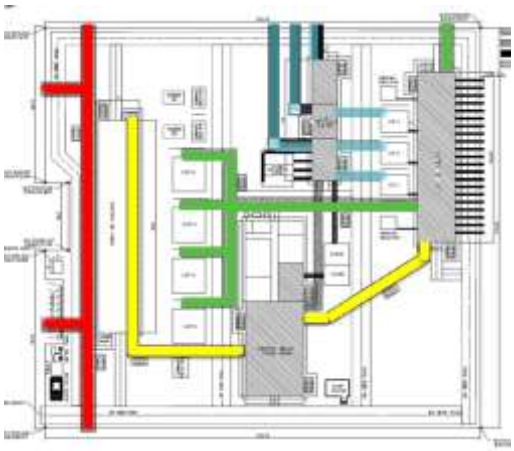
- Repositioning of Buildings keeping the incoming and outgoing feeder connection orientation as it is.
- Optimizing the interconnecting raceway (between the buildings and equipment)

Below is a snap of the 380/132/138kV GIS concept layout received with the tender.

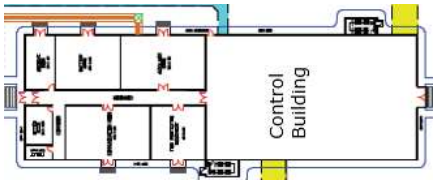
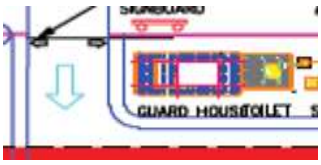
Substation – Tender Concept Layout
(380/132/13.8kV)



Detail Design – Layout – Option-1

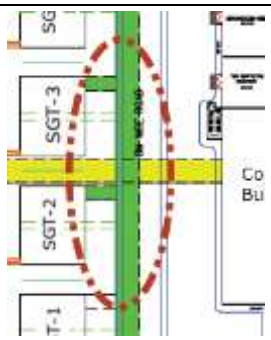


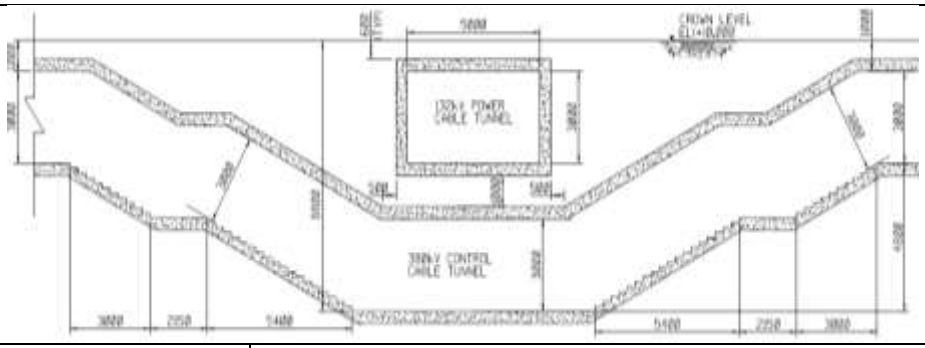
Legend:

380kV GIS Building	500MVA Transformer 380/132kV
	
132kV GIS Building	**67MVA Transformer 132/13.8kV**
	
Control Building	**Gate House (Security Building)**
	
Cable Tunnel (3M Deep x 5M Wide)	**Cable Trench**
380kV 132kV Control Cable	13.8kV Cable

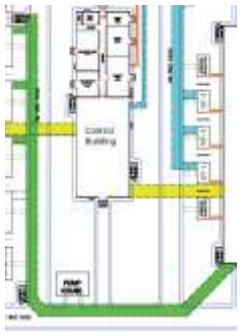
2.1.5 Challenges:

2.1.5.1. There is a crossing of 132kV Cable tunnels and Control Cable tunnel





2.1.5.2. Very Long 132kV Cable Tunnel



2.1.5.3. Layout looks simple and spacious but actually it is very compact layout.

By scrutinizing the spatial arrangement and interconnecting raceway design, it was possible to streamline the layout, achieving cost savings without compromising operational efficiency or safety.

2.1.6 Results:

2.1.6.1 This case study delves into the optimization of GIS overall substation layout, highlighting specific design modifications that led to a substantial reduction in civil costs.

2.1.6.2 The optimized layout not only reduced civil costs due to reduction in cable tunnel lengths but also enhanced maintainability and accessibility, contributing to the overall project construction schedule of the substation.

The details given below are indicative of the optimization

132kV POWER CABLE TUNNEL	Option-1	Tender
SGT to 132kV GIS building 3R-1C-2000 sqmm/phase	144	250
132kV GIS building to outside	22	53
Total Length	**166**	**303**

13.8kV POWER CABLE TUNNEL	Option-1	Tender
GT to 13.8kV GIS building (6R-1Cx630 sq mm/phase) x 3 GT-1/2/3	55	104
13.8kV GIS building to Fence (Not in EPC Contractor Scope of Supply)	0	0
Total Length	**55**	**104**

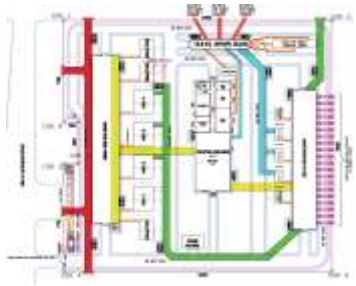
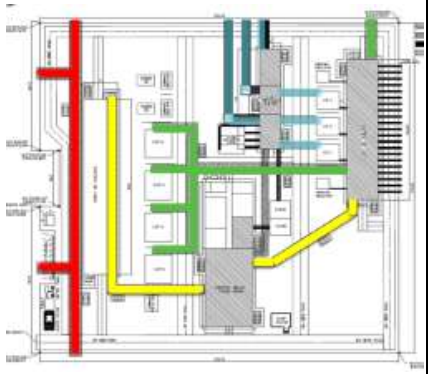
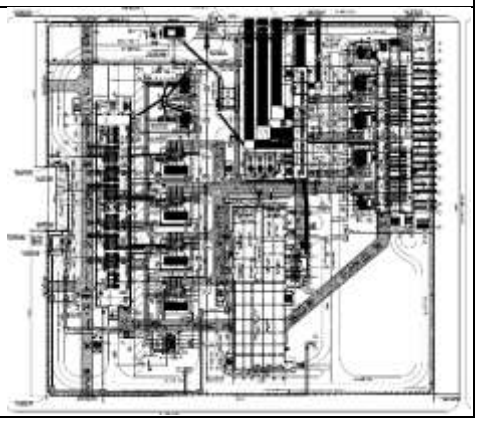
CONTROL CABLE TUNNERL	Option-1	Tender
380kV GIS building to Control building	151	150
Control building to 132kV GIS building	66	42
Total Length	**217**	**192**

- Avoided Underground crossing of 132kV Cable Tunnel & Control Cable Tunnels – Construction difficult and schedule impact
- Reduction of Civil Works for 3 x 5M Tunnel = (250-144) + (192-217) = 81M (27% of Tender length)
- Saving of 132kV EHV Cable Length = (250-144) x 3= 318M,

13.8kV Cable Length = (104-55) x 6 = 510M

- Increase in Control Cable Tunnel length by 13%

2.1.7 Snap of Layouts:

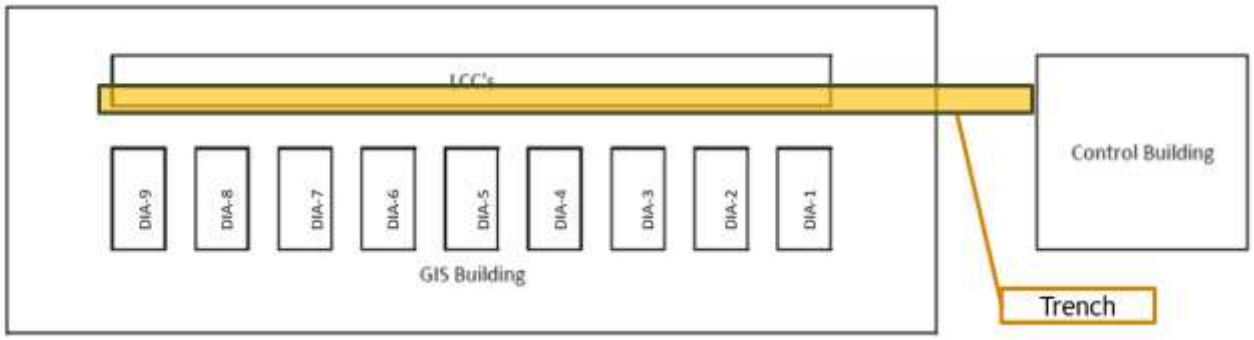
2.1.7.1 Tender Concept Layout (380/132/13.8kV)	Base Detail Design Layout	Detail Design Layout
		

2.2 Cable Raceway Arrangement for GIS Substation

2.2.1 Raceway design for the Substation projects are generally planned and design at very advanced stage of the project. During initial stage the size and number of trays are generally considered based on past project experience. However, during later stage of the project when the actual cable schedule are prepared and cables are laid at site, it may happen that at few locations the trays are heavily overfilled and at some location trays are empty without any cable.

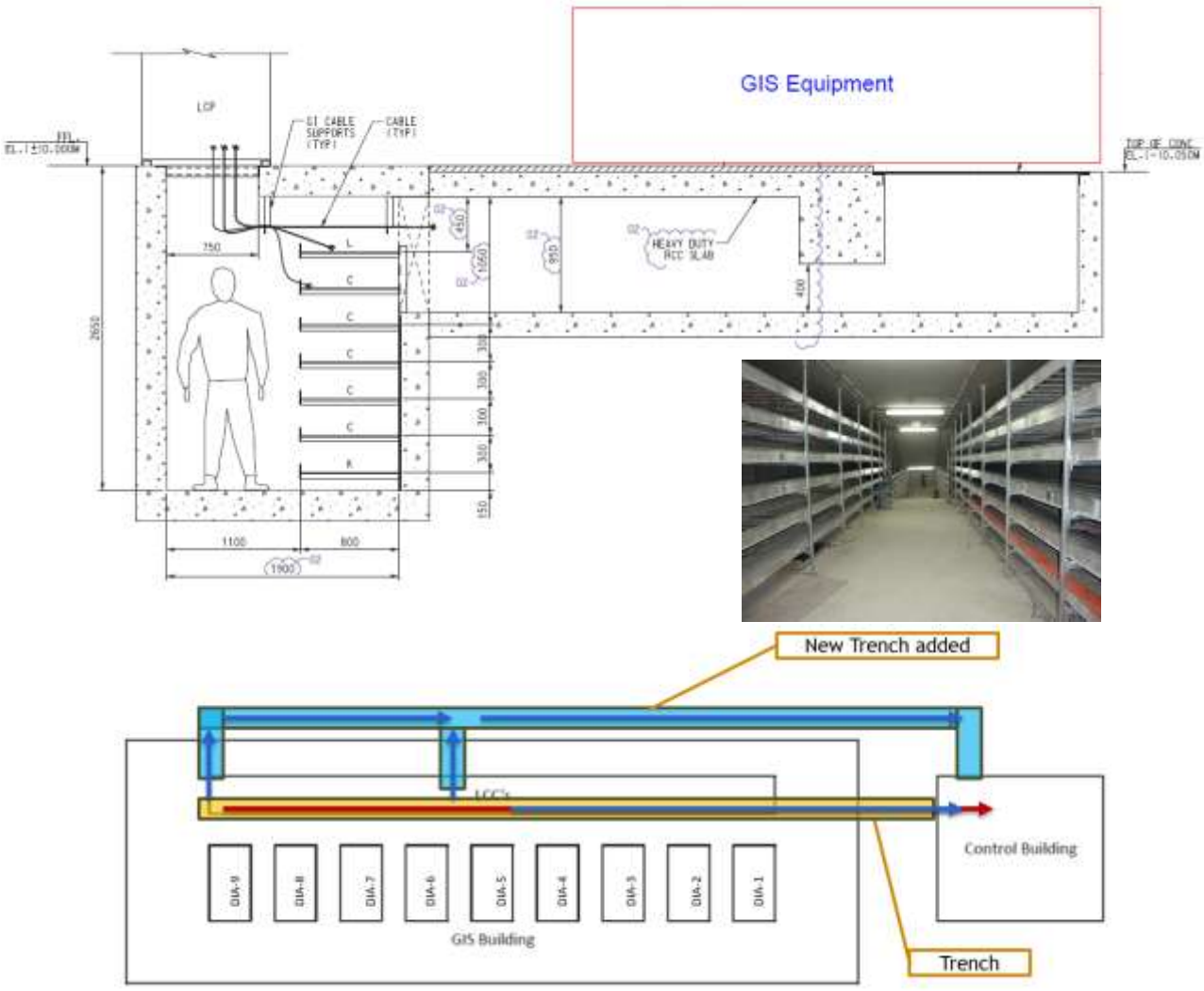
2.2.2 . Below is the snap of GIS Substation Cable Trench. Predominantly in India, the cable trenches are used for laying of cables since constructing the same is economical. However, for large substation (having many bays) the deep depth trenches pose problems from the accessibility of laying, supporting the cables and its maintenance at later stage.

Typical - Plan View of 400kV GIS Substation



Typical Section View showing the Cable Trench and Tray design (7 trays, approx. depth 2.6M):

2.2.3 From the above section taking the cable from bottom tray to LCC requires additional support to avoid the long cable weight getting applied to the LCC panel. Also cable routing from lower trays to GIS is not convenient. Following option was explored and implemented to mitigate the problem.



2.2.4 Additional trench was provided outside the building to provide alternate route of the cable. Cables from Dia numbers 6 to 9 were routed from the trays provided outside the building and cables from Dia numbers 1 to 5 were routed inside the building i.e. Number of trays got divided into two paths, reducing the total tray requirement in single trench. This is an example showcasing the requirement of thoughtful engineering at initial phase to avoid such issues. This has an indirect impact on civil design for control cable trench.

2.3 Control Cable Raceway design for International project:

A good option of designing Control Cable Raceway, wherein we have huge number of cables is depicted in below snap. 3M deep and 5M wide Cable Tunnel is provided with tray on both side and walk way in center.

This is a costly solution but will facilitate a very neat and clean design with ease of adding cables for future bays. Cable tunnels are provided with ventilation, illumination and stair case to enter/exit the tunnel.

3.0 CONCLUSION

3.1 In conclusion, the presented case studies underscore the tangible benefits of optimized design engineering in the realm of civil and structural practices for power substations. By implementing strategic changes in layout and trench design, we not only achieve cost savings but also enhance the civil structural design complexity and constructability issue.

3.2 As we move forward in the power sector, it is imperative for practicing engineers to embrace and integrate these optimization strategies into their projects, contributing to sustainable and cost-effective solutions for the ever-evolving challenges in the industry.

3.3 This paper serves as a testament to the continual improvement and innovation within the engineering field, providing valuable insights for engineers, researchers, and industry professionals alike.

About Authors

Mr. Milan Shah, currently working as Deputy General Manager-Electrical at L&T– Sargent & Lundy Limited, Vadodara, is a professional in the field of Electrical Engineering from M. S. University of Baroda. He has 31 years of experience working in Power & Substation Projects. He has significantly contributed to the success of numerous Gas and Coal-based Power Plant projects, as well as GIS Substation projects, extending his impact across international landscapes such as Oman, KSA, Qatar and Africa.

At present, Milan Shah serves as a Project Manager at L&T-S&L, spearheading Substation Projects. His wealth of knowledge and hands-on experience has been instrumental in shaping the success of these projects, showcasing his commitment to excellence in the realm of electrical engineering

Mr. Apurva Das, currently working as Manager-Electrical at L&T– Sargent & Lundy Limited, Vadodara, is a professional in the field of Electrical Engineering from M. S. University of Baroda. He has 19 years of experience working in Power & Substation Projects. He has played pivotal roles in numerous Gas and Coal-based Power Plant projects, as well as GIS and AIS Substation projects in domestic as well as International assignments.

At present, Apurva Das is contributing as a Lead Electrical Engineering for Waste to Energy, Substations and Power projects. His innovative approach and in depth knowledge have been instrumental for successful design/engineering of projects.

L&T-Sargent & Lundy Limited provides a complete range of engineering and design of power plants from concept to commissioning of open/simple cycle plants, combined cycle plants, cogeneration plants, coal based plants - both subcritical and supercritical technology and associated fields since 1995. L&T-S&L has also expanded its horizons in the Renewable (Solar/Wind/Biomass) Energy, Waste to Energy, Geothermal, Renovation & Modernization and Transmission & Distribution sector.

Role of Civil and Structural Engineers in Reliable and Environment Friendly Designs - particularly for power plants

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1.0 INTRODUCTION

1.1 Any industry has a very core and dynamic role for a Civil Engineer. From the layout and planning of the complete industry till the completion of a project, let alone the Erection of a power plant, is all done by Civil Engineers. Layouts, structure designing, Erection of critical and non-critical structures, like chimneys, Turbine Generator deck, Generator foundation, cooling towers, ash decks, compressors foundation, and infinite other structures are all developed by civil engineers.

It's an ongoing concept as you start power generation with minimalistic resources and keep on upgrading yourself in phases. Thus, unless the complete project is delivered and the whole of the Erection job is finished, Civil is pretty much in the core team.

Sustainable structural engineering is a practice that requires structural engineers, just like any other profession in any other industry, to minimize environmental pollution. This include reducing construction materials, minimising waste, minimum energy use in finished construction outputs, encouraging use of recyclable and renewable materials, and ensuring minimum damage to the natural eco system.

2.0 STRUCTURAL ENGINEERING ACTIVITIES:

2.1 The Structural Engineering activities in any project revolve round the following

- Construction activities in sites that risk destroying Flora & Fauna
- Consumption of energy during construction
- The heavy plant machinery which relies on carbon fuels
- Energy content within construction materials, and Buildings' energy requirement during the usage phase

2.2 Today, construction projects account for 40% of carbon emissions and 36% of the energy used globally. Sustainable structural engineering practices are more critical now than ever due to concerns over global warming, rising sea levels, as well as changing weather patterns.

3.0 ROLE OF STRUCTURAL ENGINEERS IN ACHIEVING STRUCTURAL SUSTAINABILITY

3.1 Structural engineers plan, analyse, design, construct, inspect, monitor, maintain, rehabilitate and demolish temporary and permanent structures or structural elements. Structural engineers are required to design robust, durable, and stable structural elements.

3.2 To achieve structural sustainability in buildings and structural components, structural engineers should focus on the following

- Consider technical, environmental, social, economic, and aesthetic aspects of the structures during the design, construction, use, and maintenance stages.
- Know the design function or purpose of the structure, such as municipal, educational, residential, coastal, historical, religious, commercial, institutional, multi-family power plants or mixed-use.
- Design with the mindset that the depletion rate of raw or natural materials is high and
- Design for a sustainable structural material that consumes lesser energy, emits minimized carbon gases and is durable and robust to perform the intended function. Structural materials include concrete, wood, steel, aluminium, plastics, and composites.
- Minimize the use of steel and concrete and maximize the use of materials with less environmental degradation.

- Design for sustainable structural elements such as beams, trusses, columns, arches, plates, shells, and catenaries. It involves practicing structural analysis of these structural elements to ensure that they are sustainable for the structure. For example, analysing the statically determinate and indeterminate structural beams under loadings.
- Design for maximum structural flexibility to enable future changes in the structural use of the building in its lifetime.'

4.0 THE AREAS OF CONCERN FOR CIVIL AND STRUCTURAL ENGINEERS:

4.1 The following are the concerns which structural and civil engineering must address in project

- Quarrying for raw materials that risks polluting water sources (underground and surface waters)
- Manufacturing and transportation of construction materials, which influence carbon emissions. For instance, cement manufacturing contributes 2.8 billion tons of carbon emissions to the environment. The current increase in construction activities and urbanization risk raising it to 4 billion tons annually.
- Structural engineers in the United States use two basic structural materials: concrete and steel. Regrettably, concrete and steel use massive amount of energy to process and emit much carbon into the environment. These structural materials have inherent benefits and will remain dominant structural systems. However, you can explore substitute structural materials. Sustainable structural engineering involves investigating construction materials with low environmental effects and use of locally available substitutes.

5.0 IMPROVEMENT IN STRUCTURAL LIFE-CYCLE PERFORMANCE

5.1 Structural engineers design most structures to reduce the initial project's cost instead of the entire project's life costs and based on agreed factor of safety. A classic example of structural failure is of Fukushima Nuclear Power Plant in Japan which was devastated in the unprecedented Earthquake of 9 on Richter scale in March 2011. The plant was designed for the usual factor of safety considering the observed earthquake levels over a proven time span and was not been designed for this unprecedented level of 9 on Richter scale.

For instance, the cost of constructing a bridge is usually lower than the costs of maintaining and demolishing it yet engineers many times fail to consider the entire structural life design costs.

5.2 You can slightly increase the initial construction costs to reduce maintenance dramatically and allow for salvage or disposal at the ultimate structural life; as a result, you will reduce the structural life-cycle costs. When you reduce the life cycle costs, structures become more sustainable than other structural engineering practices. Sustainable structural engineering has focused on improving structural life-cycle performance and made it an objective to improve structures' economic and environmental performance significantly.

- Optimise the factor of safety without compromising the quality of deliverables. However, the factor of safety should be related to structure as a whole.

6.0 ROLE OF STRUCTURAL ENGINEERS IN POWER LINE DESIGN

6.1 It was understood that transmission lines were merely gigantic multiple-span suspension bridges made up of steel, concrete, wood, aluminium, and other materials that required surveying, site work, foundations, and of course project and construction management.

6.2 It is surprising to hear people even within our profession that think of transmission and distribution power lines as something that electrical engineers do. While there is obviously a lot of electrical engineering involved in our power grids, once the wires are sized for a transmission line, the rest of it is mostly structural. In fact, ASCE (American Society of Civil Engineers) is quite active in the transmission line industry. Under SEI, there are two Standards, ASCE-10 *Design of Latticed Steel Transmission Structures* and ASCE-48 *Design of Steel Transmission Pole Structures*.

6.3 In Indian Transmission line tower are designed as per IS 802 (Part-I Section 1&2). The loadings are factored to take care of all the possible combination of loading including climatic and conductor tensions.

6.4 Use of monopole structures is also being encouraged in power transmission, particularly in the areas with space constraint.

7.0 CURRENT SUSTAINABLE STRUCTURAL ENGINEERING SOLUTIONS

7.1 Structural engineers can take several steps to control the environmental effects of the structural design and improve its value. The current sustainable structural engineering solution include the following

- Minimizing the incorporated energy in construction materials
- Minimizing on-site waste
- Minimizing energy use of the finished construction product
- Utilizing recyclable and renewable structural materials
- Maximize the durability of the structural system
- Conserving the environment serves as the natural habitat before, during, and after the structural construction phase
- Improvement in life-cycle performance
- Specification of recycled materials
- Use of substitute materials.

Having informed decisions at each stage of a new construction project bring huge environmental influence, and applying sustainable construction approaches will minimize your company's negative influence on the environment. An example is the sustainable waste management approaches during the structure's usage. Additionally, green structures require less operating costs. Research claims that applying the latest sustainable structural construction technologies could deliver annual savings worth $400 billion in global spending on energy. As a client, you must know that your specification for a new building or building modification significantly affects the project's specification and, consequently, the structure's operation costs.

7.2 For example, structural engineers who designed Traversina Bridge in Switzerland used locally available timber to maintain the bridge without additional support. The design makes the bridge sustainable since the design objective has helped establish a structure with improved environmental performance and economic life cycle costs.

Figure 1: Traversing Bridge showing the use of locally available timber to enhance its sustainability.

8.0 SPECIFICATION OF RECYCLED OR SALVAGED STRUCTURAL MATERIALS – CASE STUDY

8.1 The traditional construction method excavates raw resources and processes them into useful construction products. Structural engineers now look for alternative material sources. The built environment is estimated to contain more copper than their ore. Hence call you to consider mining the construction materials in the built environment.

Today's sustainable structural engineering recycles and salvages construction materials that former engineers sometimes extracted back from the earth. Salvaging and recycling solve growing waste disposal and landfill challenges.

The current sustainable structural engineers design and make concrete from waste products and salvaged materials. When making concrete for sustainable structural elements, you can use fly ash, waste products, and recycled aggregates to replace Portland cement and natural aggregates. We hope that future structural engineers will make concrete primarily from waste products and salvaged materials since the resultant product has low initial costs and improved environmental and engineering performance.

Moreover, structural designers have also sought to maximize structural design flexibility to give room for future changes in the structural use of the building during its service life. An example is Stansted Airport Terminal, which provides maximum flexibility and uses recycled materials. Stansted Airport Structure uses steel, the best structural material for sustainable construction. Steel modules provide a long span that enables significant interior flexibility and building expansion for future needs. Lastly, if the user does not need the structure, the structural engineer disassembles the structural elements and reuses them to build another structure. Salvaging structural materials like steel is better than recycling because of the high energy needed for recycling.

Figure 2: Stansted Airport displays the use of steel modules to provide long span and other design features that enhance its sustainability.

9.0 USE OF SUBSTITUTE MATERIALS

9.1 Structural engineers in the United States use two basic structural materials: concrete and steel. Regrettably, concrete and steel use massive amounts of energy to process and emit much carbon into the environment. These structural materials have inherent benefits and will remain dominant structural systems. However, you can explore substitute structural materials. Sustainable structural engineering involves investigating construction materials with low environmental effects. Structural Engineers work to build and implement structural elements made of paper. You can explore substitute materials specifically for structures with short life plans to attain engineering goals of economics and efficiency while minimizing construction environmental effects.

10.0 HOW STRUCTURAL ENGINEERS CAN ADOPT MORE SUSTAINABLE DESIGNS

10.1 How can structural engineers adopt more sustainable designs? Structural engineers can go deep to adopt more sustainable designs by minding:

- Sustaining living
- Zero waste
- Zero carbon

10.2 Structural engineers can mind sustainable living when designing residential structures for their clients. It involves putting effort into building structures without volatile organic materials (VOC) and sufficient water supply to promote:

- Motivation
- Good lifestyles,
- Workers' well-being and health.

10.3 Sustainable structural engineering practices that create a framework to consider consumers' health and happiness enable structural engineers to adopt more sustainable designs.

Structural engineers focus on waste reduction before, during, and after construction. However, you can think about ways to design for reuse or deconstruction. Deconstruction is the ability to retrieve a structure's elements for ready reuse. Structural engineers can use bolted joint connections instead of welded ones. Thus, you can maximize the chances to reuse the material to promote zero waste at the end of the structure's life. ~~As~~ such design of the structural system considers material durability, the usability of reversible connections, prefabrication of structural elements, and the use of the material of minimized lifetime incorporated energy. Zero waste structural design is one-way structural engineers can adopt more sustainable designs.

Structural engineers should focus on zero carbon emissions to adopt more sustainable designs, and pay attention to locally available resources for construction materials, primarily recycled or reclaimed structural materials. When considering on-site materials for construction design, you can reduce carbon emissions from long-distance transportation. In addition, you also support the local economy. Examples of such locally available materials include using grated sand to make sand for constructing outdoor structural elements like pavements. Similarly, the application of reused steel qualifies to be a locally available material that promotes zero emission. Hence, you can consider achieving zero emissions to adopt more sustainable designs.

11.0 LONG-TERM CHALLENGES IN THE FACE OF STRUCTURAL ENGINEERING

11.1 Even though sustainable engineering provides solutions to minimize environmental pollution in the built environment, several long-term challenges face structural engineers. The challenges strike the profession in 3 main areas:

- Practice
- Research
- Education

12.0 THE STRUCTURAL ENGINEERING PRACTICE

12.1 Structural engineering practice encounters crucial problems in the attempt to provide construction sustainability, such as emerging policies and the economic nature of sustainable engineering practices. The construction industry rewards structural engineers based on the initial construction costs instead of the project's life cycle costs. Sustainable structural engineering results in low life cycle costs after high initial costs. This makes structural engineers design and construct bridges and buildings with high life cycle costs and more environmental consequences. For instance, the low life cycle costs can drastically reduce spending (government and private sector) spending on structures. Therefore, effective policies allow structural engineers to account for disposal and maintenance costs and initial costs (entire life design) during structural design.

13.0 THE RESEARCH

13.1 Structural engineering is constantly focusing on evaluating and maintaining available structures. The increase in the invention of non-destructive structural testing approaches over the past few years proves the claim. It shows that structural engineers are busy improving built environment sustainability by extending the lifetime of available structures instead of building new structures. Therefore, structural engineers require new options for practicing sustainable construction. In addition, the structural engineering community needs new materials made from waste products to construct structures with lower economic costs and environmental impact. Ideally, a sustainable built environment would assist in absorbing excess carbon gases and using waste products from all other sectors to solve the landfill challenges. The objective of a sustainable built environment needs collaboration between practice, government, and learning institutions. Therefore, research in sustainable structural engineering must attach to design, policy, social and economic impacts.

14.0 CONCLUSION

14.1 Structural engineering practices consume a large share of natural resources. However, with the developing issues about climate change and the depletion of naturally existing construction materials, there is a growing pressure on structural engineers to reduce the environmental impacts of their construction practices by practicing sustainable structural construction or green building.

14.2 Revised construction policies can change the minimum required levels for designing and constructing a structure today. A structural engineer needs to know the meaning of sustainability, its importance in improving the well-being of the built and natural environment, and the steps that the construction industry takes to improve the environmental impact of construction activities on society. Therefore, you must advance your construction technology to provide structures with reduced energy needs and embodied energy.

About Authors

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Born: 25–08–1932
Completed his BE (Civil) from Faculty of Technology and Engineering, The MS University of Baroda, Vadodara in 1953. In the year 1958 he cleared the IES examination conducted by UPSC and topped. He completed his Master's in Civil Engineering in 1960 from Colorado University USA. He was awarded a UNESCO fellowship for completing a PhD in Civil Engineering. He joined as a junior lecturer in the applied mechanics department of the Faculty of Technology & Engineering, MS University of Baroda, Vadodara. Later on he became professor and H.O.D of the same department.

He has done fundamental research work on vibration of structures, Bridges & Buildings etc. He has also worked with Hindustan Construction Company Ltd. Mumbai for the development of dams and bridges. He has to his credit the publication of more than 40 research papers which have been published in the national and international journals.

He has rendered his services as an expert on many assignments which include academy and industry. He has also headed many social, cultural, sports and financial institutions in his carrier spanning 70 years. Even at this age he is very active in academic and social activities.

SM Takalkar

Born on 30 March 1948, Graduated in Electrical Engineering from The MS University of Baroda, Vadodara in 1971. He has more than 40 years' experience in Transmission and Distribution in Power Sector as well as Hydro Power. He retired from Gujarat Energy Transmission Corporation in the cadre of Chief Engineer in March 2006. Presently he is a Managing Director of Consultancy Firm named "**Takalkar Power Engineers & Consultants Pvt. Ltd**." in Vadodara. The firm is engaged in Design, Engineering and Construction Supervision of Transmission lines and Substation up to 765kV. The firm is also actively involved in industrial electrical design and hydro power designs.

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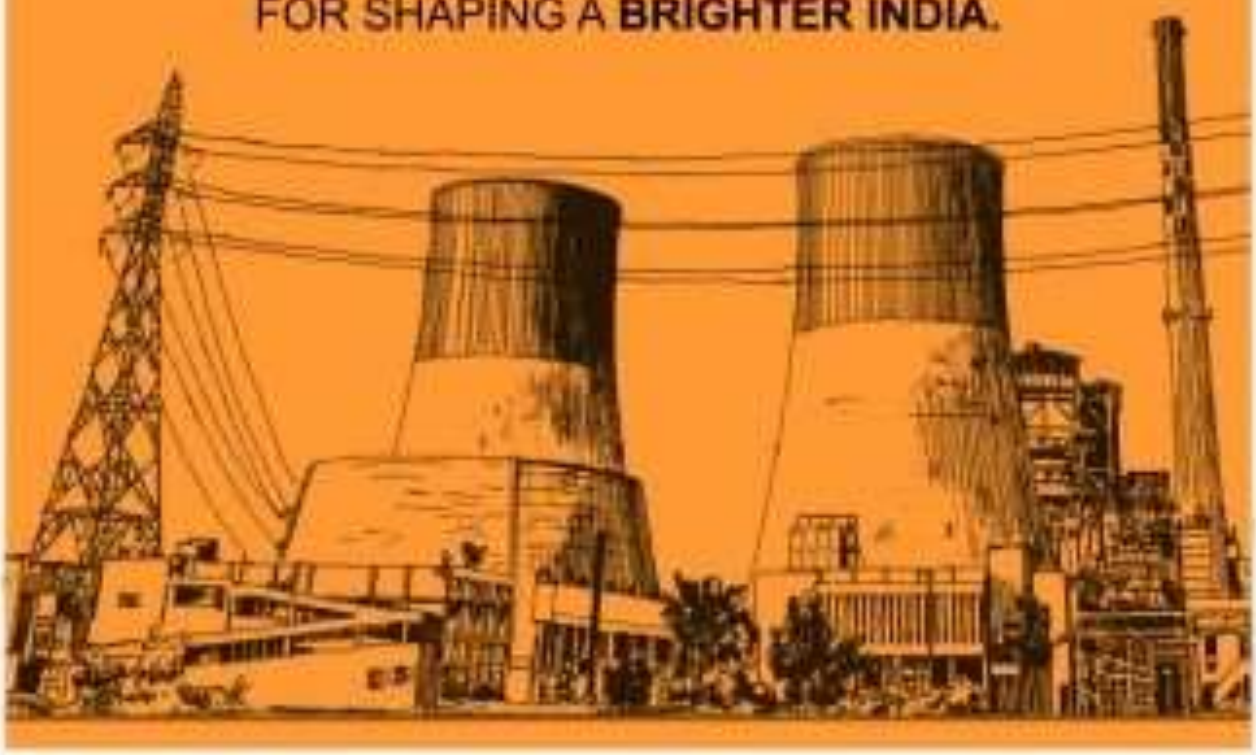

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OVERVIEW OF METHODS FOR TESTING PILE FOUNDATIONS AND STRUCTURES FOR POWER PROJECTS

Ravikiran Vaidya, Principal Engineer, Geo Dynamics, Vadodara

SYNOPSIS

India has invested hugely into power sector during the last decade. This has resulted in establishment of power plants across India. They range from coal based plants constructed by the private and public sector undertakings, nuclear power plants as well as the recently popular solar power projects. Transmission of the generated power is through extra high voltage transmission lines. Where soils are weak or large loads need to be transferred to foundations for power projects or transmission lines, pile foundations are generally the most preferred choice. The most common diameters a decade ago were between 500mm to 760mm used by NTPC and in some cases it extended to 900mm. However, these days pile diameters upto 1200mm, 1500mm and even upto 2.5M are not uncommon because of huge foundation loads in land and deep river constructions. Solar power projects on the other hand use pile diameters as low as 200mm.

Since there is limited information available and the final capacity is also greatly governed by workmanship, testing of piles is imperative before proceeding with further activity. Thus although power projects are predominantly electrical in nature, the cost and execution is greatly affected by civil works and in the inter-disciplinary world it is important that engineers are aware of concepts of testing of foundations and super structures as they several times greatly influence the delivery schedule and costs of constructing power projects or transmission line towers etc.

The current paper discusses provides introduction to various methods available for pile foundation testing including some recent advances in these methods and their limitations.

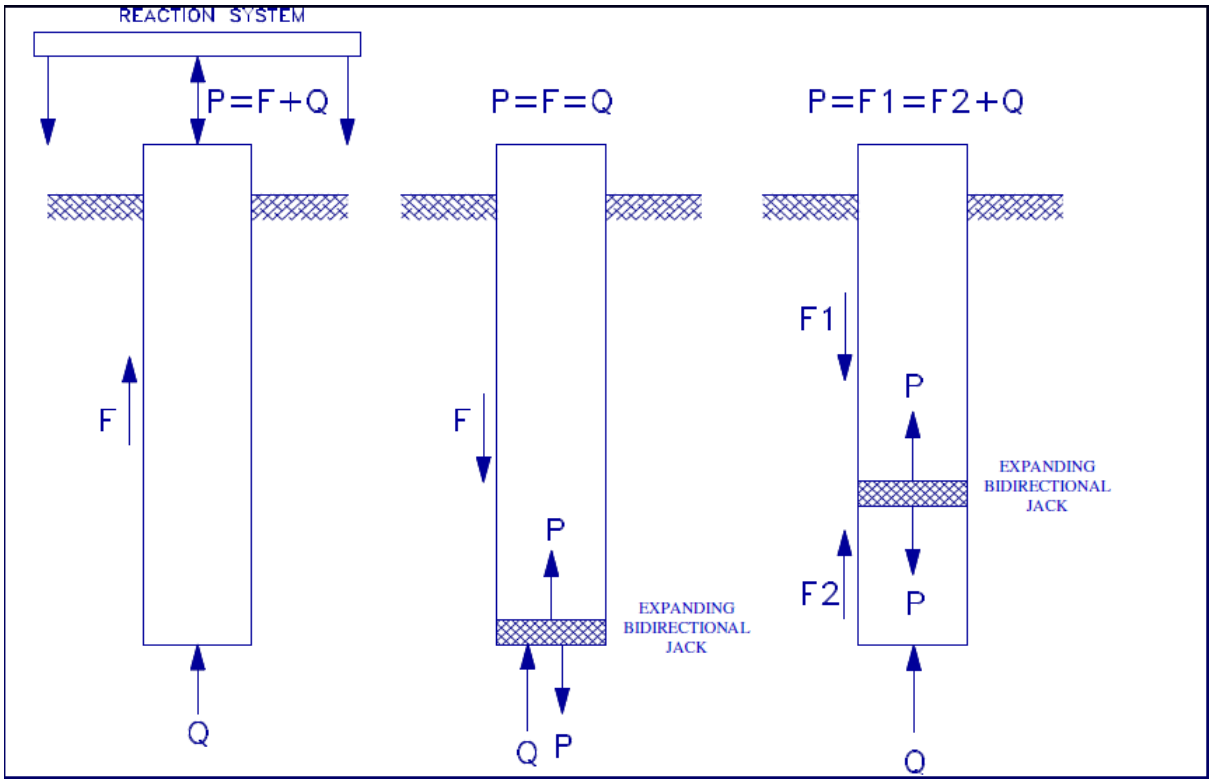
1.0 INTRODUCTION

1.1 India has a total of 106 thermal power plants and of these majority are coal based and some of them are gas based power plants. The length of transmission lines is about 5 lakhs ckM as per information available. Where soils are weak in the upper layers or in case of water bodies, the most common type of foundation for these power projects is pile foundations of various diameters and length. Power projects at Mundra, Hazira, Wanakbori, Dahej, etc. are some of examples of power projects on pile foundations. The same is true with transmission line towers where pile foundations become even more critical as the governing loads are generally uplift and lateral loads with compression loads only in some cases. Yet little importance is provided to construction and validation of pile foundations resulting in tilting or collapse of towers after every cyclone or floods. Similarly, the quality of stubs, columns or pedestals of transmission line towers need to be ensured of high quality to sustain their design life of 50 years or more. The same is true for power projects as although most coal based projects may have a life of 30-35years, the civil structures and foundations can be reused after replacing the installations which will be a dire need in the next few years.

1.2 The paper describes methods like bi-directional load testing which can replace static load testing for power sector projects. The method not only provides compression load but also provides information on uplift or tension capacity of the pile in a single test. An instrumented lateral load test is also described to show how point of fixity can be obtained in addition to only lateral load. Conventional integrity testing is used often but it is more abused then used and the paper highlights the method and its benefits, limitations as proper use of the method is expected to resolve several issues with transmission line towers. High Strain Dynamic load test provides information both on uplift and compression capacity and its application provides the relevant information in quick time and resolves conflict. TG Decks are very commonly tested with ultrasonic pulse velocity method but emerging technologies like thermal profiling is expected to provide quick information on integrity within few days of casting and is also included in the paper.

2.0 Bi-Directional Static Load Testing

2.1 A conventional static load test involves putting a kentledge or reaction piles or anchors to derive reaction and then placing jacks at the top between the pile and girders before a load test can be conducted. The process is time-taking, laborious and in several cases the contractor is allowed to construct working piles with an under-taking as otherwise there is considerable delay. Testing initial piles for transmission line towers gets extremely difficult due to the logistical issues involved in erecting girders, getting heavier cranes etc. for load test. The bi-directional static load test in comparison needs no extra or specific arrangements to do a load test other than the assembly required to cast a pile. Figure: 1 shows a schematic of various methods and also a schematic of bi-directional load testing.



2.2 As evident in Figure 1, a conventional method requires jacks to be placed at top between the reaction and the pile. During BDSLT, the jacks are normally placed at an intermediate location within the pile at a point where the top friction equals the sum of friction below the jack and the end bearing. The point of location of jack is calculated from the borehole data closest to the pile location. In a rare case wherein the total friction within the pile is equal to end bearing, the jacks can be placed at the bottom of the pile too. A typical case study for a Mumbai trans-harbour link explains the method in detail.

2.3 The project required installation of 2M, 2.2M and 3M diameter concrete bored piles into the Arabian sea. Total about 1800 piles were installed at the project by the contractors at the project. The depth ranged from 13m to 48m and they were maximum 30m into seabed. These were generally installed into rock formation with a socket length varying from 7M to 16M depending on the type of rock. The test load ranged from 6250T to 10000T. It was the first time that such a high capacity large diameter concrete bored pile installation was planned in India. These piles were installed using a reverse circulation drilling (RCD) rig which is mounted on a casing and drills into the hard rock. Figure 2 shows a picture of RCD in operation.

Figure: 2 A Reverse Circulation Drilling Rig in operation

2.4 In this case for a 2M diameter pile, the ultimate test load considering a factor of safety of 2.5 was 6250T. The total pile length was 31.5M which included 8.5M of free standing length in and above water, 6.5m into marine clay and then into weathered basalt for 16.5m. IRC: 78-Appendix 9-2018 guidelines implied that the jack capacity had to cater to 25% extra pile capacity in addition to the specified ultimate load. This implies the jack capacity had to be designed for 7812.5T. Five jacks of 900T were installed to cater to a capacity of 9000T and this exceeded the specifications of Appendix 9 of IRC: 78. Seven levels of strain gages were installed at various layers of soil stratum change to monitor load transfer and also to compute friction and end bearing within the rock socket zone. The jacks were positioned 3M from the bottom based on rock socket friction and end bearing calculations. A schematic of jack location and strain gage arrangements is shown in Figure 3.

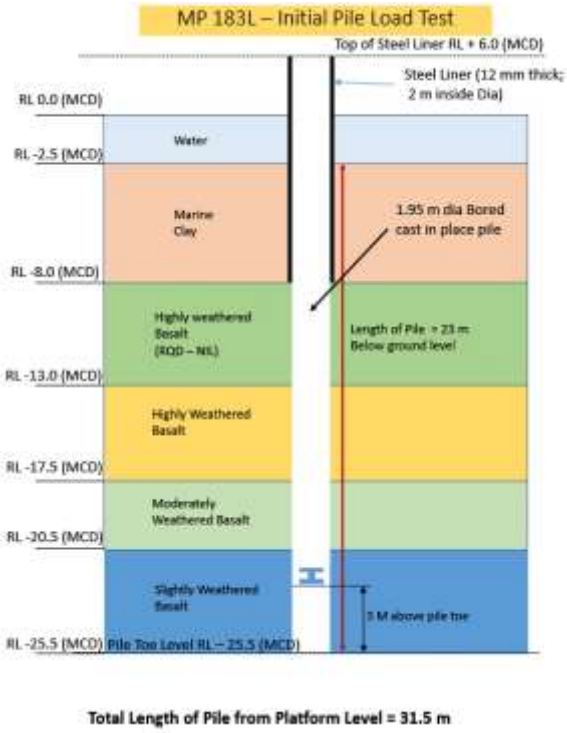


Figure: 3 Schematic of Soil and Jack Location

2.5 Four tell tales on top plate, four on bottom plate, two at the pile bottom were provided to monitor pile displacement and jack opening. In addition to measurement from tell tales, displacement sensors were installed between the top and bottom plates to monitor the jack opening independent of tell tales. Sonic logging pipes were installed to evaluate pile integrity. Thus the entire instrumentation complied with ASTM D8169/8169M-18 and IRC: 78 requirements. A picture of the assembly is presented in Figure 4. The pile was concreted using a floating barge and no issues were reported during concreting

Figure: 4 Bi-Directional Assembly ready for lifting and lowering

2.6 All instrumentation was checked post concreting and cross hole sonic logging (CSL) results also showed excellent pile integrity. Refer to Figure 5 for CSL test results.

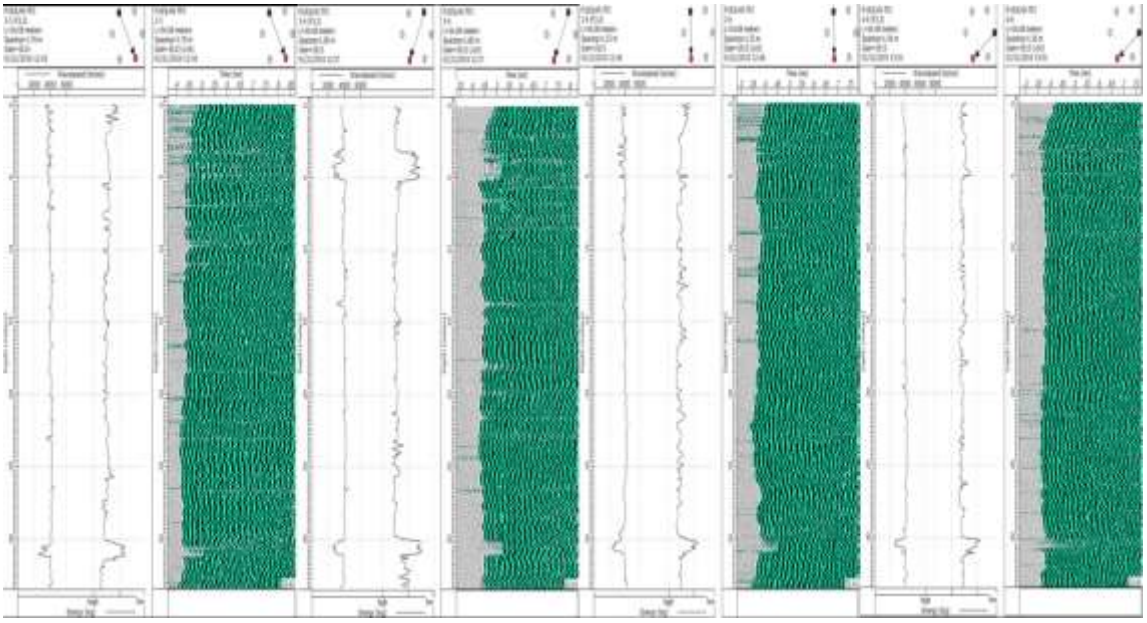


Figure: 5 CSL Test Results with Location of Jacks Defined

2.7 Thus one of the most important aspects of tremie concrete was verified and there was reasonable assurance that the test will be successful.

2.8 The test commenced after 28 days after all the instrumentation in place with two more LVDT at top in addition to those from tell tales as mentioned above. The jacks opened by only 6mm for a load of 6250T which was also verified by displacement transducers. Refer to Figure 6 for the load test curve.

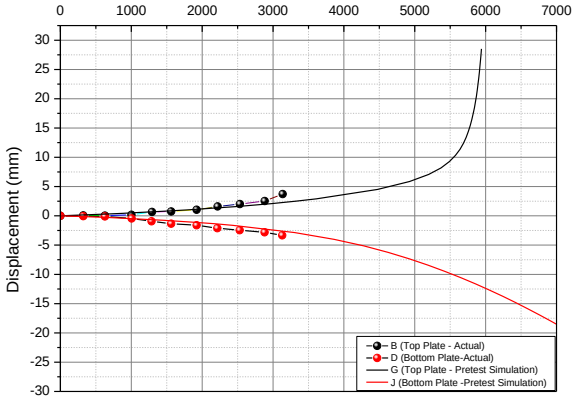


Figure: 6 Top and Bottom Segment Load Test Curve

2.9 Figure 7 also shows the results of load transfer from strain gage instrumentation. As evident form the figure, most of the load was taken by the friction in rock socket zone and the pile still had a large capacity which was not required to be measured as the test load was achieved

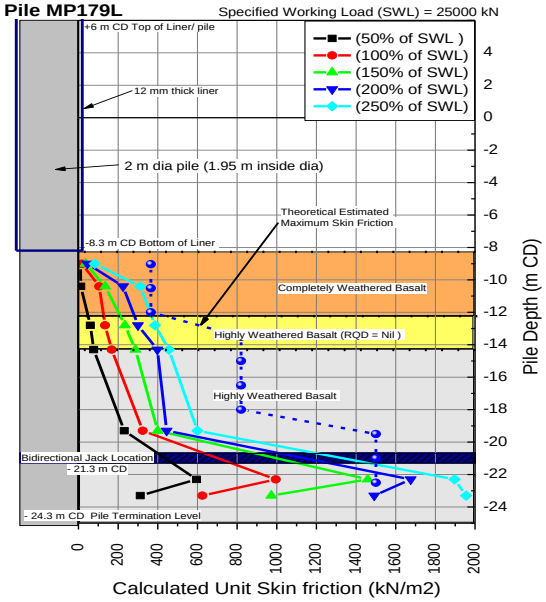


Figure: 7 Results from Instrumentation during BDSLT

2.10 The test can clearly conclude the compression capacity was more than 6250T. Since the jacks were at a point of equilibrium, it can be concluded that the uplift capacity of the pile is at least 3125T or more. This results in savings in time and cost of doing a separate test to validate tension capacity. The method confirms to various Indian and international specifications like ASTM D8169/8169M, ICE, Euro codes etc.

3.0 Lateral Load Test with Inclinometer system

3.1 A conventional lateral load test involves applying lateral load at cut-off level and measuring the deflection of the pile in the opposite direction. Figure 8 shows a picture of a conventional lateral load test.

Figure: 8 Conventional Lateral Load Test in progress

3.2 However, for structures that have huge lateral loads like transmission line towers, several times, it is important to know not only the peak deflection but the deflection along the length of the pile and the point of fixity. This will not only enable a comparison with the design p-y curves but also provide information on the point of fixity beyond which there is no deflection implying no effect of lateral load. Figure 9 demonstrates a lateral load test with inclinometer system in place (IPI type inclinometers). It also shows a lateral load test in progress with the torpedo type system with a PVC inclinometer pipe installed to 15M depth for a 2M diameter pile. The pipe was installed to 15M as the theoretical point of fixity was 12M.

Figure: 9 In-place inclinometer and testing with torpedo type inclinometer on right

3.3 The pile was loaded cyclically upto 67.4 tons of ultimate load and it was then maintained for 24 hours to obtain peak deflection. The failure criterion was considered to be displacement of 12mm.

3.4 The maximum settlement was observed by dial gages to be 8.54mm when the load was maintained for 24hrs. Once the pile was unloaded to zero load, the net displacement of the pile was observed to be 3.6mm. The elastic recovery was around 4.94mm. Load displacement plot of the pile is presented below as Figure 6. The inclinometer findings are presented in Figure 10 and shows that the deflection was zero at 1.35M from top and was defined as the actual point of fixity.

4.0 Use of Low Strain and High Strain Methods for a Project in North Gujarat

4.1 For a major power project in North Gujarat, bridge project on a major river in India, more than 2000 piles were to be installed into rock for a depth ranging from 16M-25M. Figure 10 shows the borehole data.

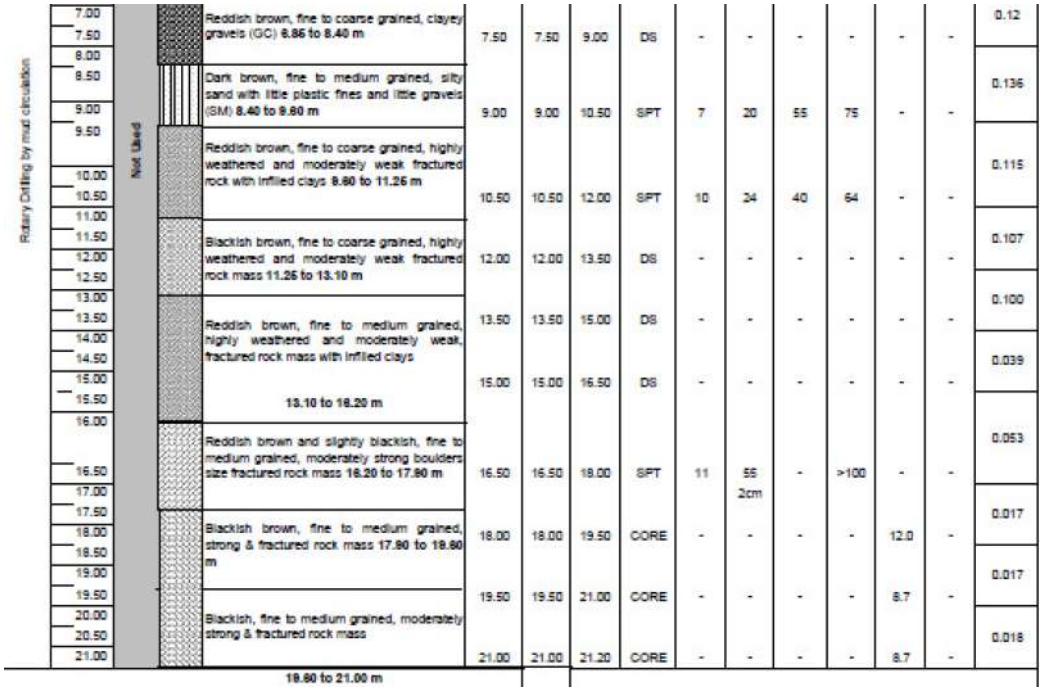


Figure: 10 Soil borehole indicating rock for North Gujarat Power Project

4.2 To save costs, the piles were installed using wash boring method with a truck mounted rotary rig (TMR). Three initial pile tests were conducted of which two piles failed and the third pile achieved the required test load of 300T. The third pile was however drilled using bailer chisel method. In the meanwhile, about 200 working piles were also cast with TMR method. At the specific request of the Engineer to the project, the author was requested to do the tests as tests done by a local team showed acceptable integrity for all piles. The integrity results for the two piles that failed in static load tests are attached in Figure 11.

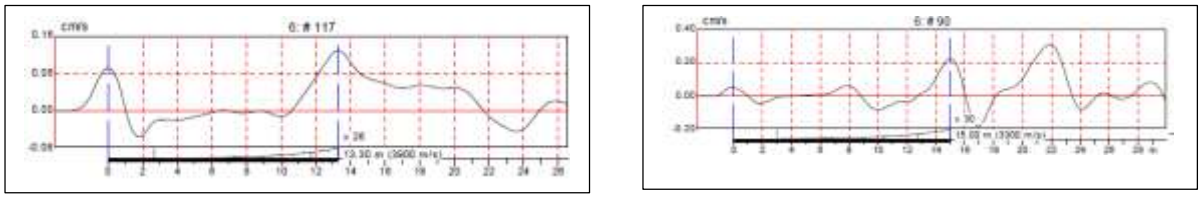


Figure: 11 Results of failed pile integrity with LSIT when for piles cast with TMR method

4.3 A high strain dynamic load test was also conducted on one of the piles and achieved a capacity of only 127T with a total settlement of 30mm. The results are presented in Figure 12.

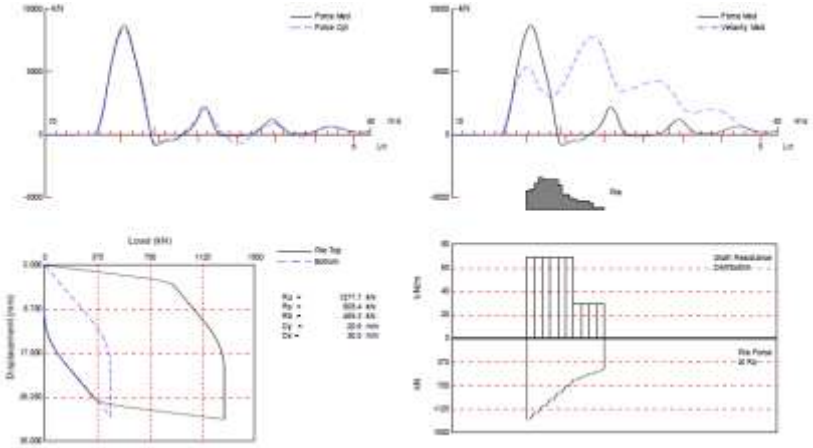


Figure 12 Results of failed HSDPT cast using TMR method

4.4 It was found on further investigation that TMR method cannot drill piles into rock. The method of placement of concrete and flushing was also inadequate and thus most of the piles with TMR were rejected. Eventually new piles were cast with either bailer chisel or a hydraulic rotary equipment and the piles could achieve the required capacity and also good integrity. The results of low strain integrity for one pile and a high strain test are presented in Figure 13 and 14 respectively.

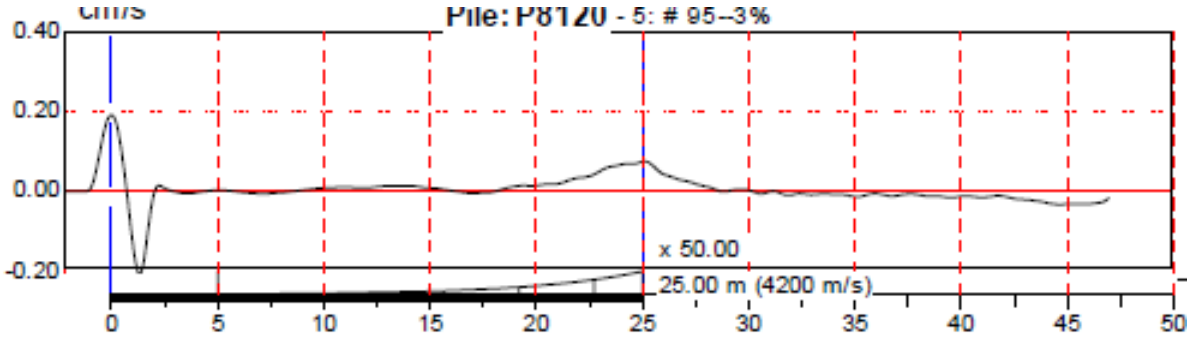


Figure 13 Results of acceptable integrity installed using hydraulic rotary equipment

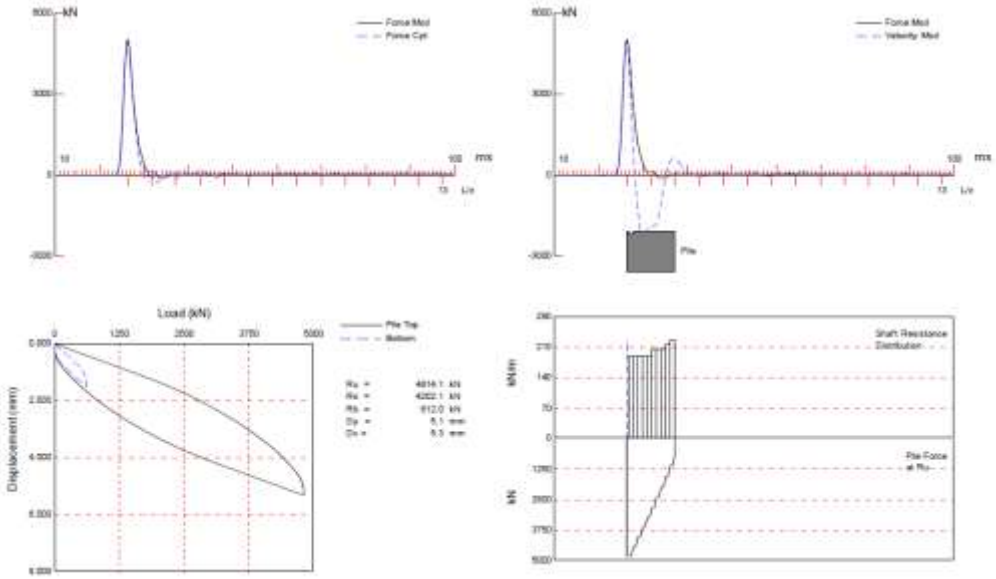


Figure 14 Results of acceptable high strain dynamic load test with full rebound

4.5 It is pertinent to note that the same methods can also be used for requalification of existing piles for old power projects or bridges, jetties that need replacement of super-structures. This is because as long as the pile top can be made accessible, the integrity and capacity of the piles can be easily verified. However, the methods need expertise in data collection and interpretation and thus should be carefully used and analyzed else it may lead to more complicated situations.

5.0 Testing of TG Decks and TG Columns

5.1 It is a standard requirement to test columns of TG Deck and the deck itself by ultrasonic pulse velocity method and in some cases also with the rebound hammer test. Over several years of experience and data collected by the author, it is observed that the rebound hammer has limited use as although it predicts the grade of concrete, the answer is based on standard curves developed in Germany several years ago. These curves may not represent the concrete cast in India. Again there are several rebound hammers available in the market and it will be almost impossible to standardize them. Thus even when used carefully, the results may only be indicative and to use only rebound hammer tests for acceptance is avoidable. The recent IS: 516-Part 5/Sec: 1 code also discourages using rebound hammer tests unless combined with UPV tests.

5.2 The UPV tests when used by the direct method indicate the wave speed through concrete. However, since member thickness is very large, it is not always possible to use direct method to evaluate speed. In such cases, the indirect method of testing is used. In the indirect method, the wave takes the shortest path and thus the results are more based on surface smoothness rather than the strength of concrete. Even the acceptance of concrete with direct methods is based on the table provided in IS: 516(Part: 5/Sec: 1)-2018 which says that any concrete with a speed more than 3700 M/sec is good. However, the code fails to specify the type of structure, application, its age, residual life etc. Thus the values remain same irrespective of the type of structure viz., bridge, nuclear project, power, residential buildings etc. In general, at all projects a reference between cube test results and UPV values is seldom done. In such cases, the UPV testing of TG decks provides very limited information.

5.3 The thermal integrity profiling is an emerging method to determine heat signature in mass concrete. Although temperature monitoring has been used for several years, each thermo coupler needed a single wire and thus marking locations and numbering the wires was a complicated task if several 100 thermo couplers were to be installed. Thus the method had limited application due to installation and data collection constraints. In comparison the TIP wires as shown in Figure 15 are connected in series at every 300mm or 500mm and can be laid within the deck and tied to the rebars before concreting. Each pre-defined wire length has only one data logger which keeps logging the heat signature every 15 minutes or at a user pre-set time. This eliminates the need for individual wires and multiple data loggers as even a few cables running through mass concrete will be sufficient to identify weak pockets within concrete and also provide information on peak temperature.

Figure 15 Thermal Integrity Profiling Assembly with TIP wires, TAP and Readout Unit

5.4 The American Concrete Institute (ACI) defines mass concrete as any volume of structural concrete in which a combination of dimensions, boundary conditions, characteristics of the concrete mixture, and the ambient conditions can lead to undesirable thermal stresses, cracking, deleterious chemical reactions, or reduction in the long-term strength as a result of elevated concrete temperature due to heat of hydration. Structural elements considered as mass concrete may be subject to monitoring to address two main concerns: a. peak temperatures b. differential temperatures.

5.5 Figure 16 shows heat signature for a pile foundation and it is expected that heat signature will be similar at the core of mass concrete and near edges.

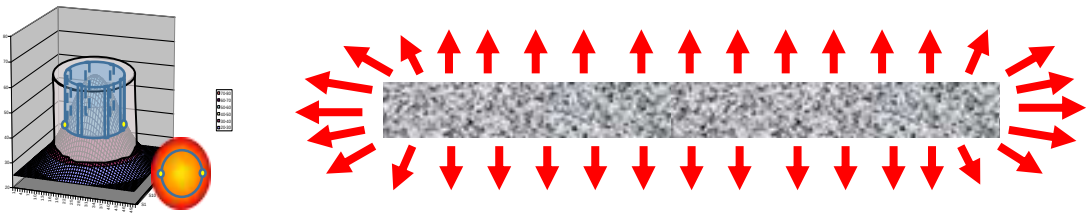


Figure 16 Heat Signature and its dissipation within concrete

A sudden drop of temperature within the core in the first 3-7 days is more likely to be a defect or weak concrete and can be easily identified. This is illustrated in Figure 17 below.

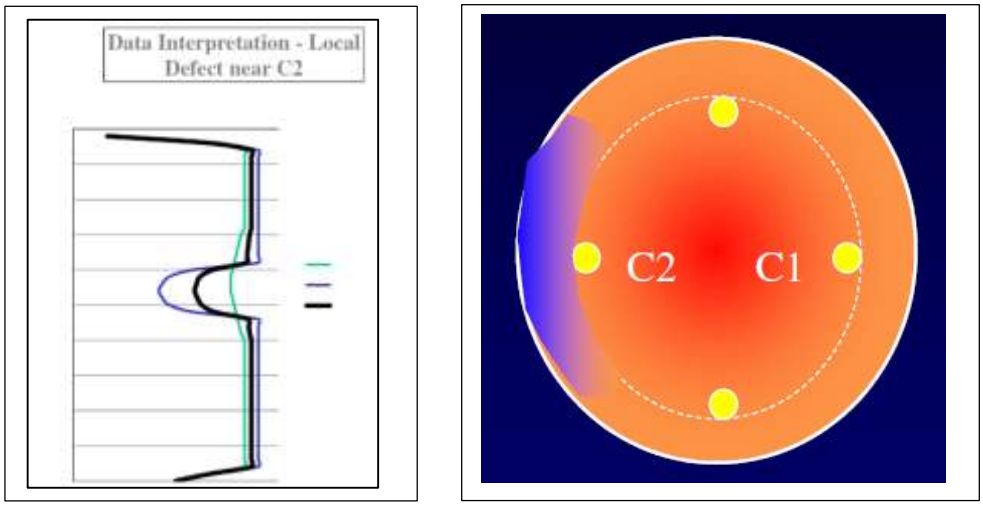


Figure 17 Indication of Defect based on temperature measurements within concrete.

5.6 The methods with thermal measurements of concrete due to recent developments are expected to provide significant information on concrete behaviour immediately after casting within the first few days rather than wait for 28 days before evaluation. Maturity concepts of concrete although not included in this paper is also likely to gain more acceptance in future years. The method is also based on measuring heat signatures within concrete.

6.0 Conclusions

6.1 Bi-Directional Static Load testing increases speed of testing and in several cases reduces cost. The method does not require any major site arrangements and thus can be easily used for testing of pile foundations in remote areas or for large loads or where time is a constraint.

6.2 BDSLT provides information not only about the compression capacity but also the uplift capacity of the pile. Instrumentation in the form of strain gages provides additional information about load transfer and friction end bearing components. Conducting tests without instrumentation is not desirable and should be always followed. The general practice is to provide strain gages at every 5m or layers of soil change.

6.3 Lateral load tests with inclinometers provide significant additional information about point of fixity. The information can be used to optimize design by reducing additional reinforcement which is specifically provided to resist lateral load.

6.4 Both the BDSLT and lateral load tests with inclinometers have significant applications for the power sector, transmission line towers and its use is expected to increase with time.

6.5 High Strain dynamic load tests and low strain integrity tests are widely used in the deep foundation industry to evaluate capacity and integrity respectively. However, the methods are also widely abused. It is important that the Engineer is aware about the methods and has basic ability to understand or read the findings before they are widely adopted. It may be advisable to have a thorough discussion with the testing expert before they are adopted at project sites as poor interpretation may lead to disastrous consequences.

6.6 Temperature measurement of concrete and thermal profiling holds immense promise its use should be encouraged for temperature measurements in mass concrete rather than solely relying on NDT methods.

6.7 A good foundation and structure is a result of careful planning, good design, execution and proper testing. A good foundation can last up to 100 years and some of the methods can be used for re-qualification of foundations for older structures.

About Author

Mr. Ravikiran Vaidya has obtained BE (Civil) in 1991 and ME (Structures) in 1994 from The MS University of Baroda, Vadodara

As a Principal Engineer of Geo Dynamics he has been instrumental in popularizing the concept of Deep Foundation Testing in India. He has created a deep foundation testing industry and today Geo Dynamics is India's premier testing company in this field. His work has resulted in the methods being accepted by all major bodies including metro rails, Highways, Railways, power, marine etc.

He has worked on all almost all the prestigious projects in the country and in more than 20 countries worldwide. He is the recipient of best paper award in Case Histories category. He is also the recipient of the prestigious "**Dinesh Mohan Award**" by the Indian Geotechnical Society for Excellence in Geotechnical Practice for the year 2018-2020.

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Seismic Analysis and Design of Control Room Building of GIS Sub-station by Using STAAD. Pro Software

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ABSTRACT

In Civil Engineering, structural Design has been given quite emphasis due to its significant role in development of society. The structural engineering is the creation of fundamental building components and members, such as slabs, beams, columns, and footings. The design process typically commences with the blueprint for the specific structure, which includes determining the placement of beams and columns. Subsequently, calculations are performed to determine vertical loads, encompassing both the permanent dead load and variable live loads. Once the load values are established, the initial structural element, often the slab, can be designed to bear the loads. The transfer of loads from slabs to beams is integral to the design process. These load transfers may take on a trapezoidal or triangular distribution, influencing the design of the beams accordingly. The loads, primarily in the form of shear forces, are then transmitted from the beam to the column. To design columns effectively, an understanding of the moments to which they are subjected is crucial. Frame analysis is conducted, typically utilizing the Moment Distribution Method. In this project, most of the columns are considered to experience axial loads with uniaxial bending. Finally, the footings are designed based on the loads stemming from the columns and the soil bearing capacity specific to the location. All structural components are subjected to rigorous checks to ensure their strength and stability.

The initial design of the building adhered to the guidelines outlined in IS 456:2000, without accounting for seismic loads, using the STAAD. Pro software. Subsequently, the structure underwent analysis to account for seismic loads, following the equivalent static analysis method. The base shear was determined in accordance with IS 1893:2016.

Keywords: STAAD-pro, Design, Seismic analysis, Building Components, Strength and stability Base Shear, IS 1893:2016

1. INTRODUCTION

For designing structures, authentic software should be selected that has features of cutting edge UI (User interface) with power full visualization tools and robust analysis capabilities. For all engineering discipline these feature is achieved through Finite Element Method (FEM). Author has selected STAAD PRO for the design under reference.

STAAD PRO is a popular choice for the structural design of multi-story buildings, factories, tunnels, bridges, and a wide range of other structures, including those made from steel, concrete, aluminium, and cold-formed steel.

The project work is to analyse a G+1 storeyed Control Room building of GIS sub-station for different load combination using STAAD Pro software.

Grades of Concrete

We use cement, sand, aggregate, and water which are mixed with a certain ratio, and concrete is cast and put in a cube of 150 mm size and put in a water bath for 28 days and afterwards, it is tested in a compression testing machine. The compressive stress result is known as the “grade of concrete“. It is expressed in N/mm².

The “M” refers to Mix and Number after M (M10, M20) Indicates the compressive strength of concrete after 28 days of curing and testing.

If we mention M10 concrete, it means that the concrete has 10N/mm2, characteristic compressive strength at 28 days. In the designation of concrete mix M to the mix and the number to the specified compressive strength of 150mm size cube at 28 days, expressed in N/mm².

The Project Work M25 grade of Concrete used.

➢ 2. LITERATURE REVIEW

Lekkala Harish Kumar: This compilation of various studies explores the processes involved in planning, designing, and assessing structures using different software tools. All the aforementioned studies collectively recommend the adoption of STAAD PRO as the preferred software for structural analysis due to its versatility and its ability to offer cost-effective solutions, whether using steel or concrete materials [11]

Anjana Gupta conducted an analysis and design of a multi-story building using STAAD PRO software. This research paper aims to explore the effectiveness of various civil engineering application software. [12]

Sakib Salam Sofi's project involves the analysis and design of a multi-story hospital building with five floors (G+5) using STAAD PRO software. The primary objective is to assess the seismic design in STAAD PRO and reduce design work time. [13]

Prof. Komal S Meshram worked on the seismic analysis of a multi-story reinforced concrete (RCC) building with seven floors (G+7) using STAAD PRO software. The project report focuses on reviewing and studying the behaviour of multi-story buildings through the Equivalent Static Lateral Force Method. [14]

➢

➢ 3. OBJECTIVE AND SCOPE

Gas Insulated Substation (Switch gear) room is a very important civil structure in Power Transmission. The design of the building has to be earthquake resilient to a greater extent. Any damage to the civil structure may damage the very expensive GIS equipment. Besides the SF_6 gas is hazardous to the human life.

The study will adhere to Technical Specification to identify project-specific requirements/details and the guidelines outlined in the Indian Standard Code IS 1893-2002 and focus on earthquake-resistant design principles. Specifically, the study involves performing an Equivalent Static Analysis for a single-story RCC control room building at one of the 220kV GIS Substation of Gujarat, situated in close proximity to Vapi, Gujarat. This analysis will be carried out out using Structural Analysis and Design software, STAAD PRO.

Computer aided analysis and design of residential building by using STAAD PRO Includes:

- Generation of structural framing plan.
- Creation of model of structure in STAAD PRO.
- Application of various load combinations on the member.
- Analysis of the structure.
- Design of the structure.

➢ 4. MATERIAL AND METHODOLOGY

The plan of the Control room Building is regular. It has a story height of ground floor H = 4.25M whereas first floor H=4.20M height. The Control Room building length is 26.70M (including Porch) and width is 16.46M so the area is 378.878 M^2

The building consists of square columns with cross- section (0.55x0.55)M, rectangular beams with cross- section (0.7 x 0.3/0.75 x 0.3/0.5 x 0.3)M and slab thickness of 150mm. Table 1 shows the methodology whereas Fig. 1 shows the designing Process.

Table 1: Methodology

Utility of building	Control Room building
No. of Storey	G+1
Grade of Concrete	M25
Grade of Steel	Fe500
Type of Steel Bars	HYSD
Unit weight of Concrete	25kN/M^3
Storey Height Ground Floor	4.25M
Storey Height First Floor	4.20M

Value	Dead and Live Load
Ground Floor	
25.39 kNM	Wall Load (230thk.)
12.70 kN/M	Wall Load (Partition)
5.89 kN/M	Aluminium Partition wall
First Floor	
19.87 kN/M	Wall Load (230thk.)
9.94 kN/M	Wall Load (Partition)
5.64 kN/M	Aluminium Partition wall
3.75 kN/M^2	Slab load (dead load)
2.40 kN/M^2	Floor finish (dead load)
10.00 kNM^2	Live load

First Floor & Mumty Floor	
4.97 kN/M	Wall Load (230thk.) parapet
3.75 kN/M^2	Slab load (dead load)
3.60 kN/M^2	Floor finish (dead load)
1.50 kN/M^2	Live load terrace
0.75 kN/M^2	Live load mumty area
5.0 kN/M^2	Live load on all stairs

STEP-1

Preparation of Building

STEP-2

Plan of Control Room Building using Auto-CAD

STEP-3

Analysis and Design of Building using STAAD PRO

Figure:1 Steps for Design

The structure is analysed and designed for live load, seismic load as per IS-1893-2002 and dead load consisting of self-weight beams, columns and slabs. The following Table show the different loads acting on the building

➢ 5. STRUCTURAL MODELING

The Control Room Building has been engineered to withstand a range of loads, including gravity loads such as dead load and live load, along with combinations of these forces. This design also integrates structural components at the extremities. The building in question is a one-story, standard reinforced concrete construction. The beams in the transverse direction (X) exhibit varying lengths, with the longest spanning 7.23 meters. Likewise, beams in the longitudinal direction (Z) have different lengths, with the maximum extending to 7.30 meters. Figure 2 visually depicts the layout of the ground floor and first floor of the control room building. Additionally, Figure-3 presents a graphical representation of the STAAD PRO model, and Figure-4 offers a three-dimensional depiction of the structure

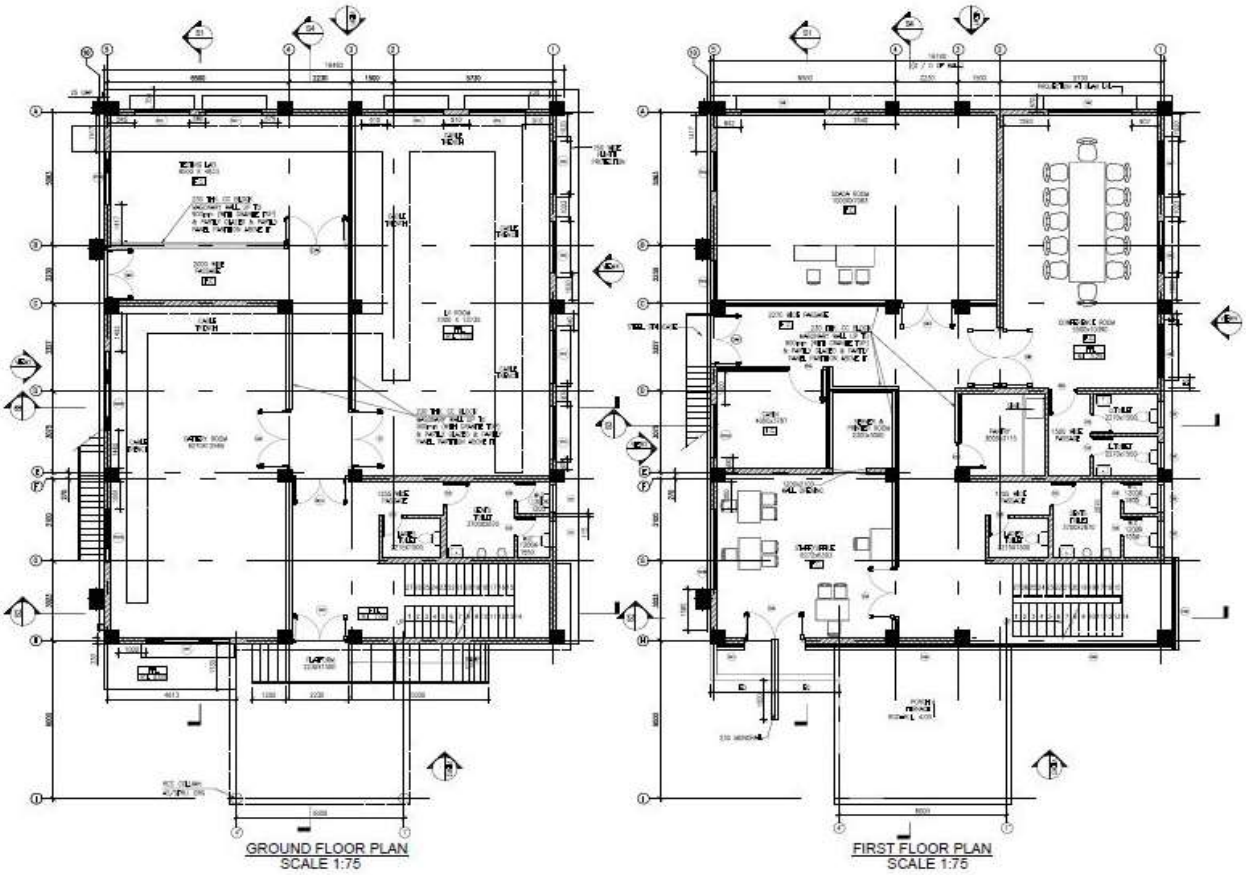


Figure 2: Typical Ground Floor *and First Floor Plan of Control Room Building*

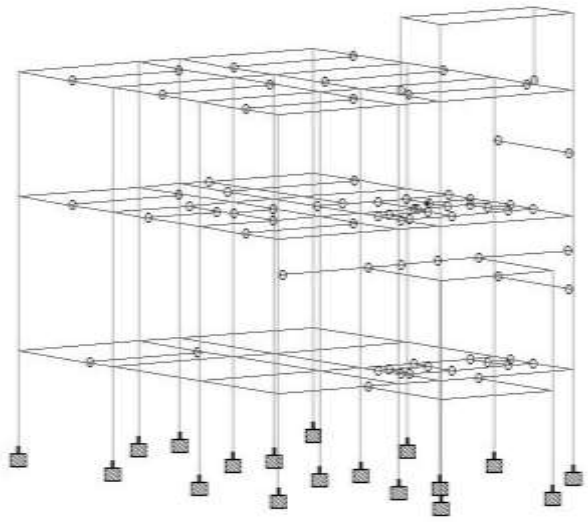


Figure 3: Typical STADD PRO modelling

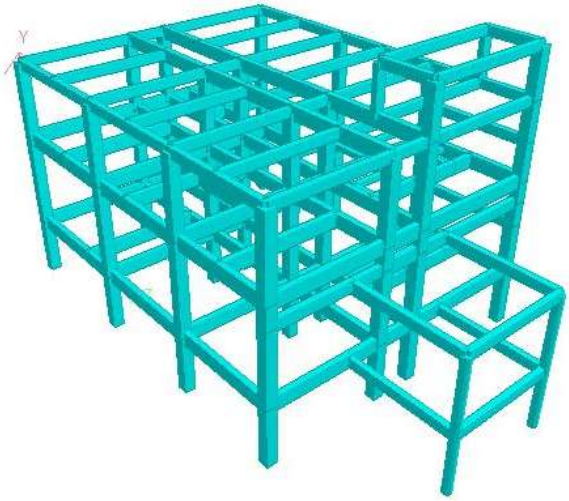


Figure 4: Typical 3D form of the structure

Load Assignment:

1. Dead load

The dead load comprise the weight of walls, partitions floor finishes, false ceilings, floors and the other permanent standing construction in the buildings. The dead load loads are estimated from the dimensions of various members of building and their unit weights. The unit weights of plain concrete and reinforced concrete taken as 25kN/M^3 .The unit weight of masonry taken as 20kN/M^3. As per IS:1893 (Part 1)-2016, the dead load have been assigned on the basis of member load, floor load, self-weight of the beams definition. Figure-5 demonstrates the distribution of the dead load on the structure.

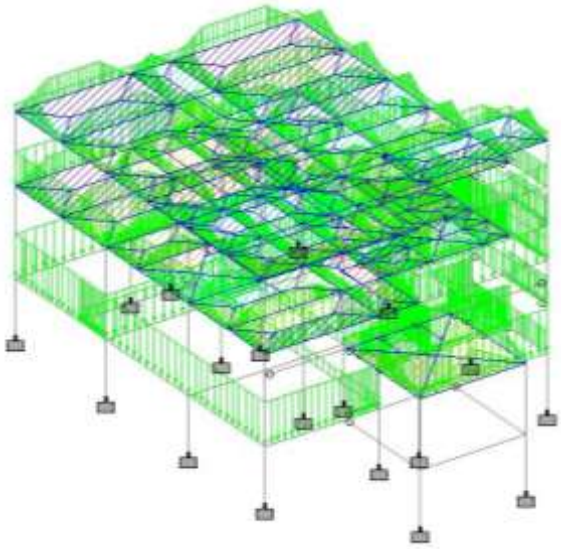


Figure 5: Dead Load acting on the structure

- 2. Live load

As per IS:875 (Part-2)-1987 and Technical Specification, live load has been assigned to the members. Figure-6 illustrates the live load applied to the structure

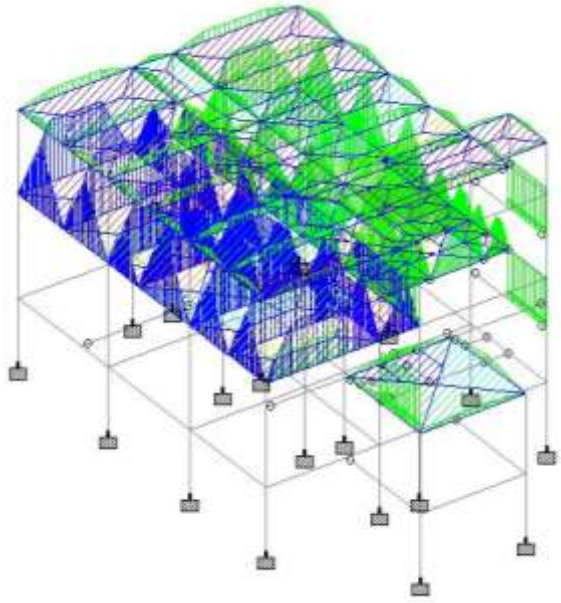


Figure 6: Live Load acting on the structure

- 3. Seismic Load
-

After defining the seismic load as per requirement of IS: 1893 (Part-1): 2016, the seismic load has been assigned with respect to +X, -X, +Z, and –Z directions with their respective appropriate seismic factor Figure 7 & Figure 8 Shows Seismic load acting transvers

direction (X) and longitudinal direction (Z) respectively on the structure.

A. Seismic weight of building (W)

Seismic weight of each floor is its full dead load plus appropriate amount of imposed load. Imposed load are considered as 50% due to Live load actin more than 3.0 kN/M^3

- B. Time Period

The fundamental natural period of a vibration (Ta), in seconds, it may be estimated by the empirical expression. T_{ax} = 0.25s and T_{az} = 0.28s

$$Ta = \frac{0.09h}{\sqrt{d}}$$

Where

h = the height of the building, in meters.

d = Base dimension of building at the plinth level along the considered direction of earthquake shaking, in meter.

- C. Determination of Design Base Shear

Design seismic base shear, $V_B = A_h W$, Figure-11 Shows the Base Shear value for Transverse direction (X) and Longitudinal Direction (Z).

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{Sa}{g}\right)}{\left(\frac{R}{I}\right)}$$

Where

Ah = Design horizontal seismic coefficient.

Z = 0.36; seismic zone factor given in Table-3 for Zone V

Table-3 Seismic Zone Factor Z

Seismic Zone Factor (1)	II (2)	III (3)	IV (4)	V (5)
Z	**0.10**	**0.16**	**0.24**	**0.36**

I=1.5; Importance factor given in IS: 1893 (Part1-5) for the corresponding structures; when not specified, the minimum value of I shall be,

a) 1.5 for critical and lifeline structure,

b) 1.2 for business continuity structures,

c) 1.0 for the rest.

R=5; response reduction factor given in IS 1893 (Table-9) for the corresponding structures.

$\frac{Sa}{g}$ = 2.5; the design acceleration coefficient considered as per Indian Standards for design, as per IS 1893 (Part-1): 2016.

a) For use in equivalent static method [*see* Fig. 2(a)]:

$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} & \begin{cases} 2.5 & 0 < T < 0.40 \text{ s} \\ \frac{1}{T} & 0.40 \text{ s} < T < 4.00 \text{ s} \\ 0.25 & T > 4.00 \text{ s} \end{cases} \\ \text{For medium stiff soil sites} & \begin{cases} 2.5 & 0 < T < 0.55 \text{ s} \\ \frac{1.36}{T} & 0.55 \text{ s} < T < 4.00 \text{ s} \\ 0.34 & T > 4.00 \text{ s} \end{cases} \\ \text{For soft soil sites} & \begin{cases} 2.5 & 0 < T < 0.67 \text{ s} \\ \frac{1.67}{T} & 0.67 \text{ s} < T < 4.00 \text{ s} \\ 0.42 & T > 4.00 \text{ s} \end{cases} \end{cases}$$

- D. Vertical Distribution of Base Shear

The design base shear (V_B) computed shall be distributed along the height of the building as per the expression

$$Qi = V_B \frac{Wihi^2}{\sum_{i=1}^{n} Wihi^2}$$

Where

Qi = Design lateral forces at floor i,

Wi= Seismic weights of the floor i,

hi = height of the floor, measured from base, and

n = Number of stories.

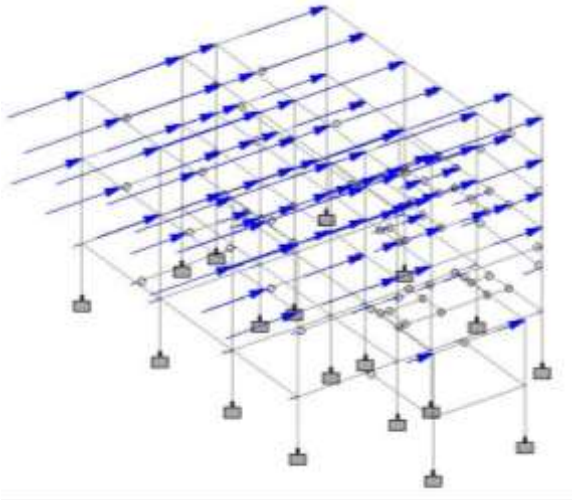


Figure 8: Seismic Load acting along transverse direction (X) of the structure

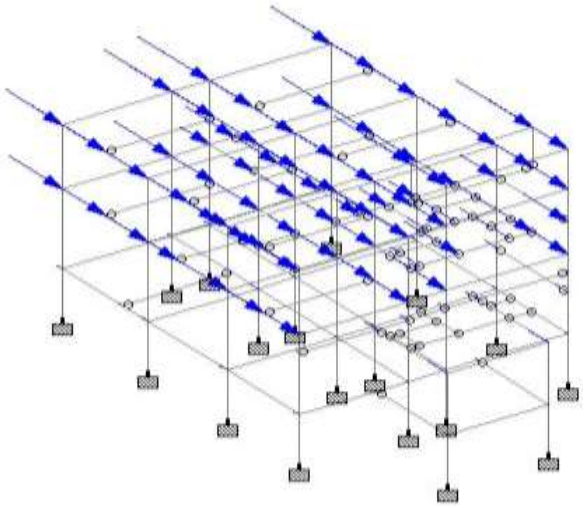


Figure 9: Seismic Load acting along along transverse direction (X) of the structure

- 6. STRUCTURE ANALYSIS

The analysis options are set before actual analysis. The analysis is performed with a scale factor 1. The number of modes is initially set as 1.5 after analysis. If the cumulative mass participation factor is less than 95%, then it is modified accordingly with base shear values obtained for the earthquake load case, the new scale factor is calculated and again the model is analysed for the new scale factor.

Figure-9 Shows the Base Shear value for Transverse direction (X) and Longitudinal Direction (Z). The bending moment diagram of the structure is displayed in Figure-10, and the shear force diagram can be found in Figure-11. However, it can be observed that the base shear value calculated from the code and by the software with the new scale factor are the same. And the combination load is applied in STAAD PRO by using formula which are given below.

1. 1.5(DL+IL)
2. 1.2(DL+IL+-EL)
3. 1.5(DL+ - EL)
4. 0.9DL+ - 1.5EL

Time Period for X 1893 Loading = 0.27600 sec.
SA/G per 1893 = 2.500 Load Factor = 1.000
VB per 1893 = 0.1350 x 17,392.34 = 2347.97kN

Time Period for Z 1893 Loading = 0.24600 sec.
SA/G per 1893 = 2.500 Load Factor = 1.000
VB per 1893 = 0.1350 x 17,392.34 = 2347.97kN

Figure 9: Base Shear Values for the Structure from Staad pro for Transverse direction (X) and Longitudinal Direction (Z).

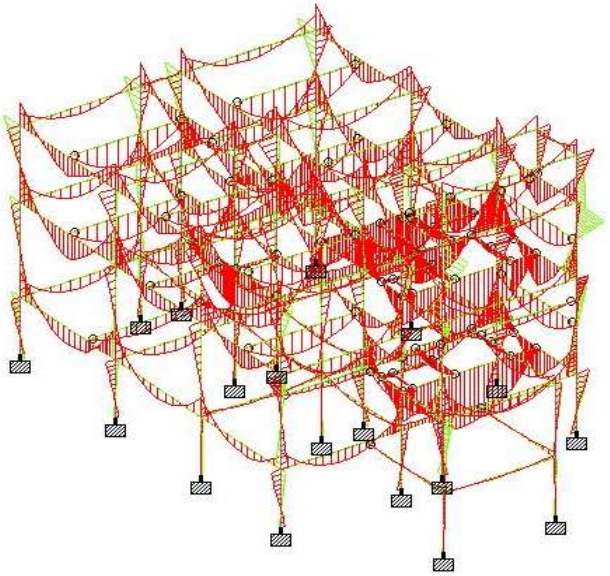


Figure 10: Bending Moment diagram of the structure

- 7. STRUCTURE STAAD PRO DESIGN RESULT

After that Staad analysis output file is generated which containing the structural design of each individual beam and column member of structure

- BEAM DESIGN RESULT FROM STAAD PRO

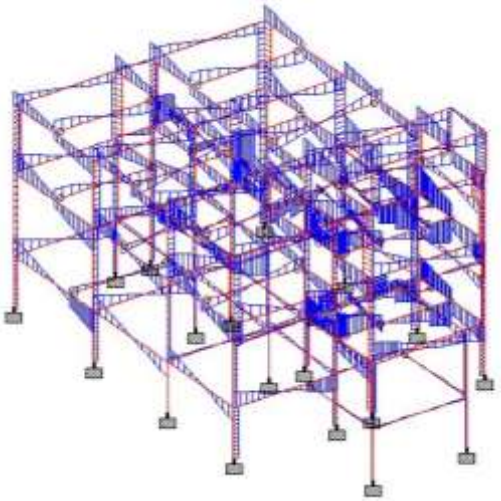


Figure 11: Shear Force diagram of the

- 7. STRUCTURE STAAD PRO DESIGN RESULT

After that Staad analysis output file is generated which containing the structural design of each individual beam and column member of structure

- BEAM DESIGN RESULT FROM STAAD PRO

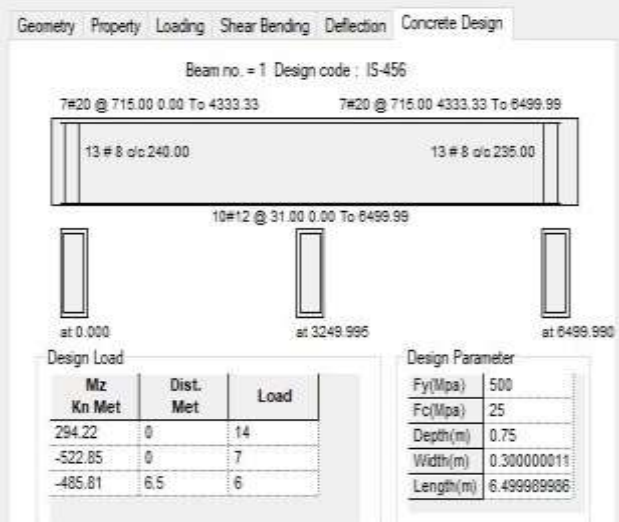
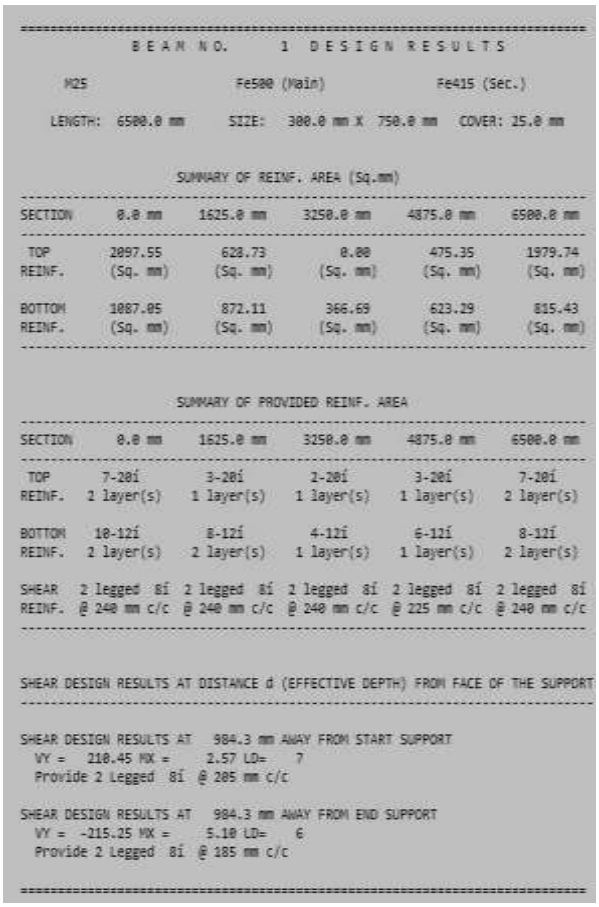


Figure 112: Beam Design showing in Staad pro Post-Processing Part



```
=============================================================================
            B E A M  N O.      1   D E S I G N   R E S U L T S

     M25                  Fe500 (Main)              Fe415 (Sec.)

   LENGTH:  6500.0 mm     SIZE:   300.0 mm X  750.0 mm   COVER: 25.0 mm


                    SUMMARY OF REINF. AREA (Sq.mm)
-----------------------------------------------------------------------------
SECTION      0.0 mm     1625.0 mm     3250.0 mm     4875.0 mm     6500.0 mm
-----------------------------------------------------------------------------
 TOP       2097.55        628.73          0.00        475.35       1979.74
REINF.     (Sq. mm)      (Sq. mm)      (Sq. mm)      (Sq. mm)      (Sq. mm)

BOTTOM     1087.05        872.11        366.69        623.29        815.43
REINF.     (Sq. mm)      (Sq. mm)      (Sq. mm)      (Sq. mm)      (Sq. mm)
-----------------------------------------------------------------------------


                  SUMMARY OF PROVIDED REINF. AREA
-----------------------------------------------------------------------------
SECTION      0.0 mm     1625.0 mm     3250.0 mm     4875.0 mm     6500.0 mm
-----------------------------------------------------------------------------
 TOP        7-20í         3-20í         2-20í         3-20í         7-20í
REINF.    2 layer(s)    1 layer(s)    1 layer(s)    1 layer(s)    2 layer(s)

BOTTOM     10-12í        8-12í         4-12í         6-12í         8-12í
REINF.    2 layer(s)    2 layer(s)    1 layer(s)    1 layer(s)    2 layer(s)

SHEAR   2 legged  8í  2 legged  8í  2 legged  8í  2 legged  8í  2 legged  8í
REINF.  @ 240 mm c/c  @ 240 mm c/c  @ 240 mm c/c  @ 225 mm c/c  @ 240 mm c/c
-----------------------------------------------------------------------------


SHEAR DESIGN RESULTS AT DISTANCE d (EFFECTIVE DEPTH) FROM FACE OF THE SUPPORT
-----------------------------------------------------------------------------

SHEAR DESIGN RESULTS AT   984.3 mm AWAY FROM START SUPPORT
  VY =   210.45 MX =     2.57 LD=    7
  Provide 2 Legged  8í  @ 205 mm c/c

SHEAR DESIGN RESULTS AT   984.3 mm AWAY FROM END SUPPORT
  VY =  -215.25 MX =     5.10 LD=    6
  Provide 2 Legged  8í  @ 185 mm c/c

=============================================================================
```

Figure 113: Beam design in detailed in Staad pro output file

- COLUMN DESIGN RESULT FROM STAAD PRO

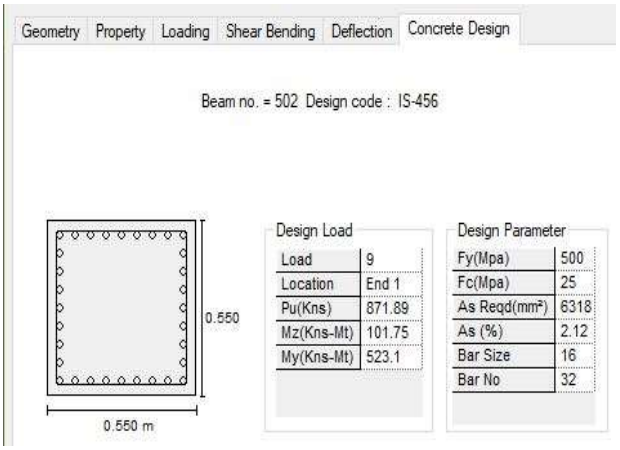
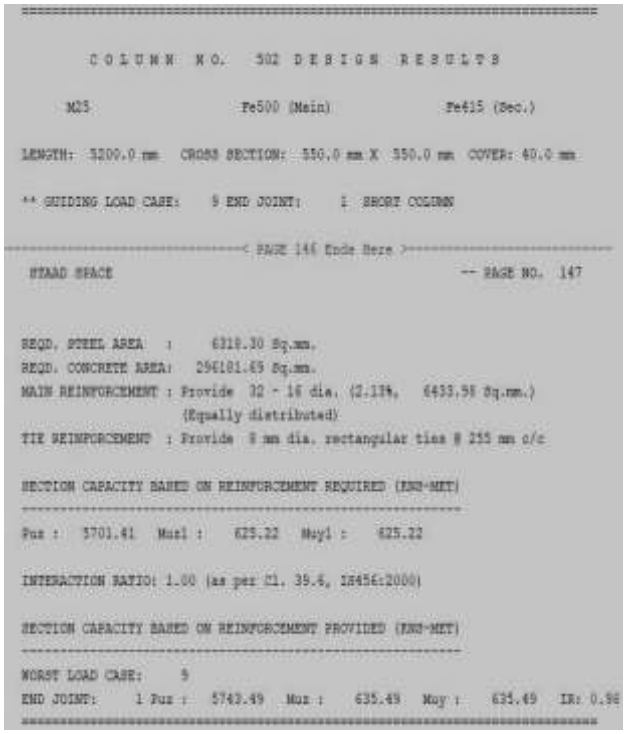


Figure 14: Column design showing in Staad pro Post-Processing Part



```
=============================================================================

        C O L U M N   N O.   502   D E S I G N   R E S U L T S

     M25                  Fe500 (Main)              Fe415 (Sec.)

LENGTH:  3200.0 mm   CROSS SECTION:  550.0 mm X  550.0 mm  COVER:  40.0 mm

** GUIDING LOAD CASE:     9 END JOINT:     1  SHORT COLUMN

-----------------------------< PAGE 146 Ends Here >----------------------------

 STAAD SPACE                                          -- PAGE NO.  147


REQD. STEEL AREA  :     6318.30 Sq.mm.
REQD. CONCRETE AREA:   296181.69 Sq.mm.
MAIN REINFORCEMENT : Provide  32 - 16 dia. (2.13%,   6433.98 Sq.mm.)
                     (Equally distributed)
TIE REINFORCEMENT  : Provide  8 mm dia. rectangular ties @ 255 mm c/c

SECTION CAPACITY BASED ON REINFORCEMENT REQUIRED (KNS-MET)
-----------------------------------------------------------------------------
Pux :   5701.41   Mux1 :    625.22   Muy1 :    625.22

INTERACTION RATIO: 1.00 (as per Cl. 39.6, IS456:2000)

SECTION CAPACITY BASED ON REINFORCEMENT PROVIDED (KNS-MET)
-----------------------------------------------------------------------------
WORST LOAD CASE:     9
END JOINT:     1 Pux :   5743.49   Mux :    635.49   Muy :    635.49   IR: 0.98
=============================================================================
```

Figure 15: Column design in detailed in Staad pro output file

Staad Pro provides design recommendations for beams and columns based on the analysis data presented in Figures 12 and 13 for beam design and Figures 14 and 15 for column design.

- **DESIGN OF SLAB**:

Slabs are the most commonly employed structural components that constitute the floors and roofs of buildings. They primarily bear transverse loads and transmit them to the supports through bending actions in one or more directions. Based on their spanning direction.

Based on their spanning direction, slabs can be categorized into two types: one-way slabs and two-way slabs. However, the design of slabs shall be carried out manually.

One-way slab: When a slab is upheld by two parallel edges on opposite sides, it exclusively spans across directions that are perpendicular to these supporting edges. It undergoes bending in a single direction, and the primary steel reinforcement is positioned along the span directions. This type of slab is commonly referred to as a one-way slab.

Two-way slab: A two-way slab is characterized by its support on all four edges and a load distribution that also occurs on all four edges of the panel, necessitating the placement of reinforcement on both sides.

DESIGN OF STAIRCASE

The primary function of a staircase is to facilitate pedestrian access within a building. The specific design and configuration of a staircase can vary significantly based on the unique circumstances of the building. Two key factors typically influence the shape and structural layout of a staircase. However, it's important to note that the staircase design is typically a manual process.

- The first factor is the type of construction used in the structure surrounding the staircase, which can be a non-load-bearing brick structure or a reinforced concrete framed structure.
- The second factor is the availability of space. In the case of the proposed building, a bifurcated staircase is being employed, consisting of two separate flights. The first flight spans from the plinth level to the lintel level, while the second flight extends from the lintel level to the roof level.

DESIGN OF FOUNDATION

- Foundation is a critical component of any building or structure, it plays a pivotal role in transmitting the loads from the structure to the mother soil. These loads can be caused by the building's own weight, as well as additional forces such as wind, earthquakes, or live loads (e.g., people and furnishings). Without a properly designed foundation, the structural integrity and safety of the entire building could be jeopardized.

One of the primary objectives in foundation design is to prevent excessive settlement and rotation.

- Settlement occurs when the foundation sinks into the soil, which can lead to uneven settling of different parts of the building, causing structural damage or even collapse.
- Rotation, on the other hand, involves the tilting or leaning of the structure. Both settlement and rotation can result from inadequate foundation design, and they must be minimized to ensure the safety and functionality of the building.

Foundations must also provide safety against sliding and overturning. Adequate measures, such as anchoring or using friction-resistant materials, must be implemented in foundation design to counteract these forces and ensure that the building remains stable, especially in the face of environmental factors like wind or seismic activity.

- Sliding refers to the horizontal movement of the structure, which can occur if the foundation is not anchored properly.
- Overturning involves the tipping or tilting of the building.

The ultimate goal of foundation design is to create a stable, safe, and long-lasting base for the entire structure. This involves a deep understanding of the interaction between the building and the soil, as well as the use of appropriate engineering methods and materials to ensure that the foundation can withstand the stresses and forces it will encounter over period of time. By doing so, engineers can guarantee the safety, integrity, and longevity of the building while minimizing the risk of structural problems related to settlement, rotation, sliding, or overturning.

- 8. EXECUTION PHOTO GALLARY
-

- 9. CONCLUSION

In the present study, Control Room Building has been planned as per electrical requirement and designed (Beams, Columns, Slab, Stairs, Footings and Seismic load analysis by using Equivalent Static method) using STAAD Pro software.

The dead load, live load and earthquake loads are calculated using IS: 456-2000 and IS 1893: 2002. Concrete grade M25 and HYSD bars Fe500 as per IS: 1786-1985 are used.

- Using STAAD Pro the analysis of multi storey building is completed much quicker when compared with manual analysis (Kani's method).
- Designing using Software like STAAD reduces a lot of time in design work. Besides it facilitates optimization.
- Details of each and every member can be obtained using STAAD pro.
- Accuracy is improved by using software.
- The analysis and design has been done with the help of STAAD Pro using Indian standard code.

About Athours:

Er. Pinkesh M Dalal is a Deputy Engineer (Civil) presently working in GETCO, CO.

He graduated in Civil Engineering from Dharamsinh Desai University, Nadiad in 2013 and joined GETCO, as Junior Engineer (Civil). His area of work was Construction of Sub-station Project Work, Construction of Line Tower Foundation and Land Acquisition.

He was elevated to Deputy Engineer in 2022 and is now involved in RCC Structural Designing and Sub-station (GIS/AIS) Designing Work.

Er. Pradip P Shah is a retired Executive Engineer (Civil) GETCO.

He passed his Diploma in Civil Engineering from the MS University of Baroda, Vadodara in 1994 and joined erstwhile GEB as a Junior Engineer (Civil).

During his tenure as Deputy Engineer, he was in-charge of Design and Engineering of Transmission Lines up to 400kV and sub-station, Towers & Foundations.

After superannuation in 2023, he started working as Consultant providing services of design, engineering & supervision of Transmission Lines & substations up to 765kV. He is also a visiting Faculty at GETRI.

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He was elevated to Deputy Engineer in 2021 and is now involved in construction of 66kV/ 220kV/400kV projects, repair and maintenance. He also handled land acquisition, site surveying, estimating, planning, execution, supervision, quality control, billing, contract closing, etc. He is also designing RCC structures.

Civil related O&M challenges in GIS cable Cellar Rooms, Data Centers and Substation Control Panel Room

Jagdish Sandhanshiv,
Director Knowledge Cluster (Care4Cable)

Summary - The weakest civil points in basement areas are those where utilities like power cables, sometimes pipes are entering in the building premises. This gap between cables and wall is sealed using some kind of mortar. However over the period of time the mortar develops cracks and water leakage starts. A sump pit and dewatering system is in design to take out the water that is leaking and accumulating. The stagnant water at the basement is a source of mosquitos and health hazards to the people working near the premises. The problem becomes so difficult that during every or alternate pre monsoon season the civil O&M department has to spend money on rework or during monsoon deploy additional pumps to pump out water. During the Care4cable workshop on high voltage underground cables we came across this issue and solution, however lack of coordination between civil and electrical is making this issue tedious/ complex to solve, time consuming, risk to life and sometimes impossible to solve. Author wants to highlight this gap and discover best practices worldwide to avoid this in India.

Introduction

Heavy water leakage through underground cable entry wall penetrations at New GIS Substation in a Utility. Attempts to seal the space between cables and wall with various mortars and sealants could not stop the leakage. It was a serious issue for their project department. This rarely happens in project stage, normally these issues comes up after the project is handed over to O&M.

Power sector projects are completed as per engineering drawings and coordination among contractors, consultants, quality and project teams. After completion of the project it is handed over to the Civil O&M team and then these buildings are actually tested by multiple rainy seasons / floods.

The weakest civil points in basement areas are those where utilities like power cables, sometimes pipes are entering in the building premises. After cables are installed by the cable department the task of sealing the gap between cables and wall is with the civil department. This gap between cables and wall is sealed using some kind of mortar. However over the period of time the mortar develops cracks and water leakage starts.

Rodents - Lizard, Snakes, Mouse gets a passage through small underground openings. Not only our design or implementation risks the life of rodents but life of service personnel working arond in the surrounding. And this can cause supply disruption

Problem Examples -

A new GIS substation and cables entering through a regular circular sleeve is good. However the number of cables are so dense that there is no space between the cables that will stop water entry from outside in. The cable fill % in this sleeve is almost 90%. This could have been avoided and maintained less than 70%.

Since this is below ground level and the outside trench was full of water, heavy water leakage was observed even before the project was handed over to the Operation and Maintenance team. The solution story is not the scope of this paper; the same can be shared during the presentation.

Problem Example of wall openings

EHV transmission substation Ladakh

Data Center Chennai

The trench which is a channel for taking cables from outside of building to inside of control panels is also a door to water ingress and rodent entry. In most of the substations these trenches are filled with water.

The gap / problem can be clearly seen on the LHS image. On RHS image when problem is solved using conventional mortar / contractor option the water ingress problem reappears.

At Mumbai metro receiving station, through a trench few cables are entering the control panel room through a 1M x 1M opening below ground. Right hand image shows what was actually expected.

Before Condition

Modified/Corrected by LeakXpert.com

If civil provides sleeves for individual cables, the cable installation team has observed limitations to route the cables through those sleeves. Cases are observed where such sleeves are provided, and the wall was broken for cable entry. So this needs more study of success stories of how individual cables (heavy and having low bending radius) can be routed through individual sleeves.

O&M Challenges

1. The mortar develops cracks over the period of time so this becomes a recurring maintenance issue to solve during pre-monsoon work. The problem becomes so difficult that during every or alternate pre monsoon season the civil O&M department has to spend money on rework
2. Using PCC or heavy design of work to seal is not the solution. In fact breaking of old material with live cables becomes a hazard for rework. However the same practice is used.
3. Even after sending pre monsoon funds on the work most of the time the issue remains unresolved due to complexity of work, wrong material, wrong method etc.
4. During the rainy season, if the dewatering pumps are not sufficient to pump the water out additional pumps need to be deployed.
5. The stagnant water at the basement is a source of mosquitos and health hazards to the people working around the premises
6. At some places Sewage water, rodents also enter the basement through cables.
7. This issue gets highlighted up to top management and remains a lifetime pain for the management in cities that get flooded like Mumbai, Vadodara etc.
8. The stagnant water increases moisture/humidity in control and panels and sometimes causes partial discharge or tripping.

While working on the Care4Cable underground cable workshop, we came across these problems. One of the Care4Cable cluster associates Polywater USA is from Minnesota where temperatures are -40 degree C. The R&D solution factory designed a solution to solve such problems in the USA. However we learnt that the problems outside India are relatively very simple geometries.

Route cause

1. In substation control rooms, trenches entering the control room are left open without a barrier wall. Or poor efforts are made in design specifications or detailed engineering, so sealing of this opening is left to brick and mortar work which can never solve the problem. During O&M, customers land up paying multiple times more than what initial cost.
2. A sump pit and dewatering system is in design to take out the water that is leaking and accumulating. This seems to be an easy escape to avoid detail engineering to have a 100% leak proof system.
3. Lack of detailed coordination between civil design and electrical design to address these civil O&M issues at the project stage.

Solutions

1. Awareness - Designers from Civil Projects and Electrical design should pay visits to minimum five sites of GIS Cables Cellar rooms, flooded trenches of control panel rooms, Data centers etc. to witness the real life situation during monsoon.
2. Survey - Best practices from across the country and all over the world should be studied to simplify this complex issue.
3. More detailed engineering should be done for cable grouping, routing and wall crossing.
4. Coordination between electrical and civil is important to have regular geometries to seal instead of a very complex geometry.
5. The sealing material should be compatible with cable jacket material.

Solution: 1 -Conventional mortar sealing. This material is not suitable for cable jacket material. It seals the gap but water still leaks.

Solution: 2 - Polywater USA has designed FST250 material that expands 10 times. It not only seals gaps but during expansion reaches and fills all small cavities around the wall insides and cable outsides for lifetime. One product code for all types and sizes of cables and ducts. It not only seals regular shapes but irregular and odd geometries too.

Another solution is mechanical seals by companies like Hauff Technik Solutions /MCT Brattberg / Roxtec. These are mechanical seals with a better aesthetic appearance. However, solutions with multiple part codes need to be ordered and are specific to the job.

About Author

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Jagdish Shankar Sandhanshiv is Graduate Electrical Engineer from Walchand College of Engineer, Sangli having 23 years of working experience in Power Sector. He is Director of Knowledge Cluster initiatives like Trafocare – workshop on transformers and Care4Cable Underground Cable Workshop.
www.care4cable.org www.trafocare.org

Engineering Support for Aviation Work for Heli-Lifting of Transmission Towers

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1.0 INTRODUCTION

1.1 Erection of transmission lines has been always a challenging activity in transmission line industry due to various reasons like non-accessibility of tower locations, difficult terrain, large space required etc.

1.2 Conventional Tower erection methodology includes:

A. Build-up method: A process in which tower is erected using individual member erection manually by using winch taking long time.

B. Ground assembly method: A method in which the complete tower is assembled on ground horizontally and then raised / erected using large cranes.

C. Section method: This is similar to ground assembly, but in sections.

1.3 The above methodologies require more time, infrastructure and large space requirement and most difficult part being to bring large cranes in inaccessible / difficult tower locations.

1.4 This complex constrains have led industry to develop a more recent and faster methodology called Helicopter method. The project aim was to convert tower design into Heli lifting design concept by converting into helicopter lift able segments.

2.0 REQUIRMENT FOR HELICOPTER METHOD

2.1 Helicopters are being used for project constructions in several countries. The helicopters are efficient in carrying out various project activities, especially in difficult terrains and where time saving is key to the projects. Few examples for such activities are field survey, support to construction and erection works, O&M tasks, safety support etc.

2.2 Erection of towers in segments using Helicopter brings advantages like faster erection, mitigation of large space requirement at each tower location etc. however; it has its own challenges.

3.0 CHALLENGES IN THE ASSIGNMENT

3.1 The helicopter proposed for this assignment is Super Puma with its optimal lifting capability around 3.5 to 4.5 MT/trip.

3.2 Bifurcation work on ready-made Towers, which were already designed and detailed structural drawings, with limitation of 4.5MT.

3.3 A total four types of tower with 19 different configurations are worked out for bifurcation into various segments, introducing temporary members to avoid large deflection / deformation of main legs.

4.0 DETAILS OF ACTUAL WORK:

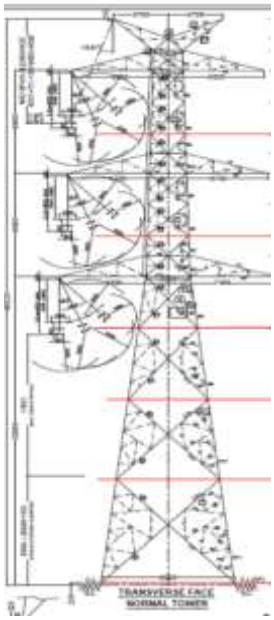
4.1 Design Phase:

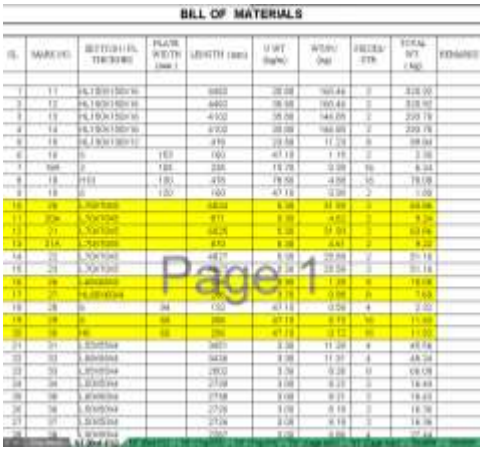
4.1.1 A target maximum lift weight of 4500 kg was used to determine required bifurcation points of the tower. This includes weight of tower sections, galvanizing, plates and hardware, slings, rigging equipment, guides, and stops. Existing splice locations were utilized as bifurcation points where possible.

4.1.2 The Weight of banana guides is taken as 50 kg each (four installed per section, for a total of 200 kg). Weight of slings, rope lead lines, and hardware taken as approx. 50 kg total per lift.

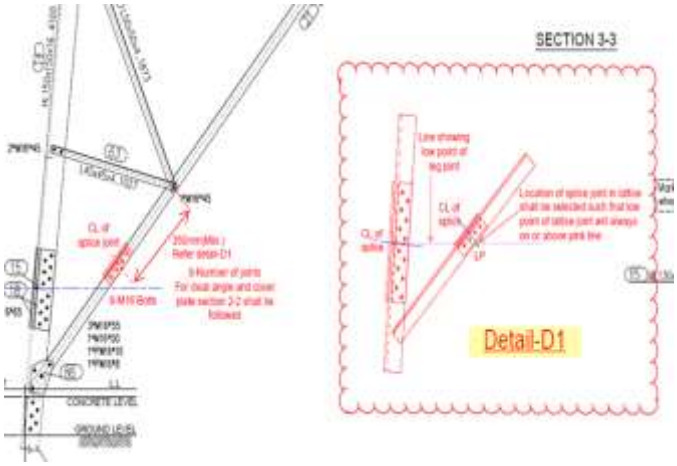
4.1.3 Design Methodology: Preparation of Methodology in line with global practice (Workable with all Super Puma Operators). Heli lifting philosophy: (3.5MT to 4.5 MT Limitation philosophy/ lift as per ground conditions)

4.1.4 Tower Bifurcation: Bifurcation philosophy (Marking on drawings, location of Butt joints and temporary member's requirement considering 3.5MT to 4.5 MT capacity of lifting)

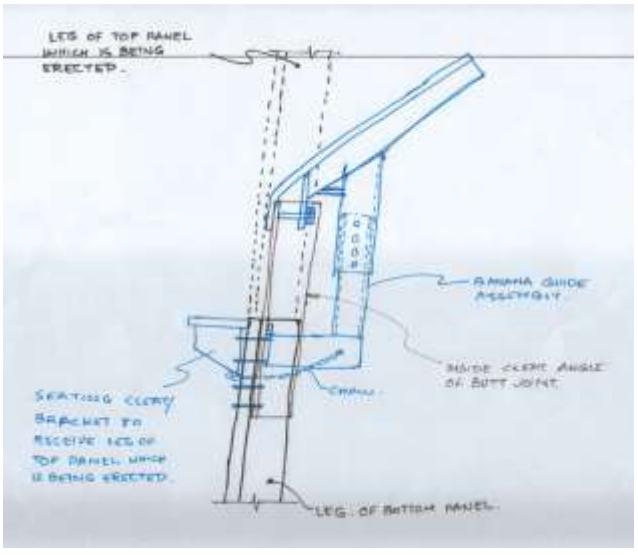




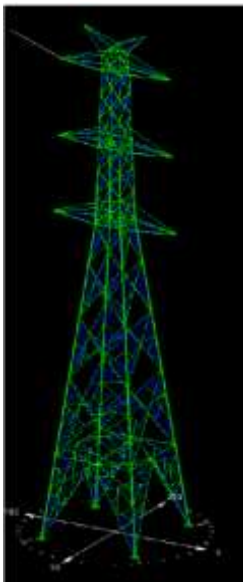
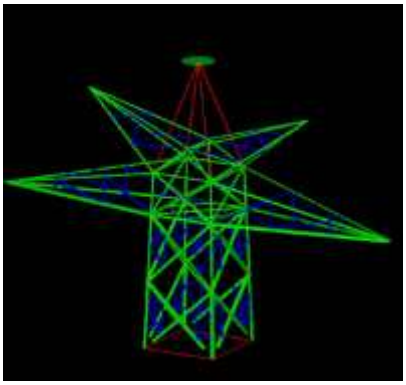
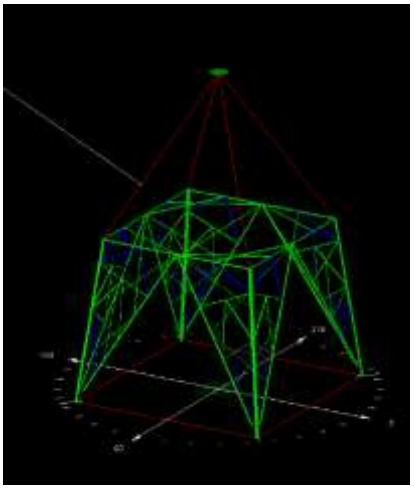
4.1.5 Design of additional joints: Finalization of additional permanent members (if required)



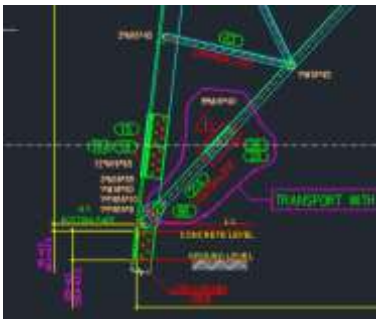
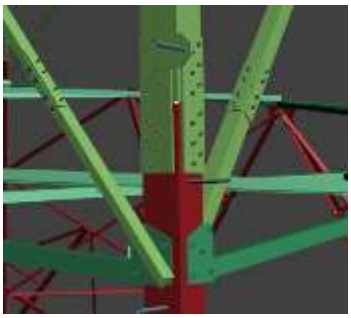
4.1.6 Banana guide assembly: For segment installation by helicopter, temporary guide arrangement is required. Design and drawing of temporary guide arrangement.



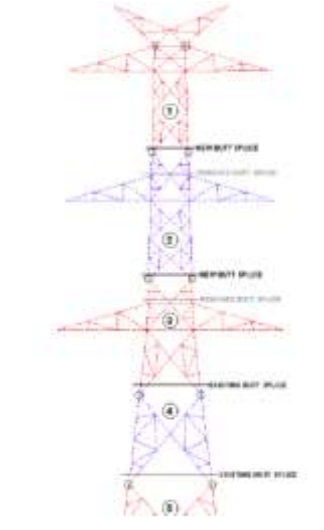
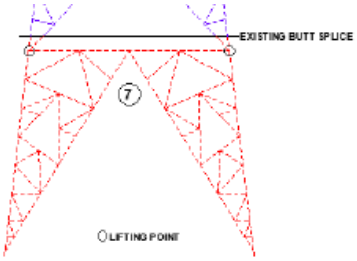
4.1.7 Design of bifurcated panels: Modified PLS Tower model prepared with required additional joint/ permanent (if any) and Temporary members (Lifted Condition) with bifurcated panels, additional temporary members, analysed with lifting condition, checked utilization ratio, design of temporary member, marking on GA etc.

4.1.8 Preparation of drawings & 3D model & BOM: Prepared drawings in AutoCAD/Sketch for required additional joints and strengthening as per above Item Number 5 and 8 (temporary member/ permanent member (if any) work.
BOCAD model modification, Structural drawings, Shop sketches and BoM for additional Joint /Member

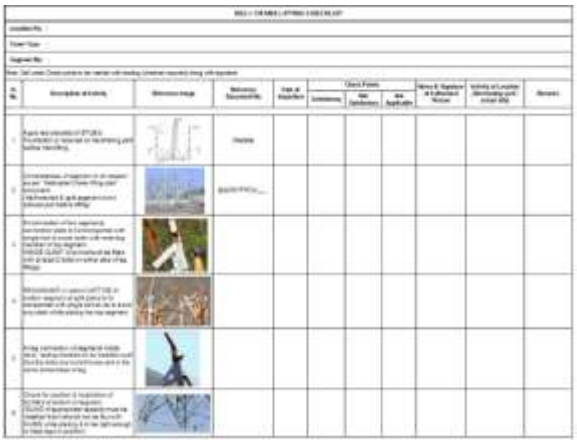



4.1.9 Mark-up of lifting points & temporary member details in drawing

5.0 ***Training & Checklist***:

5.1.1 Prepare check list and Training for Heli-Lifting to Project Execution Team. Since, the activities at site / testing are complicated and use of new technique, we have performed detailed training and have support client with detailed checklist for erection, safety and quality check points



6.0 ***Crane Lifting and Testing:*** Preparation, Measuring Weight of Segment & Lifting Erection and lifting activities at test bed was inspected by our team and all the parameters of design, safety and quality observed to satisfaction.

Segment preparation

Crane lifting Segment wise and Fixing one above each other

Completion of Crane lifting activity

7.0 MAINTAIN A SAFETY PERIMETER

7.1.1 When working with overhead cranes, objects falling from height are one of the most common threats. A falling load can cause serious injuries, fatalities, or structural damage to the project and surrounding properties.

7.1.2 No ground crew should go within 6M of the mast during erection, which erection teams also learnt during their training.

8.0 CONCLUSION:

8.1.1 Project engineering has been successfully completed as per project requirements and client's satisfaction and mock lifting have been tested by cranes in test bed.

ABOUT AUTHOR

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Non Destructive Testing of Concrete Structures – Purpose, Methods, Advantages and Limitations

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1.0 INTODUCTION

1.1. Reinforced Cement Concrete (RCC) is designed in India as per IS 456. In international practice reference is made to ASCE or BIS standard. It is usual to take cube tests (after 7, 14, 21, 28 days) for design mix of concrete. However, when reinforcement bars are positioned and concrete is poured the structure comes into the existence. It is extremely difficult to test the concrete structure for its designed test once it is cast.

1.2. The cement concrete work can be manual or through mixer machine depending upon the quantum of work. For large quantity of concrete work or due to other constraints, it is now usual to use Ready Mix Concrete (RMC).

1.3. Hand mixed and machine mixed concrete (at site) are subject to variation in quality due to variation in solid ingredients (cement and aggregates) proportion of water cement ratio and quality of water and ingredients. The RMC is expected to be more reliable due to controlled conditions of manufacturer. However, human factor is common in all the methods of mixing.

1.4. The quality and quantity of reinforcement also a matter of concern in the cast RCC.

1.5. For quality of assurance at the construction stage non-destructive testing can reduce the gap between design strength and actual strength.

1.6. Similarly for ascertaining the residual life of concrete structure NDT is helpful.

1.7. The presentation below brings in salient features of NDT.

2.0 IMPORTANCE AND NEED OF NON-DESTRUCTIVE TESTING

2.1. It is often necessary to test concrete structures after the concrete has hardened to determine whether the structure is suitable for its designed use. Ideally such testing should be done without damaging the concrete. The tests available for testing concrete range from the completely non-destructive, where there is no damage to the concrete, through those where the concrete surface is slightly damaged, to partially destructive tests, such as core tests and pullout and pull off tests, where the surface has to be repaired after the test.

2.2. Non-destructive tests of concrete is a method to obtain the compressive strength and other properties of concrete from the existing structures. This test provides results and actual strength and properties of concrete structure. Non-destructive testing can be applied to both old and new structures.

2.3. The standard method of evaluating the quality of concrete in buildings or structures is to test specimens cast simultaneously for compressive, flexural and tensile strengths.

2.4. The main disadvantages are that results are not obtained immediately; that concrete in specimens may differ from that in the actual structure as a result of different curing and compaction conditions; and that strength properties of a concrete specimen depend on its size and shape.

2.5. There is no direct measurement of the strength properties of structural concrete once it becomes part of the building for the simple reason that strength determination involves destructive stresses, several non- destructive methods of assessment have been developed. This is for the fact that certain physical properties of concrete can be related to strength and can be measured by non-destructive methods. Such properties include hardness, resistance to penetration by projectiles, rebound capacity, resonant frequency and ability to transmit ultrasonic pulses & X- and Y-rays and Electrical property of Concrete.

These non-destructive methods may be categorized as penetration tests, rebound hammer tests, pull-out techniques, dynamic tests, radioactive tests, maturity concept etc. It is the purpose of this presentation to describe these methods briefly, outlining their advantages and disadvantages.

3.0 PURPOSE OF NON-DESTRUCTIVE TESTS ON CONCRETE

3.1. A variety of Non Destructive Testing (NDT) methods have been developed or are under development for investigating and evaluating concrete structures. These methods are aimed at estimation of strength and other properties; monitoring and assessing corrosion; measuring crack size and cover; assessing grout quality; detecting defects and identifying relatively more vulnerable areas in concrete structures.

3.2. There could be many parameters such as materials, mix, workmanship and environment, which influence the results of measurements. Moreover, these tests measure some other property of concrete (e.g. hardness) and the results are interpreted to assess a different property of concrete e.g. strength, which is of primary interest.

3.3. Following are the key Purposes of Non-destructive Tests

- Estimating the in-situ compressive strength
- Estimating the uniformity and homogeneity
- Estimating the quality in relation to standard requirement
- Identifying areas of lower integrity in comparison to other parts
- Detection of presence of cracks, voids and other imperfections
- Monitoring changes in the structure of the concrete which may occur with time
- Identification of reinforcement profile and measurement of cover, bar diameter, etc.
- Condition of pre-stressing/reinforcement steel with respect to corrosion
- Chloride, sulphate, alkali contents or degree of carbonation
- Measurement of Elastic Modulus
- Condition of grouting in pre-stressing cable ducts

4.0 TYPICAL SITUATION LEADING TO OPT FOR NDT

4.1. Typical situations where non-destructive testing may be useful are, as follows:

- Quality control of pre-cast units or construction in situ.
- Removing uncertainties about the acceptability of the material supplied owing to apparent non-compliance with specification.
- Confirming or negating doubt concerning the workmanship involved in batching, mixing, placing, compacting or curing of concrete.
- Monitoring of strength development in relation to formwork removal, cessation of curing, Prestressing, load application or similar purpose.
- Location and determination of the extent of cracks, voids, honeycombing and similar defects within a concrete structure
- Determining the concrete uniformity, possibly preliminary to core cutting, load testing or other more expensive or disruptive tests
- Determining the position, quantity or condition of reinforcement
- Increasing the confidence level of a smaller number of destructive tests
- Determining the extent of concrete variability in order to help in the selection of sample locations representative of the quality to be assessed.
- Confirming or locating suspected deterioration of concrete resulting from such factors as overloading, fatigue, external or internal chemical attack or change, fire, explosion, environmental effects.
- Assessing the potential durability of the concrete.
- Monitoring long term changes in concrete properties.
- Providing information for any proposed change of use of a structure for insurance or for change of ownership.
- If the results of NDT are leading to rejection of concrete structure, rehabilitation can be worked out depending upon gravity of deviation from acceptance level.

5.0 BASIC METHODS FOR NDT OF CONCRETE STRUCTURES

5.1. The following methods, with some typical applications, have been used for the NDT of concrete:

- Visual inspection, which is an essential precursor to any intended non-destructive test. Many imperfections, such as external cracks, can already be found with visual inspection, which makes this type of inspection a **simple but powerful procedure**.
  An experienced civil or structural engineer may be able to establish the possible cause(s) of damage to a concrete structure and hence identify which of the various NDT methods can be deployed for any further investigation of the problem.
- Half-cell electrical potential method, used to detect the corrosion potential of reinforcing bars in concrete.
- Schmidt/rebound hammer test, used to evaluate the surface hardness of concrete.
- Carbonation depth measurement test, used to determine whether moisture has reached the depth of the reinforcing bars, which is like to cause corrosion.
- Permeability test, used to measure the flow of water through the concrete.
- Penetration resistance or Windsor probe test, used to measure the surface layer hardness. This test does not provide a precise determination of strength. The probe test is useful in assessing quality and relative strengths of concrete only.
- Cover meter testing, used to measure the distance of steel reinforcing bars beneath the surface of the concrete and also possibly to measure the diameter of the reinforcing bars.
- Radiographic testing, used to detect voids in the concrete and the position of stressing ducts.

- Ultrasonic pulse velocity testing, mainly used to measure the compressive strength of the concrete.
- Sonic methods using an instrumented hammer providing both sonic echo and transmission methods.
- Tomographic modelling, which uses the data from ultrasonic transmission tests in two or more directions to detect voids in concrete.
- Impact echo testing, used to detect voids, delamination and other anomalies in concrete.
- Ground penetrating radar or impulse radar testing, used to detect the position of reinforcing bars or stressing ducts.
- Infrared thermography, used to detect voids, delamination and other anomalies in concrete and also detect water entry points in buildings.

6.0 METHODS OF NON-DESTRUCTIVE TESTING OF CONCRETE

6.1. Following are different methods of NDT on concrete:

- Penetration method
- Rebound hammer method
- Pull out test method
- Ultrasonic pulse velocity method
- Radioactive methods

6.2. Penetration Tests on Concrete

6.2.1. The Windsor probe is generally considered to be the best means of testing penetration. Equipment consists of a powder-actuated gun or driver, hardened alloy probes, loaded cartridges, a depth gauge for measuring penetration of probes and other related equipment.

6.2.2. A probe, diameter 0.25 in. (6.5 mm) and length 3.125 in. (8.0 cm), is driven into the concrete by means of a precision powder charge. Depth of penetration provides an indication of the compressive strength of the concrete.

6.2.3. Although calibration charts are provided by the manufacturer, the instrument should be calibrated for type of concrete and type and size of aggregate used.

6.3. Benefits and Limitations

6.3.1. The probe test produces quite variable results and should not be expected to give accurate values of concrete strength. It has, however, the potential for providing a quick means of checking quality and maturity of in situ concrete.

6.3.2. It also provides a means of assessing strength development with curing. The test is essentially non-destructive, since concrete and structural members can be tested in situ, with only minor patching of holes on exposed faces.

7.0 REBOUND HAMMER METHOD

7.1. The rebound hammer is a surface hardness tester for which an empirical correlation has been established between strength and rebound number.

7.2. The only known instrument to make use of the rebound principle for concrete testing is the Schmidt hammer, which weighs about 4 lb (1.8 kg) and is suitable for both laboratory and field work. It consists of a spring-controlled hammer mass that slides on a plunger within a tubular housing.

7.3. The hammer is forced against the surface of the concrete by the spring and the distance of rebound is measured on a scale. The test surface can be horizontal, vertical or at any angle but the instrument must be calibrated in this position.

7.4. Calibration can be done with cylinders (6 by 12 in., 15 by 30 cm) of the same cement and aggregate as will be used on the job. The cylinders are capped and firmly held in a compression machine.

7.5. Several readings are taken, well distributed and reproducible, the average representing the rebound number for the cylinder. This procedure is repeated with several cylinders, after which compressive strengths are obtained.

7.6. Limitations and Advantages

7.6.1. The Schmidt hammer provides an inexpensive, simple and quick method of obtaining an indication of concrete strength, but accuracy of ±15 to ±20 per cent is possible only for specimens cast cured and tested under conditions for which calibration curves have been established.

7.6.2. The results are affected by factors such as smoothness of surface, size and shape of specimen, moisture condition of the concrete, type of cement and coarse aggregate, and extent of carbonation of surface.

8.0 PULL-OUT TESTS ON CONCRETE

8.1. A pull-out test measures, with a special ram, the force required to pull from the concrete a specially shaped steel rod whose enlarged end has been cast into the concrete to a depth of 3 in. (7.6 cm).

8.2. The concrete is simultaneously in tension and in shear, but the force required to pull the concrete out can be related to its compressive strength.

8.3. The pull-out technique can thus measure quantitatively the in-situ strength of concrete when proper correlations have been made. It has been found, over a wide range of strengths, that pull-out strengths have a coefficient of variation comparable to that of compressive strength. They can also be used for Comparative studies.

8.4. Limitations and Advantages

8.4.1. Although pullout tests do not measure the interior strength of mass concrete, they do give information on the maturity and development of strength of a representative part of it. Such tests have the advantage of measuring quantitatively the strength of concrete in place.

8.4.2. Their main disadvantage is that they have to be planned in advance and pull-out assemblies set into the formwork before the concrete is placed. The pull-out, of course, creates some minor damage.

8.4.3. The test can be non-destructive, however, if a minimum pullout force is applied that stops short of failure but makes certain that a minimum strength has been reached. This is information of distinct value in determining when forms can be removed safely.

9.0 ULTRASONIC PULSE VELOCITY METHOD

9.1. At present the **ultrasonic pulse velocity method** is the only one of this type that shows potential for testing concrete strength in situ. It measures the time of travel of an ultrasonic pulse passing through the concrete. The fundamental design features of all commercially available units are very similar, consisting of a pulse generator and a pulse receiver. Pulses are generated by shock-exciting piezoelectric crystals, with similar crystals used in the receiver. The time taken for the pulse to pass through the concrete is measured by electronic measuring circuits.

9.2. Pulse velocity tests can be carried out on both laboratory-sized specimens and completed concrete structures, but some factors affect measurement:

- There must be smooth contact with the surface under test; a coupling medium such as a thin film of oil is mandatory.
- It is desirable for path-lengths to be at least 12 in. (30 cm) in order to avoid any errors introduced by heterogeneity.
- It must be recognized that there is an increase in pulse velocity at below-freezing temperature owing to freezing of water; from 5 to 30°C (41 - 86°F) pulse velocities are not temperature dependent.

- The presence of reinforcing steel in concrete has an appreciable effect on pulse velocity. It is therefore desirable and often mandatory to choose pulse paths that avoid the influence of reinforcing steel or to make corrections if steel is in the pulse path.

9.3. Applications and Limitations

9.3.1. The **pulse velocity method** is an ideal tool for establishing whether concrete is uniform. It can be used on both existing structures and those under construction. Usually, if large differences in pulse velocity are found within a structure for no apparent reason, there is strong reason to presume that defective or deteriorated concrete is present.

9.3.2. High pulse velocity readings are generally indicative of good quality concrete. A general relation between concrete quality and pulse velocity is given in Table.

10. Table: Quality of Concrete and Pulse Velocity

General Conditions	Pulse Velocity ft/sec
Excellent	Above 15,000
Good	12,000-15,000
Questionable	10,000-12,000
Poor	7,000-10,000
Very Poor	below 7,000

9.3.3 Fairly good correlation can be obtained between cube compressive strength and pulse velocity. These relations enable the strength of structural concrete to be predicted within ±20 per cent, provided the types of aggregate and mix proportions are constant.

9.3.4 The pulse velocity method has been used to study the effects on concrete of freeze-thaw action, sulphate attack, and acidic waters. Generally, the degree of damage is related to a reduction in pulse velocity. Cracks can also be detected.

9.3.5 Great care should be exercised, however, in using pulse velocity measurements for these purposes since it is often difficult to interpret results. Sometimes the pulse does not travel through the damaged portion of the concrete.

9.3.6 The pulse velocity method can also be used to estimate the rate of hardening and strength development of concrete in the early stages to determine when to remove formwork. Holes have to be cut in the formwork so that transducers can be in direct contact with the concrete surface.

9.3.7 As concrete ages, the rate of increase of pulse velocity slows down much more rapidly than the rate of development of strength, so that beyond a strength of 2,000 to 3,000 psi (13.6 to 20.4 MPa) accuracy in determining strength is less than ±20%.

9.3.8 Accuracy depends on careful calibration and use of the same concrete mix proportions and aggregate in the test samples used for calibration as in the structure.

9.3.9 In summary, ultrasonic pulse velocity tests have a great potential for concrete control, particularly for establishing uniformity and detecting cracks or defects. Its use for predicting strength is much more limited, owing to the large number of variables affecting the relation between strength and pulse velocity.

10.0 RADIOACTIVE METHODS OF NDT

10.1 Radioactive methods of testing concrete can be used to detect the location of reinforcement, measure density, thickness and perhaps establish whether honeycombing has occurred in

structural concrete units. Gamma radiography is increasingly accepted in England and Europe. The equipment is quite simple and running costs are small, although the initial price can be high. Concrete up to 18 in. (45 cm) thick can be examined without difficulty.

11.0 Equipment for Non Destructive Testing

11.1 According to their use, non-destructive equipment can be grouped as under:

- Strength estimation of concrete
- Corrosion assessment and monitoring
- Detecting defects in concrete structure
- Laboratory tests

12.0 CONCLUSION

12.1 NDT is a very important tool for quality control, quality assurance and maintenance in the various structures i.e. Resident buildings, Industrial buildings, Oil and gas industry, power sector etc. as it can help prevent failures that could have an adverse impact on safety, reliability, and the environment. Aerospace industry and medical equipment manufacturers need a consistent NDT service to ensure the correct usage of equipment and safety measurements.

12.2 NDT has several advantages, including the preservation of the original material it tests. Engineers can accomplish this through the use of non-destructive methods to analyze materials, which enables them to make necessary adjustments without performing additional work.

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INCREASING ROLE OF CIVIL ENGINEERING IN MINING INDUSTRY

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ABSTRACT-- *Mining in India meets the mineral demands of various industries. It employs about six million people and also generates revenues of 2.3% of India's gross domestic product. Civil Engineering is an amalgamation of science, art, professional skills and engineering achievements. The requirement of the appropriate construction material of required specification plays an important role to achieve strength and construction economy of structure. Civil engineering works are carried out on or inside the ground surface. Civil Engineers are required to ensure that mines operate safely, efficiently and profitably. They play an important role in a mine's engineering department, helping to establish budgets and monitor project progress, as well as designing mine pit and installing new infrastructures.*

1. INTRODUCTION

India is now concentrating on harnessing its energy and on development of infrastructure. This would require large scale demand for construction material, hydroelectric power generation, large scale mining, tunneling, underground metros, large scale excavations and slope stabilizations.

As the geomorphological and geological processes operate relentlessly and subject an area to continual change, therefore geomorphic setup and geology of the site are vital for the success of all sorts of civil engineering works in general and for structures made in subsurface conditions in particular. Earthquake Engineering, an independent and very advance civil engineering subject too need sound knowledge of tectonics and structural geology for identifying the earthquake prone areas. Coastal and Marine Engineering, Environmental engineering etc. are disciplines which also need interface of civil engineering and engineering geology. The Natural Disaster studies also require knowledge of earth science in general and of civil engineering in particular for working out ways of mitigating them.

2. INVOLVEMENT OF CIVIL ENGINEER IN MINING INDUSTRY

Civil engineers are most frequently hired by a mining construction company, are placed in an office, and do not spend much time on construction sites. To design and price new works, they rely on extensive construction experience or estimations based on analysis of previous projects.

At mining sites, civil engineers could undertake a variety of tasks. Some of the responsibilities include assisting in the planning & designing of mine, carrying out scientific studies for pit slope stabilities, overburden dumps and supervision of the various construction work and structures related to Mines, also study blueprints, maps, survey reports, and other topological or geologic data with their skills. Moreover, civil engineers are also in charge of overseeing constructions and surveying works.

In addition, creating technical reports or providing advice, as well as inspecting project sites and testing materials, may be part of the roles of civil engineers in the mining industry.

The role of civil engineer offers you opportunities all over the world, in mines harvesting a range of minerals. It can be a very satisfying career suited to many people.

3. QUALIFICATIONS REQUIRED

Usually, you will only need a Bachelor’s degree to become a civil engineer in the mining industry. There’s no need to pursue any further qualifications, although they can help you to secure a position. Some mining companies will require that you have relevant work experience before you can take up a position. There are several related areas of study that could help aspirant access career. These subjects include structural, hydraulic and transportation engineering.

You might also find that geo-mechanics, hydrology, construction and economics can help you find a job in this area.

4. JOB OPPORTUNITIES

Choosing to go into civil engineering in mining could open up employment opportunities for you around the world. If you would find companies supplying a number of large and small mining outfits. The mining industry is an excellent choice for any civil engineer who wants to do something different, be paid well and have the chance to take on a senior role.

5. WORKS HANDLED:

The activities of Civil Engineering discipline are of diverse nature and are broadly categorized below:

5.1 ENABLING WORKS:

5.1.1 Temporary approach road

- Roads (Haul Roads) for mineral and overburden mass movements.

5.1.2 Temporary hutments for office/ camp living

- Construction & O&M for accommodations for officers/workers/others.

5.1.3 Temporary water supply arrangements

5.1.4 Erection platforms for HEMM

- Workshops, Fabrication & Maintenance shops of Heavy Equipment Mining Machineries.

5.1.5 Temporary stores & workshops etc.

5.2 INFRASTRUCTURES:

5.2.1 Office buildings and rest house.

- Construction & Maintenance of Administrative Buildings such as Office, Rest Shelter for Workers, First Aid Rooms, Occupational Health Center, Vocational Health Center, Canteen, Bathing and Sanitation facilities for employees, etc.

5.2.2 Pit head bath & cap lamp room

- Construction of Storage facilities for miners' equipment.

5.2.3 Incline mouth roofing & walling

- Construction of tunnels for incline mine passage and steel/concrete roofing.

5.2.4 Headgears & shaft sinking

- Structural design and construction of shaft (Vertical tunnel) trusses.

5.2.5 Haulage rooms & fan houses

- Painting and Minor Repairing Work of Haulage Room (i.e. mineral transportation haulage in underground mine) and Fan House (i.e Mine ventilation structures) etc.

5.2.6 Haul roads & permanent roads

- Quantity estimate for road constructions and Estimation of Gradient.

5.2.7 Regional/ Central stores

5.2.8 Regional/ Central workshops

5.2.9 Railway sidings & wharf walls

- Construction of Railway stations, laying of railway tracks, wagon loading centers, etc. for transportation of mineral via railway.

5.3 HEAVY INDUSTRIAL STRUCTURES

5.3.1 Coal handling plants

- Such as Conveyor Corridor, Crusher Units, mineral beneficiation plants, etc.

5.3.2 Winder house Check dams Bridges

- Soil sample test for construction of foundations and others.

5.3.3 Underground transport system

- Haul Roads, Pillars, etc.

5.4 TOWNSHIPS:

5.4.1 Residential buildings

5.4.2 Colony & approach roads

5.4.3 Water supply arrangements

- Construction & Maintenance of residential accommodation facilities, development of town-ship for officers / workers / others and roofing and walling of ancillary structures

5.4.4 Sewerage/ drainage systems

5.4.5 Plantations & landscaping

- Development of Green belt around the mine boundary and other infrastructures.

5.5 WATER SUPPLY:

3.1.2 Intake arrangements (wells, weirs)

3.1.3 Treatment plants (ETP/STPs)

- Construction of Water treatment plants and laying of water Pipe Line.

5.5.3 Treated water storage & distribution

5.5.4 Overhead reservoirs

- Construction of Overhead Tanks, etc.

5.6 WELFARE BUILDINGS:

5.6.1 Construction and maintenance of welfare buildings, such as

- Institute Schools & colleges
- Shopping centers
- Post office & Banks
- Hospitals & Dispensaries
- Recreation Centers & Clubs
- Canteens
- Auditoriums & Stadiums

6. APPLICATION OF CIVIL ENGINEERING IN DEALING WITH SLOPE STABILITY ISSUES IN OPENCAST MINE

Now a day slope stability accidents are one of the leading causes in surface Mining operations. Unexpected movement of ground causes the potential to endanger lives, demolish equipment, or destroy property. The economic consequences and operational problems associated with slope instabilities dictate the need of appropriate slope monitoring and management measures. Slope management basically constitutes anticipation, pre detecting of likely changes and control of slope behaviour. Needless to emphasize the requirement of mass exploitation of mineral deposits to meet the local and global demands in the present era of globalization, and also the importance of scientific opencast mining to greater depths. Therefore, the recent advances in design guidelines and slope monitoring are important for improved safety and eco-friendly mining for sustainable development of mineral deposits of India, and also for other civil engineering applications including embankments, Dams, road ways.

The civil engineering applications include:

- Slope design for mine benches, Embankments, railways, road and civil construction.
- Application of soft computing techniques for prediction of slope stability.
- Ground water issues in slope stability.
- Financial aspects of slope stability.
- Slope failures, and demonstration of numerical models.
- Mine Waste Disposal, Environment and Safety.
- Design, Stabilization and Monitoring of Surface Mine Slope.

The process of design consider observations on rock behaviour and fracture mechanics, monitoring of rock movements and stresses together with an assessment of the simplified mechanisms of failure to obtain an understanding of ground behaviour. The role of water regime in cohesive soils, while the water regime as a whole has a subdued influence on rock slope stability. Computer methods assess the interaction of ground interacting rock failure mechanisms in complex geological conditions. Based on the same a detailed study of different strength parameters of slope forming materials and analysis of documented slope failures, generalised slope design guidelines are evolved.

7. PHOTOGRAPHS OF MINE WORKING

8. CONCULSION AND FUTUREWORK

The Civil Engineering Department as stated in the beginning, is entrusted with the responsibility of providing leadership for various Welfare and Mine development activities including Service Buildings, Haul Roads, Coal handling Plants etc. In addition, the department is in-charge of the maintenance of the assets of the company already existing as Roads, Buildings, Plants, etc. The civil engineering applications provides ambitious welfare plans in the form of housing, water supply, community developments etc. and these are executed and maintained by the Civil Engineering Department of the companies. The Civil Engineers at various tiers of the department should therefore know his duties well and discharge them diligently. In discharge of his duties the Engineers at every level shall keep in mind the following guidelines in general.

Civil engineers will continue to find opportunities in opencast as well as in underground mining engineering projects and developments from civil engineering construction works will continue to find application in underground and opencast mining. The two industries are inter-related and we must continue to work together.

About Authors

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c. Published Paper in International Research Journal on “Ground Improvement Techniques: Lime Stabilization”. ISSN no.2320-723x, Vol-1, Issue-3, Chennai. (April, 2014)
d. Published Paper in ICLTCSEIE International Conference Proceeding. [March, 2013].

Rectification of Foundation in Switchyard before Commissioning – Case Study.

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1.0 Introduction:

1.1 In Madhya Pradesh a wind power developer planned to establish a 33/220kV pooling sub-station. The civil work including gantry column foundation was completed & equipment were also erected. The sub-station was scheduled to be commissioned. However due to unseasonal rain, the foundation of equipment support structure settled or tilted. This occurrence also resulted into damage to the 33kV & 220kV sub-station equipment.

1.2 Since the soil around the foundations (which failed) was disturbed to a greater extent, it was not possible to remount structures & equipment just by bringing back the foundations in their own position. Further, it was also not possible to re-align the foundations as it would mean a change in layout and elevation.

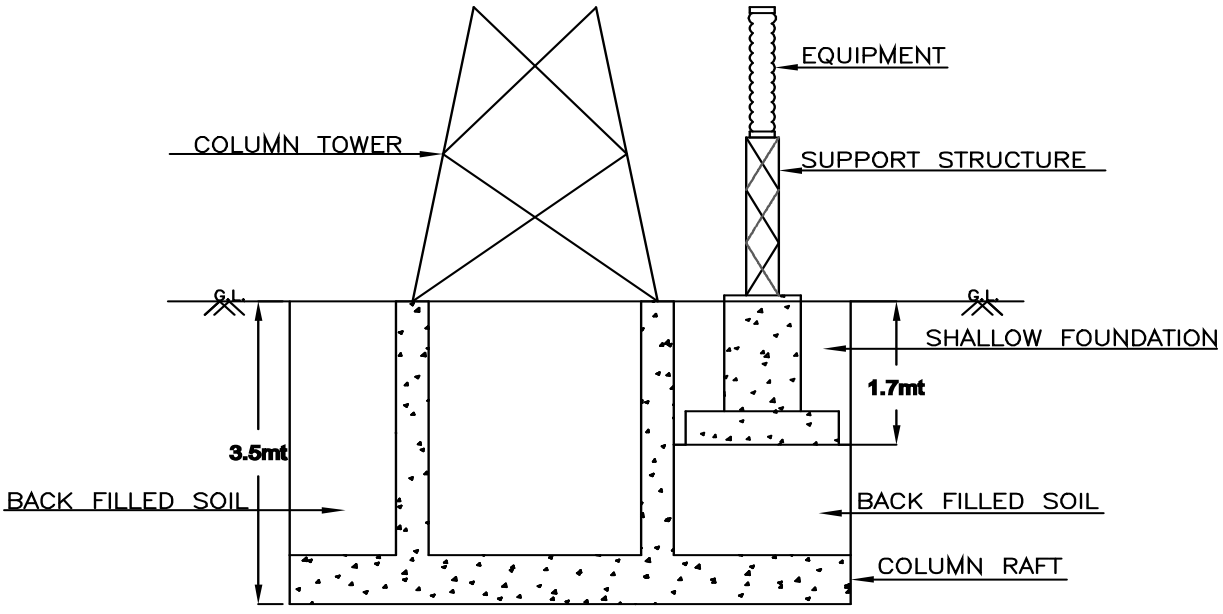
1.3 The matter was referred to the company of the authors with a request to immediately move to the site. The Authors' company accepted the request & the authors moved to the site immediately.

2.0 Observations:

2.1 During the site visit it was observed that many equipment support foundations were either tilted or settled. The equipment support structure along with the equipment (CT / PT / Isolator etc.) also settled or tilted along with the foundations.

2.2 Many equipment in 220kV & 33kV switchyard also got severely damaged due to hitting to the other structure or ground.

2.3 It was observed that the foundations which settled or tilted were cast as per the layout plan & were falling in the excavation line of raft foundations of gantry column (tower) which was 3.5M deep as shown below. The equipment support foundation was 1.7 to 1.8M deep.



2.4 Besides, the backfilling of the column foundation was not proper. Therefore, the equipment support structure foundations were found to be settled during abrupt rain.

2.5 The plan layout of 220kV & 33kV showing the damaged/affected foundations is attached here with figure 1.

Image 1: Photographs of tilted/settled foundations

Image 2: Photographs of tilted/settled foundations

Image 2: Photographs of damaged equipment.

3.0 METHODOLOGY FOR RECTIFICATION OF DAMAGED FOUNDATIONS

3.1 Depending upon the severity of the damage the foundations to be rectified were divided into 3 cases as follows.

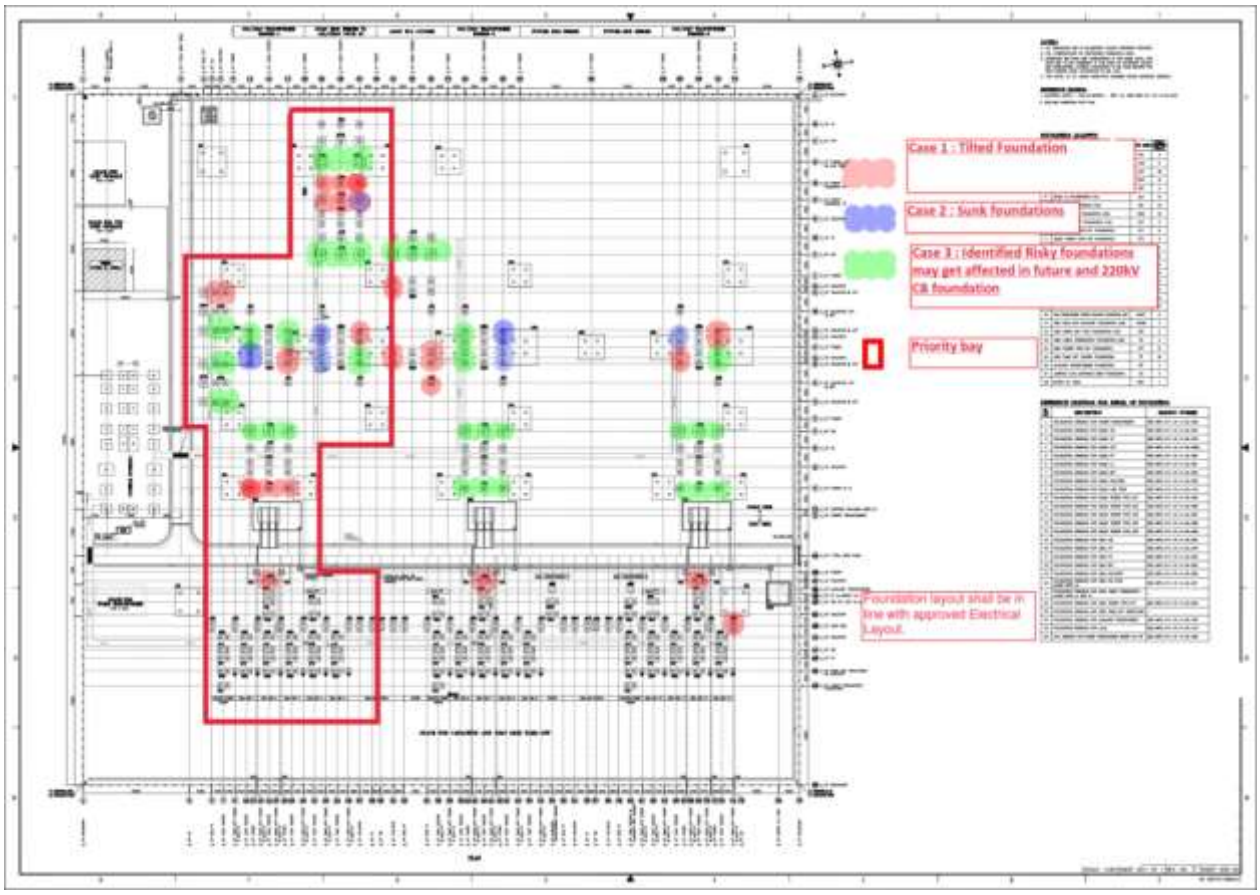


Figure 14: Foundation Layout

3.1.1 Tilted Due to Case 1: The Foundations Which Have Saturation of Soil

In this case the foundations are tilted in one direction or two directions and simultaneously found to have sunk. Mostly such foundations are for individual phase equipment such as CT, PT, LA, PI etc. The reasons for such an occurrence are indicated above. Such foundations are disturbed due to one side stringing of jumpers and saturation of soil. The following steps were recommended for rectification of such foundations.

- Removal of all jumpers from the top of the equipment, Removal of Equipment, and its support structure. Now only foundation column and anchor bolts will be left as they are

- Reference to the correct drawing is essential before taking up rectification work.
- The soil around the columns and footings shall be removed in such a manner that the entire foundation is free to move in any direction.
- Bring the foundation back to its original position by mechanical force. Use of winch is recommended.
- Lean concrete (M-7.5) to be filled below footing wherever void is found below the footing and proper compaction is difficult. The alignment of the foundation shall be critically checked with reference to the Northing-Easting or foundation footprint layout.
- After the foundation is aligned, backfill the foundation by mixing crusher dust (Quarry dust) with the excavated soil in the ratio of 1:1. The backfilling should be done gradually with the layers of 250mm (Maximum) followed by watering and compaction. Initially backfilling shall be done up to the level where the proposed tie beam is supposed to rest.
- Auguring shall be done (manual/mechanical auguring) for casting of a micro pile as shown in the relevant drawing attached herewith. If stiff layers of soil, rock or concrete are encountered, auguring will be stopped & piles will be made to rest on this hard surface.
- The pile should be cast in-situ with reinforcement protruding above the pile level. The concrete should be allowed to settle for 24 hours.
- The soil at the level where the tie beam is expected to rest shall be properly dressed and well compacted. A 50mm PCC layer shall be provided above the compacted soil as a base and leveling course. As per the attached detailed drawings of equipment foundations, the anchor bars shall be provided in the column by drilling holes and chemical anchoring using HILTI RE 500. The drilled holes shall be up to the length indicated in the attached drawing. This is required for proper connection of foundation column with the beams and thereby allowing transfer of compressive, tensile and shear loads from column to the beam and beam to the micro pile. Shuttering shall be provided on two sides as per the attached drawing of beam. The protruding reinforcement of the pile and the part of the anchor bolt from foundation column, should be connected to the reinforcement bars of beam as per standard practice. Similarly Concrete mix of the same ratio (i.e., main foundation) shall be poured in the tie beams and piles.
- Backfilling shall be done next day very carefully after removal of shuttering of the beam to avoid any damage to the beam and pile with proper compaction and watering. It is preferable to cure the beam and repair column top for 7 days from the ground level.
- The finished ground level (FGL) shall be attained by proper dressing of soil and spread of metal/gravel.

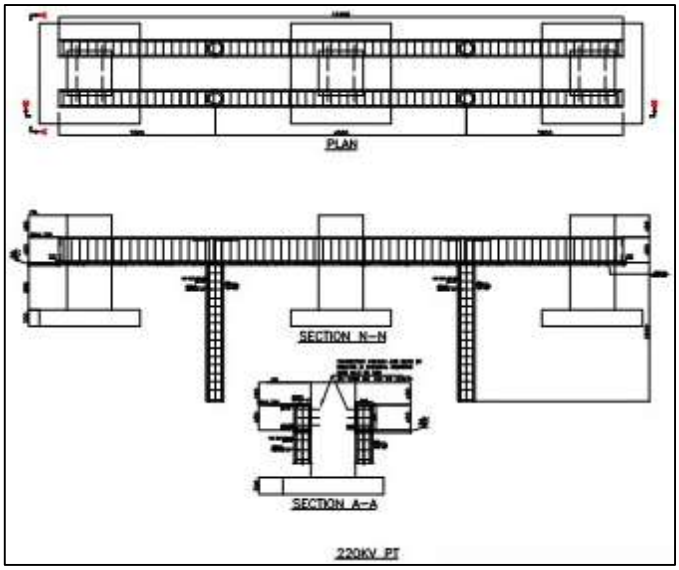


Fig:-2 220kV PT

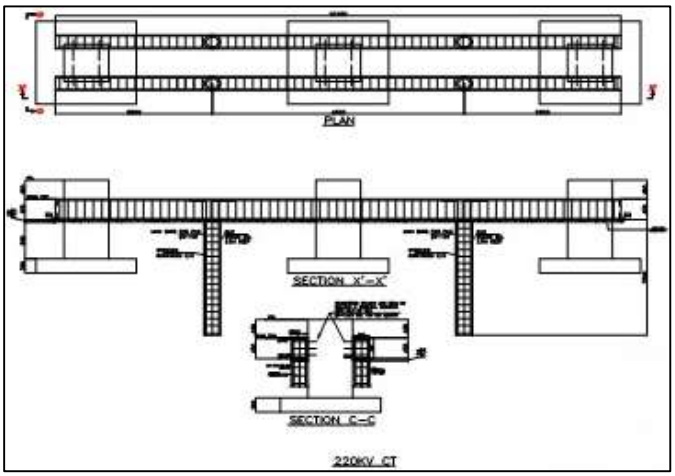


Fig:-3 220kV CT

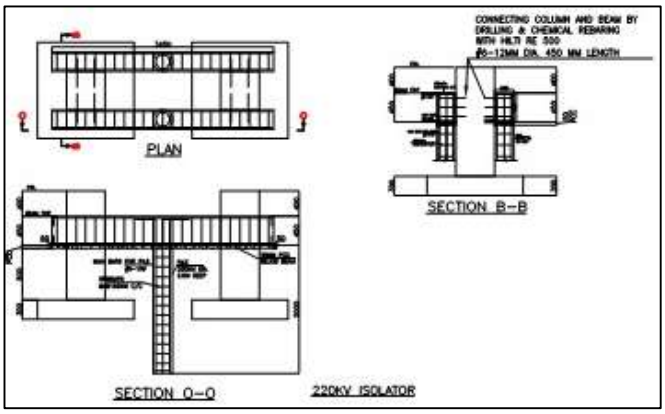


Fig:-4 220kV Isolator

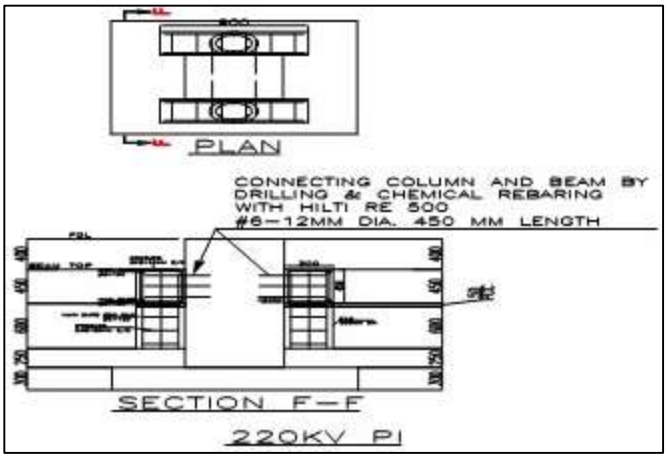


Fig:-5 220kV PI

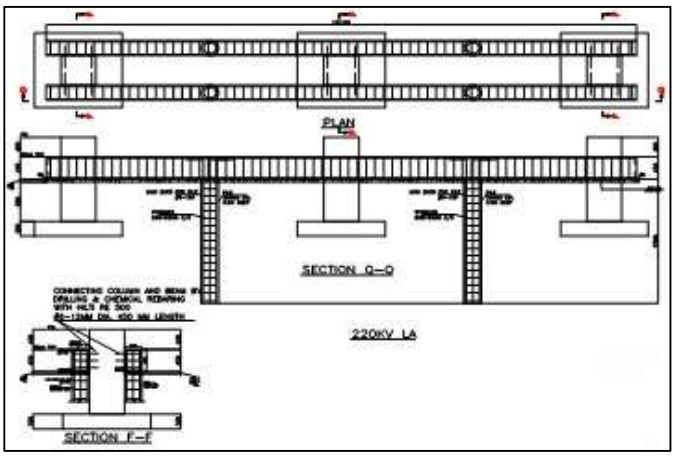


Fig:-6 220kV LA

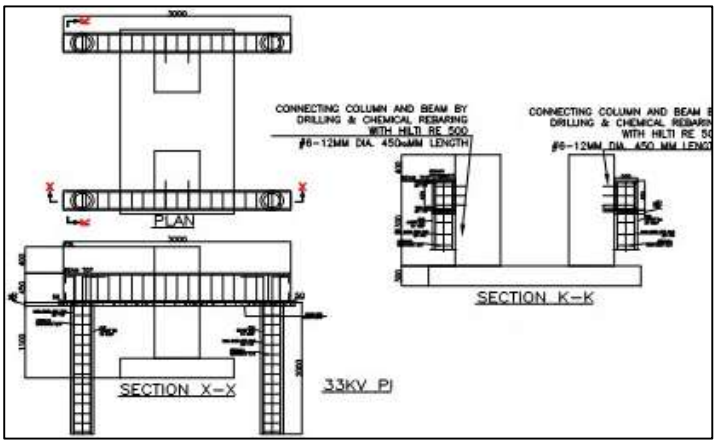


Fig:-7 33kV PI

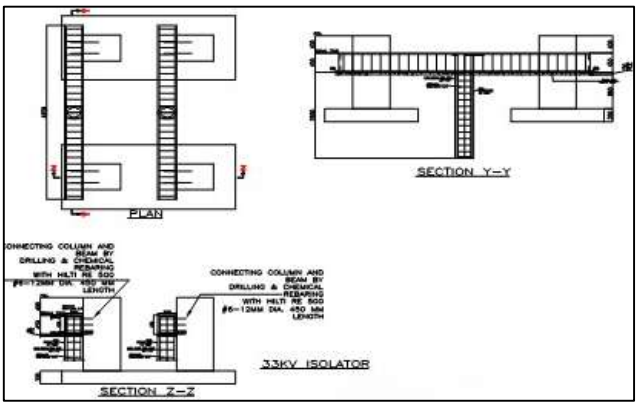


Fig:-8 33kV Isolator

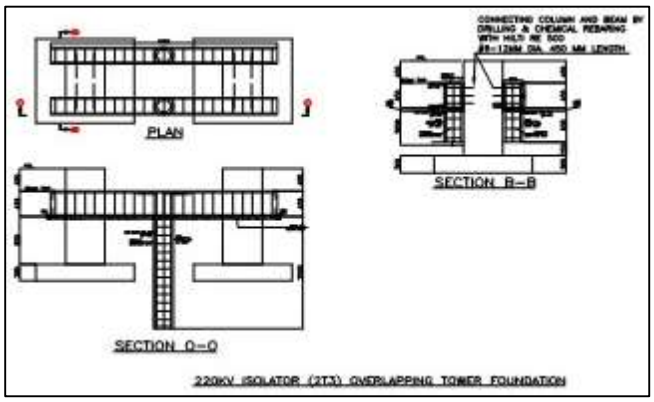


Fig:-9 220kV Isolator

3.1.2 Case 2: Some Foundations Which Have Sunk & Tilted Due to Soil Saturation

3.1.2.1 Some foundations are disturbed due to the stringing of jumpers & line/bus-bar conductor as well as saturation of soil at the footing level. The following steps may be followed for rectification of such foundations.

- Removal of all jumpers from the top of the equipment, Removal of Equipment, and its support structure. Now only foundation column and anchor bolts will be left as they are.
- Reference to the correct drawing is essential before taking up rectification work.
- The soil around the columns and footings shall be removed in such a manner that the entire foundation is free to move in any direction.
- Lean concrete (M-7.5) to be filled below footing wherever void is found below the footing and proper compaction is difficult. The alignment of the foundation shall be critically checked with reference to the Northing-Easting or foundation footprint layout.
- After the foundation is aligned, backfill the foundation by mixing crusher dust (Quarry dust) with the excavated soil **in the ratio of 1:1.** The backfilling should be done gradually with the layers of 250mm (Maximum) followed by watering and compaction. Initially backfilling shall be done up to the level where the proposed tie beam is supposed to rest.
- Auguring shall be done (manual/mechanical auguring) for casting of a micro pile as shown in the relevant drawing attached herewith. If stiff layers of soil, rock or concrete are encountered, auguring will be stopped & piles will be made to rest on this hard surface.

- The pile should be cast in-situ with reinforcement protruding above the pile level. The concrete should be allowed to settle for 24 hours.
- The soil at the level where the tie beam is expected to rest shall be properly dressed and well compacted. A 50mm PCC layer shall be provided above the compacted soil as a base and leveling course. As per the attached detailed drawings of equipment foundations, the anchor bars shall be provided in the column by drilling holes and chemical anchoring using HILTI RE 500. The drilled holes shall be up to the length indicated in the attached drawing. This is required for proper connection of foundation column with the beams and thereby allowing transfer of compressive, tensile and shear loads from column to the beam and beam to the micro pile. Shuttering shall be provided on two sides as per the attached drawing of beam. The protruding reinforcement of the pile and the part of the anchor bolt from foundation column, should be connected to the reinforcement bars of beam as per standard practice. Similarly Concrete mix of the same ratio (i.e., main foundation) shall be poured in the tie beams and piles.
- The finished ground level (FGL) shall be attained by proper dressing of soil and spread of metal/gravel.

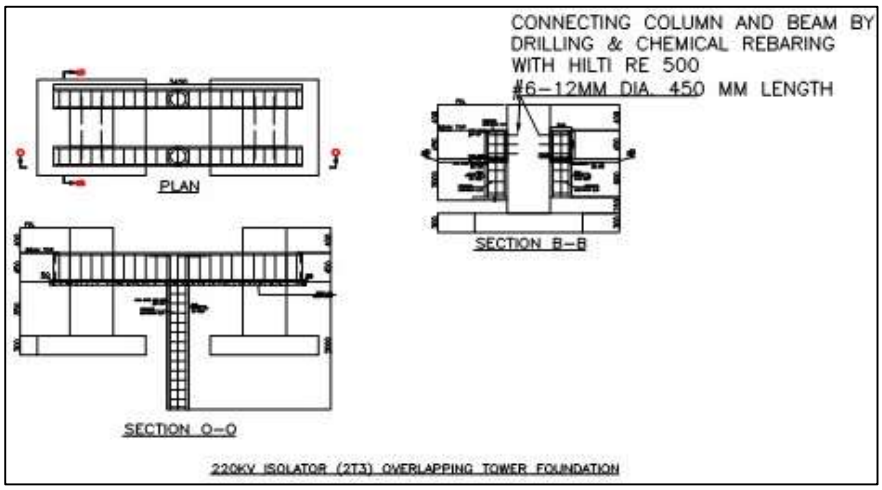


Fig:-10
220kV Isolator

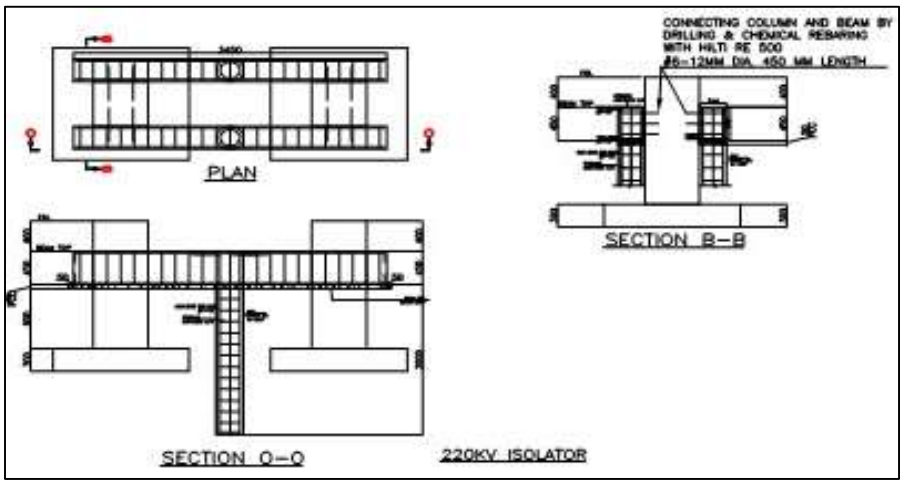


Fig:-11
220kV Isolator

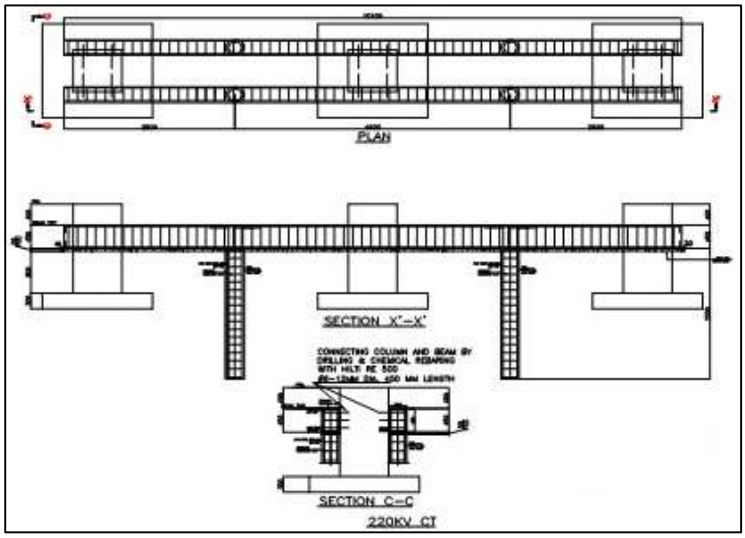


Fig.:-12 220kV CT

3.1.3 Case 3: Foundations which are likely to get affected in future.

3.1.3.1 Considering the conditions mentioned in section 2.0 above, some foundations (enclosed foundation layout fig (1) are identified as a likely to get affected in future & following precautions:

- The alignment of the foundation shall be critically checked with reference to the Northing-Easting or foundation footprint layout. Similarly, the top of the foundation bolt shall be checked with reference to the finished ground level (FGL).
- If the soil around such foundations is not found to be settled and if no settlement or tilting is observed, it may not be necessary to remove the equipment or the structure. However, for the purpose of safety stay wires or stay ropes shall be provided at the bottom of the equipment (or on the top of support structure). The stays shall be provided making an angle of 45° with vertical (in elevation).
- Reference to the correct drawing is essential before taking up rectification work.
- The excavation shall be done for laying RCC beams as per the relevant drawing.
- Auguring shall be done for in-situ casting as indicated in the attached drawing. Manual auguring shall be preferred. While auguring for micro pile if any void is encountered, the augur shall be removed, and soil shall be filled in the void. Similarly, if any stiff layer of soil, rock or concrete is encountered, auguring will be stopped, and piles will be made to rest on this hard surface. The protruding reinforcement of micro piles shall be connected to the reinforcement of the tie beam. Lean concrete (M-7.5) shall be laid below the tie beam.
- Similarly Concrete mix of the same ratio (i.e., main foundation) shall be poured in the tie beams and piles. After the micro piles and beams are cast and the beams are properly connected to equipment support foundation column by chemical anchoring using HILTI RE 500, backfill the foundation by mixing crusher dust (Quarry dust) with the excavated soil in the ratio of 1:1. The backfilling should be done gradually with the layers of 250mm (Maximum) followed by watering and compaction.
- The finished ground level (FGL) shall be attained by proper dressing of soil and spread of metal/gravel.

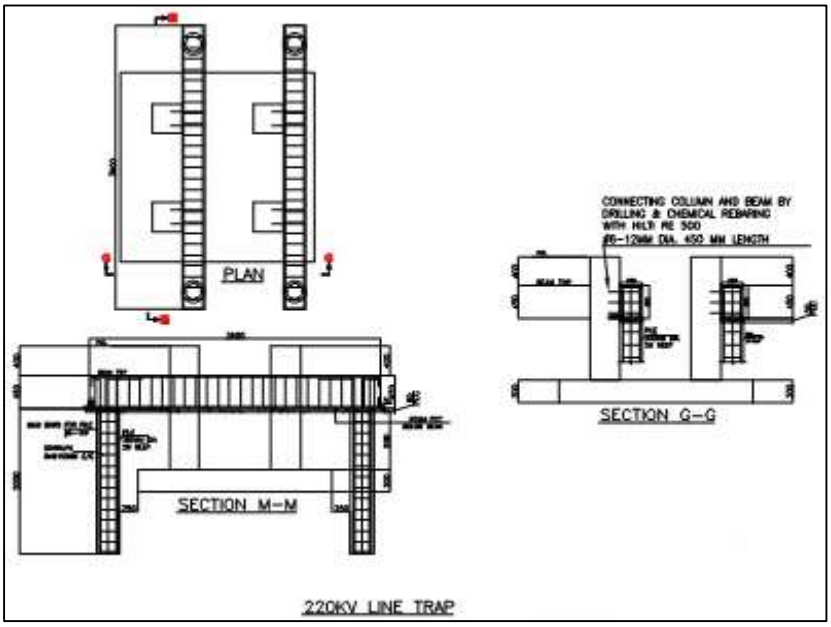


Fig.:- 13 220kV Line Trap

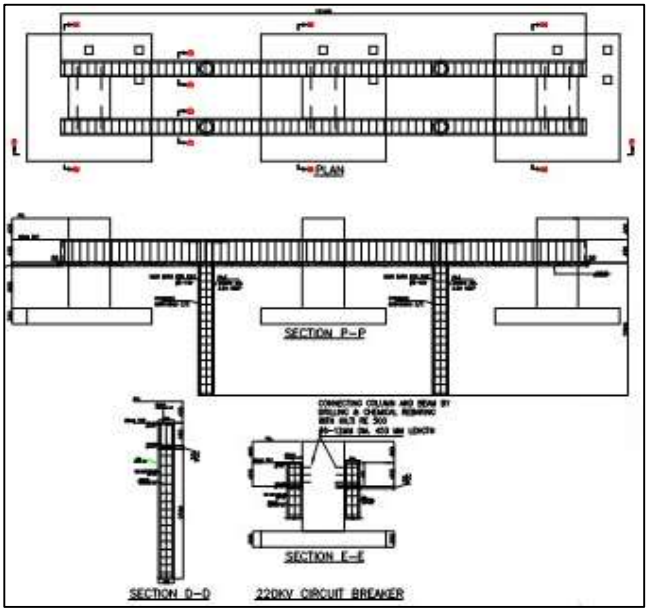


Fig.:- 14 220kV CB

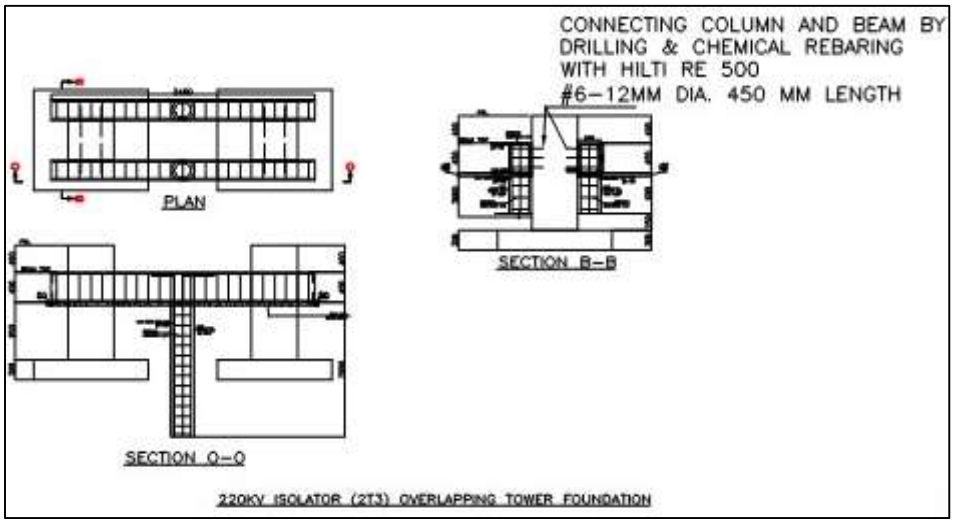


Fig.:- 15 220kV Isolator

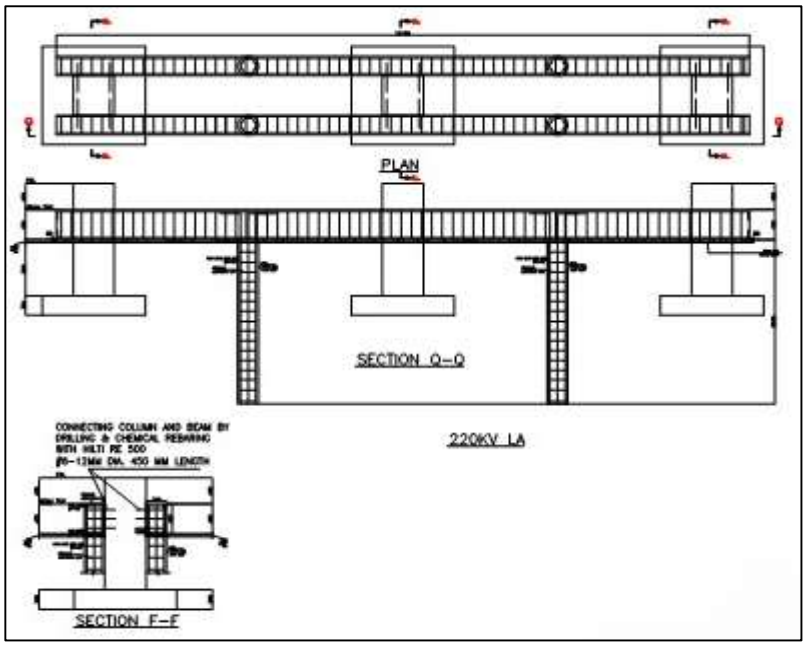


Fig.:- 16 220kV LA

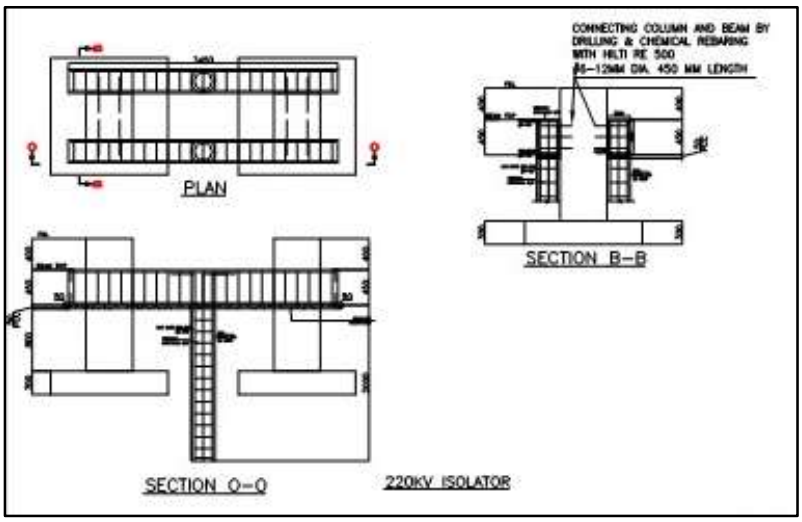


Fig.:- 17 220kV Isolator

4.0 Recommendation:

4.1 The compaction of soil and backfilling by the mixture of crusher dust (Quarry dust) and the excavated soil is very important and must be ensured by a joint inspection by senior officers

4.2 The structure and equipment should be re-erected on the rectified foundations only after 3 days after completion of rectification work.

4.3 While re-stringing, it is advisable to provide stays/anchors to ensure that the foundation do not deform again. After stringing is over the stays/anchors should be gradually removed. As a matter of fact, the stays/anchors should be kept a little loose and observe them while stringing work is under progress. If any one of the stays/anchors starts getting tightened, the stringing work should be stopped, and corrective measures should be taken.

5.0 Conclusion

5.1 If the sub-station structure foundation (open cast) are damaged, they can be repaired, rectified, or retrofitted. This is required to be done as the space available in sub-station is fixed & plan layout is also fixed & cannot be changed.

5.2 Micro piling & anchoring is the best option for securing the damaged open cast foundation of the substation/switchyard.

About Author

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Born on 30 Mar 1948 and graduated in Electrical engineering from the MS University of Vadodara in 1971. He has more than forty years' experience in transmission and distribution of power as well as hydro power. He retired from Gujarat Energy Transmission Corporation in the cadre of Chief Engineer in March 2006. Presently he is a **Managing Director** of Consultancy Firm named "Takalkar Power Engineers & Consultants Pvt. Ltd." in Vadodara. The firm is engaged in design, engineering and construction supervision of transmission lines and substation up to 765kV. The firm is also actively involved in industrial electrical design and hydro power designs.

Kuldeep Barge

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Application of Ground Improvement and Settlement Control for Power Plant Foundation System

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1.0 Introduction:

1.1 Geotechnical engineering plays a key role in all civil engineering projects since all facilities are built on suitable foundation system on soil/rock or within the sub-surface. Geotechnical engineering is the systematic application of techniques which allows construction on geomaterials, i.e. soil and rock. Despite notable progress in geotechnical engineering, many solutions are still judgemental, which is mainly due to the natural inherent inhomogeneity of soils, unpredictable behaviour under various environmental conditions. Additionally, soils are more sensitive to local environmental conditions compared to other prefabricated building materials such as steel or concrete.

1.2 To determine soil properties and ground profile, detailed geotechnical investigation along with geophysical testing like Multichannel Analysis of Surface Wave (MASW), Seismic Refraction Test (SRT), etc. are employed.

1.3 The analysis of ground behaviour, assessment of ground movements, slope stabilization, ground condition improvement, deep excavations, excavation of tunnels / other underground openings and the design of foundations etc. involves application of geomechanics (rock mechanics and soil mechanics) to predict the future behaviours in respect of soil-structure interaction and to assess the serviceability requirements for structural safety.

Fig. 1 depicts different applications of geotechnical engineering.

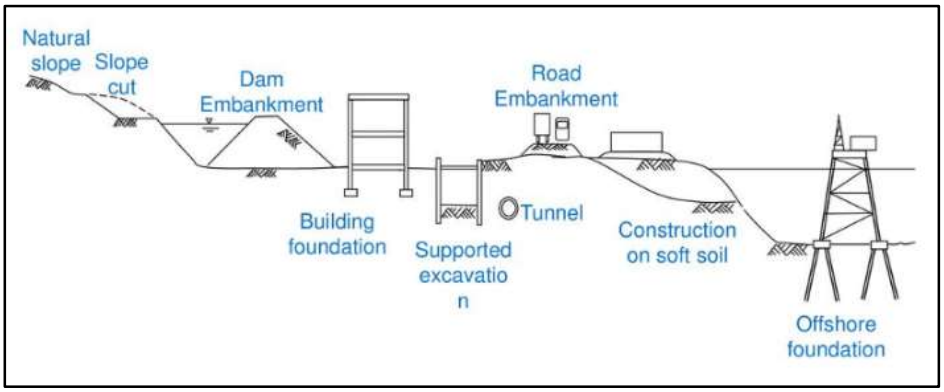


Fig. 1. Different Application of Geotechnical Engineering

1.4 Selection criteria for foundation majorly depends on two factors, i.e. factors related to ground (soil) conditions and factors related to loads from the structure. Economic feasibility is also another aspect for selection of foundation system. The design and choice of the foundation system is only as good as its execution in the field, hence the choice of foundation should be made keeping in mind the ease with which it can be executed/constructed at the site.

1.5 Fig. 2 explains the steps involved in selection of foundation system depending upon the soil type, load requirement, space & resource availability, time duration for project completion.

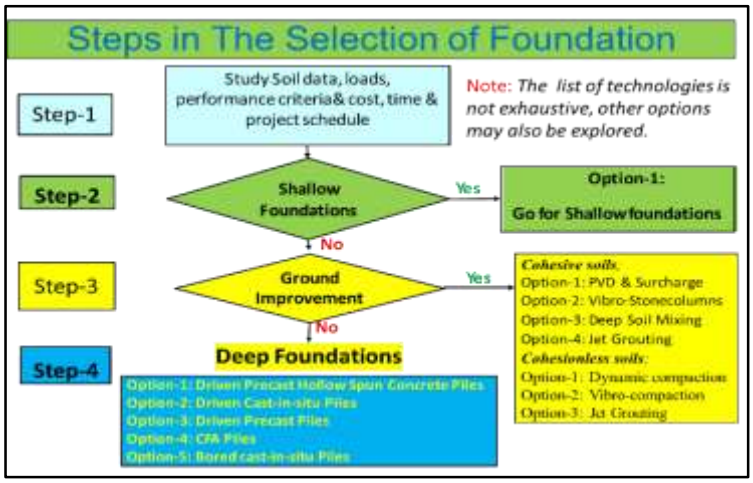


Fig. 2. Flow diagram for Selection of Foundation

2.0 Case Studies with different Application for the Assessment Oo the Ground Condition / Control and Selection of Foundation System:

2.1 Use of Geophysical Method to Determine Cavities/Voids and Dynamic Properties of Soil

2.1.1 MASW (Multichannel Analysis of Surface Wave) is a Geophysical Investigation technique, a non-destructive seismic survey method which generates a shear-wave velocity (Vs) profile (i.e. Vs versus depth), conducted to detect various soil parameters in addition to the presence of underground voids or cavities. The characteristics of soil materials (e.g. shear modulus, elasticity modulus, bulk modulus, fundamental vibration frequency, seismic amplification and Poisson's ratio) are closely related to the shear wave (S) velocity. Site characterization by evaluating velocity parameters at a shallow level is significant to establish the earthquake shaking on the ground surface. Therefore, the determination of S velocity variations of subsurface layers is important in geotechnical engineering.

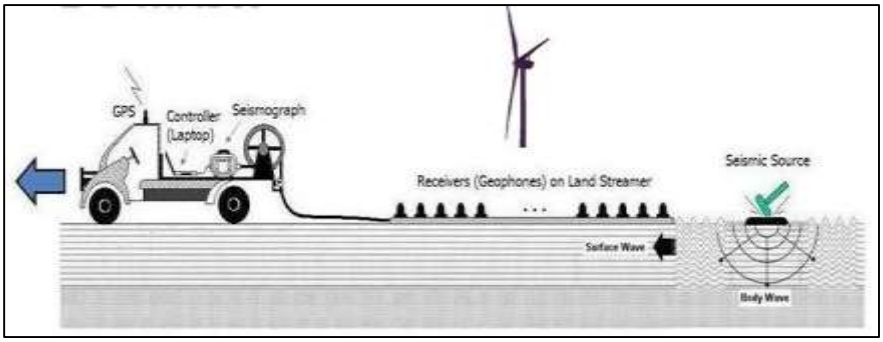


Fig. 3. 2D MASW method

2.1.2 This type of data acquisition configuration resembles a conventional seismic refraction survey. However, MASW field parameters (e.g. source power, sampling interval, record length) are more flexible than seismic refraction or reflection survey. This method is more predominant in gulf countries and is often use to collect various informations.

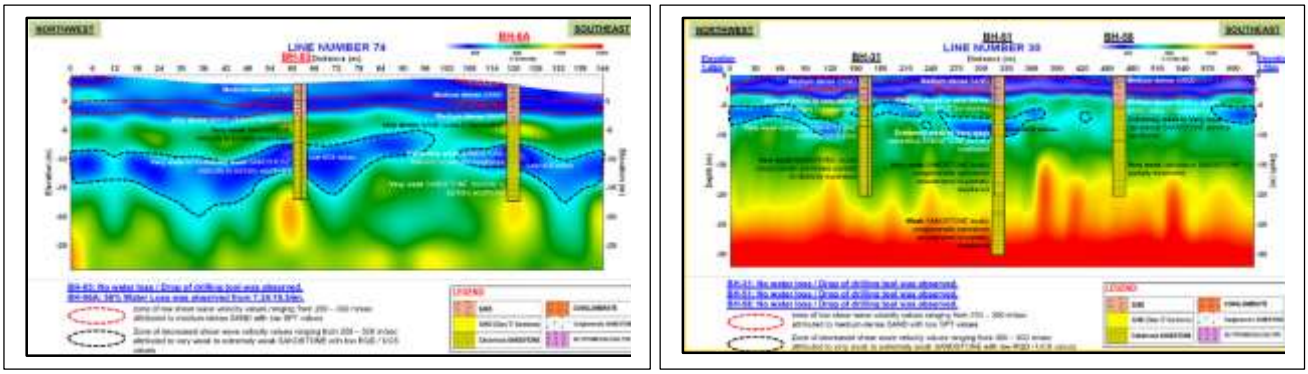


Fig. 4. MASW Plots Mapped with Boreholes

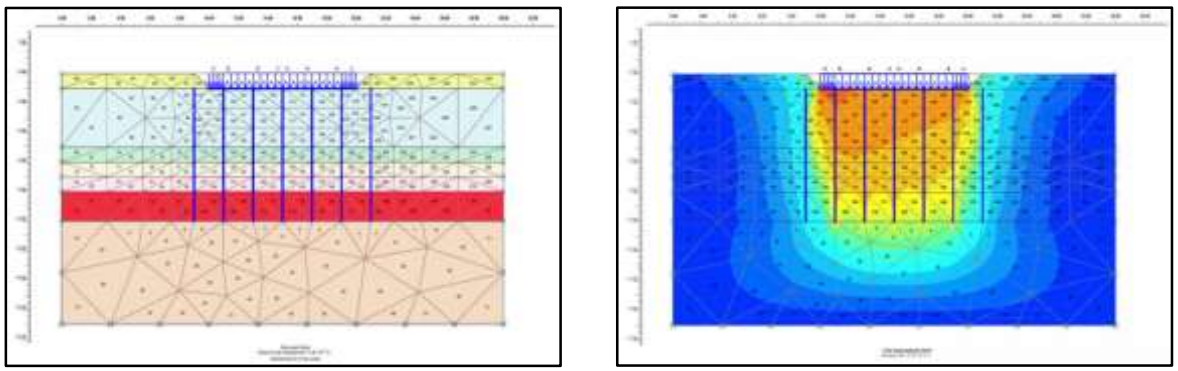
2.1.3 Conclusion: The subsurface conditions across the site can be classified with different seismic layers w.r.t shear wave velocities. The Images use a temperature colour scale, with blue showing areas of low velocity and dark red areas of high velocity. In general, a cavity can be identified in a 2D plot of the shear wave velocity (Vs) with high reliability, if a small area of low dynamic stiffness (low values of Vs) is embedded within a zone of higher stiffness (higher –Vs values). In addition, other parameters such as type, shape, and size of the anomalies, type of geologic formations, and the geological conditions under the site have to be considered.

2.2 Jet Grout for Soil Mass Stabilisation

2.2.1 Project located in CIS country comprises of simple cycle gas turbine power plant. Soils comprises of SP to SM (Sand to Silty Sand) till 20m to 30m borehole depth. Ground water table is encountered at shallow depth, being aggressive with high percentage of sulphates and chlorides. Requirement of bearing capacities for GTG, Stack and other heavy structures were in range of 150kPa whereas maximum available bearing capacity was around 100kPa. Hence considering local practice and engineering evaluation, Mass stabilization in the form of Jet Grouting was employed to introduce stiff element to transfer the load from ground surface to a soil stratum capable of supporting the load.

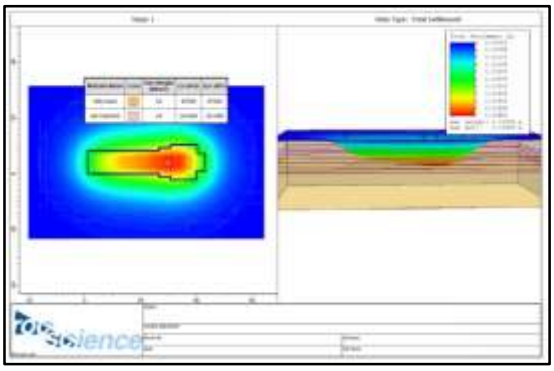
2.2.2 As per design, Jet grout columns of 800mm diameter and 9m length were found sufficient to achieve required bearing pressure of 150kPa without instigating settlement limits. Spacing of jet grouting was kept as 2M c/c in square grid pattern. A load transfer platform of 1m thick is provided between jet grout columns and structure foundation. Permissible settlement limits as per design criteria is set as 25mm. Hence, settlement assessments were done for virgin soil condition and after mass stabilization against maximum pre described limits.

2.2.3 Jet grout columns were introduced as embedded beam row element in Finite element analysis using plaxis 2d to conduct settlement analysis. In parallel, equivalent stiffness was used to model volume element for continuum mechanics analysis in Settle3D. Fig 5. shows geological ground model of jet grouted columns in Plaxis 2d and Settle3D



a. Plaxis Input for Jet Grouting

b. General Vertical Displacement Profile after Jet Grouting (13.02mm)



c. Settlement Analysis of the Settle3D for after Ground Improvement (s=5.35mm)

Fig. 5. Plaxis 2D and Settle3D analysis on virgin soil for GTG Foundations

2.2.4 Conclusion: Post ground improvement enhancement ratio was observed quite high from theoretical analysis, however, conservatively, ratio of 25% was taken for further assessment of all static and dynamic parameters. Software analyses demonstrated that actual problem for GTG foundation was not bearing capacity but was limited to settlement of soil.

2.3 Densification o Fly Ash Deposits equipped with Rammed Stone Column

2.3.1 An Indian project site consist of fly ash deposit varying from 7.5M to 18M depth. Field SPT value varies from 4 to 22 till 18M depth. Very weak to moderately strong granitic gneiss encountered to the final explored depth of 30M. In addition, site was susceptible to liquefaction. Hence, looking to the site conditions and findings from geotechnical investigation report, ground improvement using rammed stone column in triangular pattern at spacing of three times diameter was proposed. All open foundations requiring less than 10 T/M^2 bearing capacity shall be supported on 900 mm diameter stone column in equilateral triangular pattern at 1.8M c/c (cross section as shown in Fig. 6). For load greater than 10 T/M^2, stone column of 600 mm diameter along with piles shall be provided (cross section as shown in Fig. 7). Sand blanket of 500 mm over stone column is proposed as drainage layer. Extension of 1000 mm on the edges outside the stone column is provided to cater bulging due to foundation load.

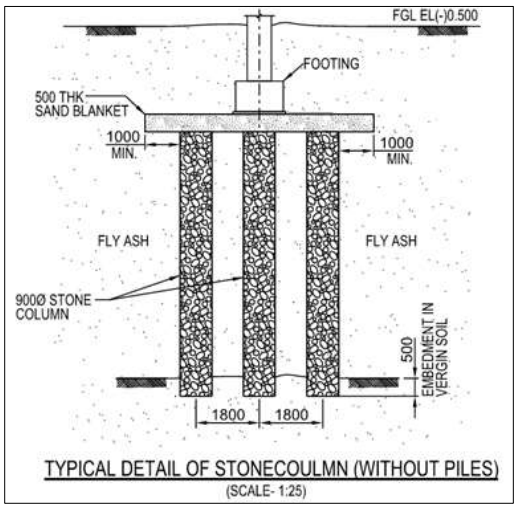


Fig. 6. Typical detail of stone column for Open foundation

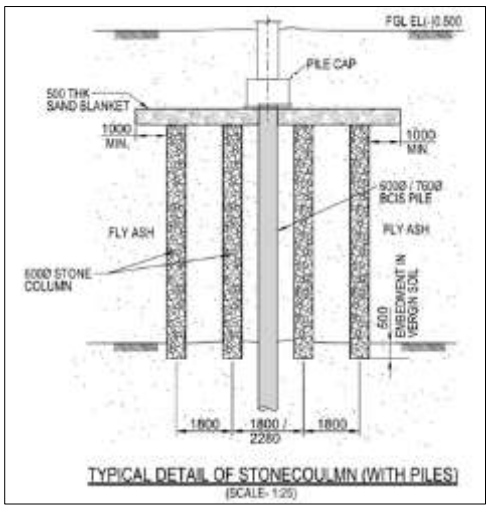


Fig. 7. Typical detail of stone column with piles

2.3.2 To validate design assumptions and parameter enhancement, pre and post SPT's along with SCPT's were conducted followed by full scale field load test on individual and group of stone column. Initial stone column test on single column (ISCT–3) and on group of stone columns (ISCT–4) for open foundation are conducted at different location as shown in Fig. 8. Results of pre and post CPT near stone columns are presented in Fig. 9.

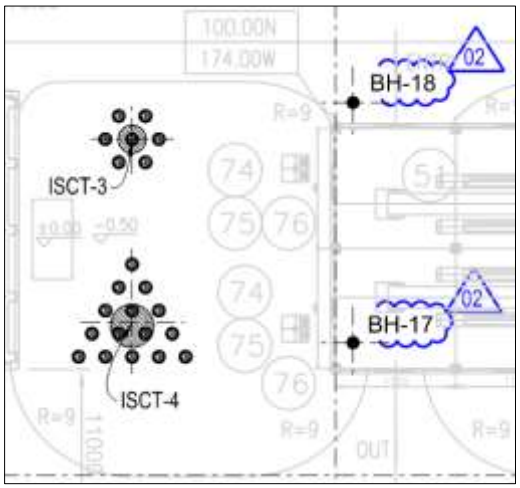


Fig. 8. Initial stone column test layout for individual and group of stone column

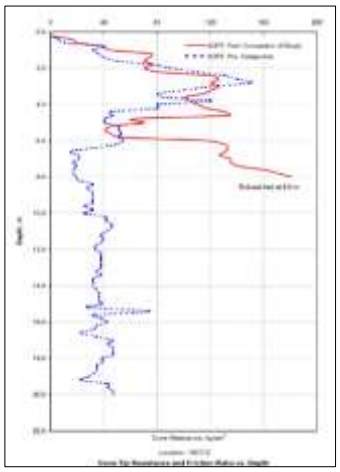


Fig. 9. Cone Tip Resistance and Friction Ratio v/s Depth for ISCT - 4

2.3.3 Similar full scale load test at different locations were performed for 600mm diameter piles with 600mm diameter stone column as shown in Fig. 10. Due to presence of fly ash to maximum 18.0 m depth, lateral pile capacity for entire project was very low. Typical Load settlement graph of lateral pile load test for 600 mm dia. pile from group 2 is shown in Fig. 11.

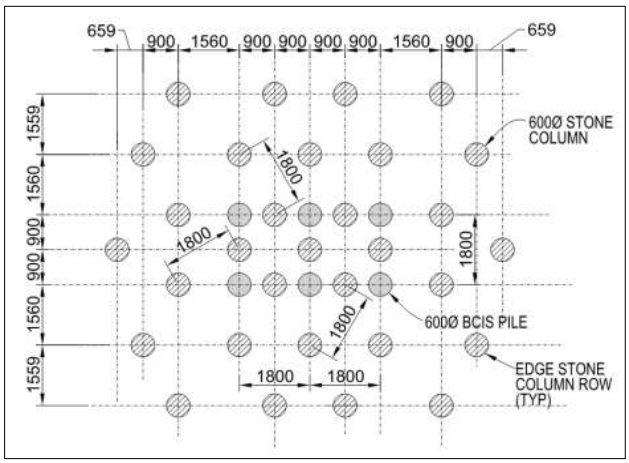


Fig. 10. Stone column configuration for 600mm dia. piles

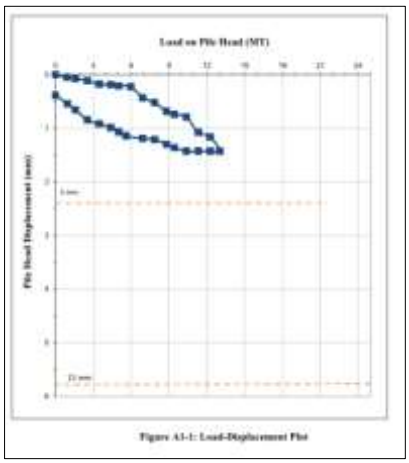


Fig. 11. Load v/s Settlement of 600 mm dia. piles

2.3.4 Conclusion:

a. Insertion of Stone Column shows that gain in shear strength happened significantly and fly ash deposit got densified after addition of stone aggregates.
b. The percentage increase in in-situ properties after installing stone column varies from 130 to 450.
c. Pile load test with single row of stone column around piles and two rows of stone column around piles were conducted. Results shows considerable amount of gain in lateral capacities with two rows of stone column.

d. Lateral deflection has reduced to a greater extent. In addition, liquefaction susceptibility also mitigated efficiently.

2.4 Ground Improvement using Prefabricated Vertical Drain (Pvd)

2.4.1 Project in Bangladesh had soft clay extending to 9M depth below existing ground level (EGL) where SPT ranges from 2 to 7, followed by salty sand. Susceptibility of liquefaction was also high. Proposed finished grade level (FGL) was approximately 4.5M above EGL which was supposed to be done by dredged sand. Hence, after evaluating different foundation options, ground improvement using PVD was assigned. PVD provided opportunity to convert piled foundation to open foundation where bearing pressure requirement were in the range of 100kPa to 120kPa. Entire plot plan was categorized based upon load requirement and different spacing viz. 1.1M, 1.2M & 1.5M in triangular pattern was designed for PVD installation.

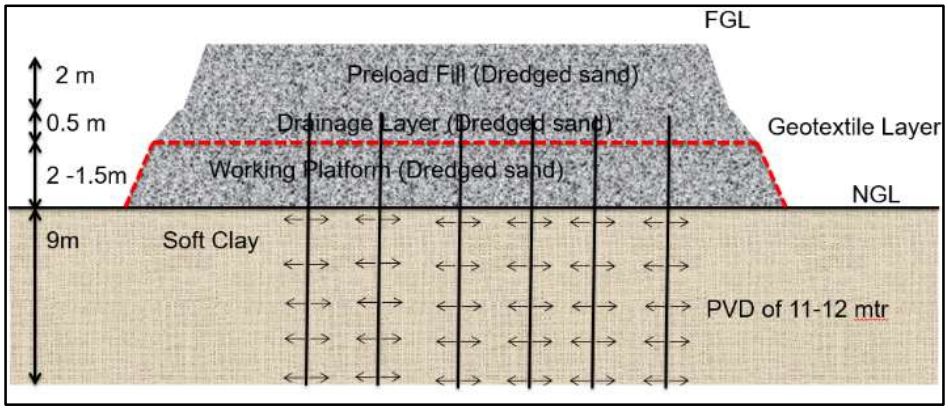


Fig. 12. Ground Improvement using Prefabricated Vertical Drain

2.4.2 For pressure requirements greater than 120kPa, pile foundation was selected. 6 month waiting period was recommended for proper consolidation in order to completely eliminate negative drag and mobilization of clay strength to get advantage of ground improvement. Rapid squeezing out of water helped in early pore water dissipation, which resulted in increased stiffness values. Hence, pile foundation with desired capacity suffices the requirements.

To measure the movement of soil, settlement monitoring system was installed. This helped in gauging the settlement after PVD installation.

2.4.3 Conclusion:

a. Uniform settlement provided better site control for grade slabs, sumps, drains, pits and other lightly loaded facilities.
b. Eliminated the chances of any negative drag on piles for long term performance

3.0 Discussion and Conclusion:

3.1 The use of geophysical investigation along with traditional geotechnical investigation play an important role in assessment of sub-surface conditions/remediation and selection of optimal foundation system leading to the successful completion of any project meeting the overall cost and schedule. Use of few ground improvement techniques for industrial applications have been depicted above while discussing the case studies. Apart from the above methods, there are many more practices and techniques in geotechnical engineering which are used to collect information of soil strata and enhance the existing sub-surface condition. The method selection varies from site to site based on project data and geophysical conditions.

About Authors

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Born on 24 April 1984 and graduated in Civil Engineering from the The MS University of Baroda, Vadodara in 2006. He completed his Master's degree in Soil Mechanics and Foundation Engineering from NIT Surat in 2008. He has more than 16 years of experience in Geotechnical field and has gained experience in Steel & Power sector, Industrial, Transportation, Infrastructure, Marine, Port, Harbour, Railways, Tunnels, Metro, Underground Structures-Stations and Specialized Geotechnical works. He is working as Assistant General Manager in L&T – Sargent & Lundy Limited and operating from Vadodara, Gujarat, India.

Vishal Pathak

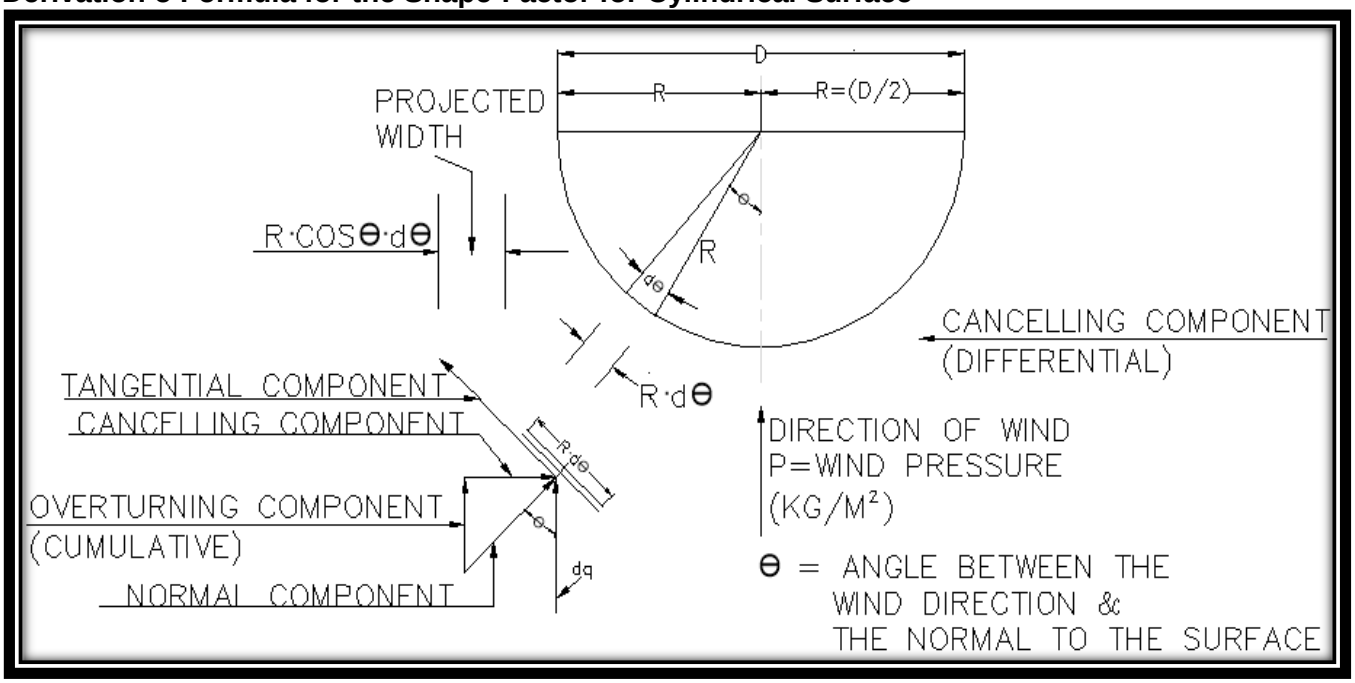
Born on 12 December 1994 and graduated in Civil Engineering from IET Lucknow in 2017. He completed his Master's degree in Geotechnical Engineering from IIT Kharagpur in 2020. He has more than 3 years experience in Geotechnical Engineering and presently he is working as an Executive Engineer in L&T- Sargent & Lundy Limited and operating from Vadodara, Gujarat, India. The firm is committed to providing complete power plant engineering services.

A Revolutionary Approach to calculate the Wind Load on Steel Towers under Diagonal Wind Condition

Girish A Desai
"Bhavani Consultant"

1. It is customary to assume that for Square or Rectangular based Steel Towers, Diagonal Wind Condition is more critical as compared to the Orthogonal Wind Condition. At best, we can call it axiomatic and superficial because it is based upon the apparent data that two corner legs are lying on the Neutral Axis and there is only one pair of Corner Legs to resist / balance the overturning moment.
2. However, if we re-examine this "Customary Assumption" by starting from First Principles, we will conclude which is just the opposite – viz. Diagonal Wind Condition is NOT Critical and Orthogonal Wind Condition shall govern the Design. This will lead us to developing more economical Design without compromising safety.
3. To substantiate this fact, we propose to first take up the "Shape Factor" for Cylindrical surfaces -which is Analytically more complex but almost universally accepted – say for tall Chimneys in Thermal Plants for developing "Natural Draft". Practically most of the International Standards recommend the Shape Factor = 0.60 for Cylindrical surfaces. Reference may please be made to the Attached Annexure – 1, wherein we have given the complete derivation / intermediate steps of the process using the method of Integral Calculus. Here, the theoretical value comes to 0.667 (2/3). On closer investigation, we understand that the recommended value = 0.60 is based upon the experimental study from the "Wind – Tunnel" Test. (the factor responsible for this lower value is the formation of "vortices" on the leeward side).
4. The next step is to extend this logic to the simpler case of a square / rectangular based tower under Diagonal Wind Loading. It is pertinent to remember that the angle – "θ " is the angle between the direction of flow of wind and the line Normal to the surface exposed to the wind. This angle θ is constant and hence can be taken out of the Integration sign. The result is – the shape factor comes to 0.5 (1/2). Alternatively, the Reduction Factor with respect to the Orthogonal Wind Load will be equal to 0.707 ($1/\sqrt{2}$).
5. During the past twenty-five years or so, we have tried to convince the concerned Authorities to amend the relevant Standard Codes of Practice. The response was shocking and unbelievable – practically all were convinced about the fundamental correctness of our approach, but they also felt that this is too revolutionary to be accepted by the community and it is not worth trying to propagate the idea!
6. What next? We can try "Ulti Ganga". Instead of trying to convince people about our Analytical finding, we can first prove it experimentally in a "Wind Tunnel" Test with the help of stain-gauges and then support it with the Analytical Derivation. This may perhaps go down their throats. One such practical application of this principle is the near horizontal wind screens of modern cars as compared to the near vertical wind screens in the old cars prevalent about twenty years back.
7. If the above postulation is converted into a research project & if the research becomes successful, it will take the tower industry by storm. The weights of all the towers will be drastically reduced. This is going to save not only the cost but precious resources as well.

Derivation o Formula for the Shape-Factor for Cylindrical Surface



We shall consider unit height

(Perpendicular to plane of paper)

Wind Load on the element = *dp*

$dq = p \cdot R \cos\Theta \cdot d\Theta$

Normal Component = $p \cdot R \cos\Theta \cdot \cos\Theta \cdot d\Theta$

Overturning Component = $p \times R \cos\Theta \times \cos\Theta \times \cos\Theta \times d\Theta$

$= p \times R \cos^3\Theta \times d\Theta$

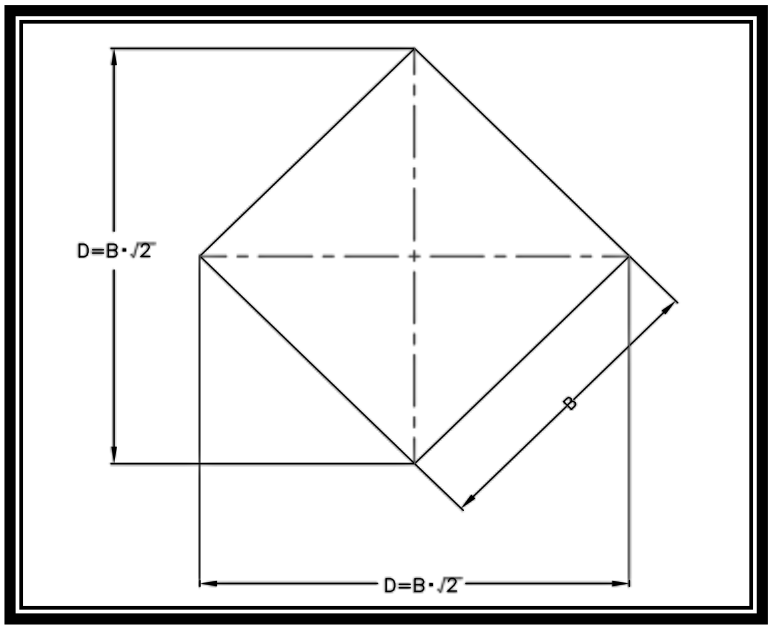
$\therefore dp = p \times R \cos^3\Theta \times d\Theta$

$\therefore$ Total Force = $P = p \int_0^{\pi/2} 2 \cdot R \cdot \cos^3\theta \cdot d\theta$

$\therefore P = pD \int_0^{\pi/2} \cos^3\theta \times d\theta = pD \left[\sin\theta - \frac{1}{3}\sin^3\theta \right]_0^{\pi/2}$

$= \frac{2}{3} \times D \times p \qquad \therefore \text{Shape Factor} = \frac{2}{3} = 0.667$

Adaptation of above formula to Diagonal - Wind Loading on Tower- Structures



The general formula is

$P = p \cdot D \cdot \int_0^{\pi/2} COS^3\theta \cdot d\theta$

This case is much simpler because

The Angle θ is constant $= \frac{\pi}{4} = 45°$

Hence $COS^3\theta$ can be taken out of the $\int$ Integration sign

$$\therefore p = p \cdot D \cdot COS^3\theta = p \cdot D \cdot \left(\frac{1}{\sqrt{2}} \times \frac{1}{\sqrt{2}} \times \frac{1}{\sqrt{2}}\right)$$

$$= \frac{1}{2\sqrt{2}} \cdot p \cdot D$$

For Stress-Calculations, This has to be increased by $\sqrt{2}$

Hence, the shape factor = $\frac{1}{2}$ = 0.50

(The recommended value in IS:875, S =1.20)

About Author

Er. Girish Desai Graduated from The MS University of Baroda, Vadodara in the year 1964 with First Class and Distinction, standing first in the University.

In 1966, he completed his Master of Technology from the Indian Institute of Technology, Mumbai with specialization in **Structural Engineering**. His dissertation was adjudged as an **"Excellent**".

He joined M/s Kamani Engineering Corporation (now known as KEC International Ltd) Mumbai in 1967 as Design Engineer.

In 1977, he joined M/s Tata exports as a Chief of Design department.

Since 1981 he has been working as an independent consulting structural engineer under the title of "Bhavani Consultant".

Use of Tubular sections for Transmission Towers

Bipin B Shah – Sr. Vice President (Engg.)
Jyoti Structures Limited

The Indian power sector is witnessing the adoption of advanced tower and structure technologies for expanding the network with minimum right of way (RoW), greater structural integrity, reduced maintenance costs, improved energy efficiency and better grid reliability. The deployment of tower designs such as monopoles and multi-circuit towers that meet these requirements are gaining traction.

In recent years the requirement of designing transmission line towers for cyclonic condition for coastal area, has become a great challenge. Due to much heavier loads, towers are becoming very heavy. Also, the towers are actually not designed for cyclonic condition, where there would be torsional loads on tower and also there could be effect of sand storms / dust storm on tower members. Making provision for higher wind pressure by applying K4 factor, makes tower stronger but may not be sufficient for actual cyclonic conditions.

In the circumstances, it is desirable to effectively reduce the effect of wind velocity on tower member. The best possible option to make tower stronger against wind using aerodynamic members. Circular sections are most aerodynamic shape.

Circular steel sections are now a days easily available in India and are already used in other industry like telecom tower, PEB's, etc.

There are many advantages of tubular sections, such as:

1. Section modulus of circular section is much more compared to the same weight of angle sections
2. To maintain slenderness ratio of tubular section, same as that of angle sections, much higher unsupported length can be provided.
3. Due to much higher unsupported length, number of supporting members (redundant) will be very less
4. Base width of towers can be reduced, resulting lesser cost of ROW.
5. Erection will be easy and faster, as compared to angle lattice towers
6. Number of bolts to be connected will be much less.
7. Cost of tower will be much less.

Challenges of adopting tubular sections.

1. Welding is involved as, ends of tubular members to be welded.
2. Cost of raw material for tubular sections is slightly more than cost of angle section.
3. Strict quality checking of welded connection is required at factory.

It may be noted that transmission towers with tubular sections are already in use in countries like China, South Corea and Japan.

Properties of tubular sections are available is respective IS code. Also, brochures of reputed manufacturers are available for various mechanical properties being used in design of towers.

भारतीय मानक
Indian Standard

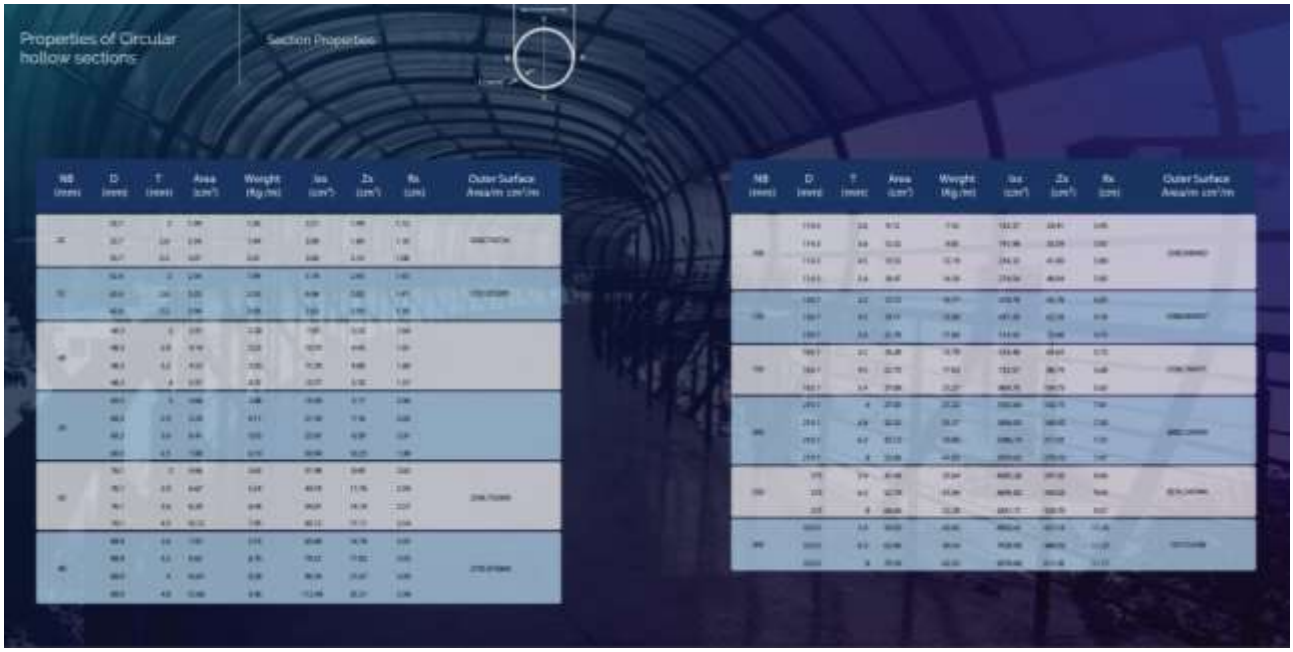
IS 1161 : 2014

इस्पात नलिकाएँ संरचनात्मक उपयोगों के लिये — विशिष्टि

(पाँचवाँ पुनरीक्षण)

Steel Tubes For Structural Purposes — Specification

(Fifth Revision)



Connections:
Tube to tube connections are not as simple as angle to angle connections. There are various type of connections possible. Which can be as per convenience of designer and fabricator.

Tower testing:
The procedure for type testing for towers with tubular sections, will be same as that of lattice tower with angle sections. However, the behavior of connections need to be established. Hence type testing of lattice towers with Tubular sections is must, till the connections are established.

Provision in IS code:
In the latest IS code for design of transmission towers (IS 802), there is provision of welded connections. Hence, there is no restriction on designer for adoption of tubular sections for transmission towers.

Recommendations:
The designers and industry utilities are advised to use the superior geometric properties of tubular sections. High grade tubular sections ensures better performance under high wind / cyclonic condition with lighter weight of structural system. It encourages use of long unsupported lengths requiring lesser members.

About Author

Bipin B Shah (M: 9824409026) has been serving the transmission line industry since more than 40 years in the field of Design and Engineering. He is member of many IS code committees, Member of CBIP panel of experts.

He was awarded as "**Distinguished Member**" from CIGRE – Paris for year 2014. Safe and optimized design of towers and foundation, and adopting new approach for optimizing cost of transmission line is his core competency.

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મધ્ય ગુજરાત વીજ કંપની લિમિટેડ

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- રાષ્ટ્રવિકાસ

મધ્ય ગુજરાત વીજ કંપની લિમિટેડે તેના માનવંતા ગ્રાહકો માટે Android Mobile App (MGVCL App) લોન્ચ કરેલ છે. જે વીજ ગ્રાહકો મોબાઇલ પર Google Play Store દ્વારા ડાઉનલોડ કરી નીચે જણાવેલ તથા બીજા પણ લાભો સાથે મોબાઇલ મેસેજ સેર્વિસ પણ મેળવી શકશે.

૬ મહિનાના વીજ બિલ ડાઉનલોડ ની સુવિધા | છેલ્લા ૬ મહિનાના પેમેન્ટ ની માહિતી | ત્વરિત ઓનલાઇન પેમેન્ટ ની સુવિધા

કેક સંદેશ બાબત : મધ્ય ગુજરાત વીજ કંપની લિમિટેડનાં માનવંતા ગ્રાહકોને જણાવવાનું કે MGVCL દ્વારા ૧૦ આંકડા નાં મોબાઈલ નંબર અથવા બીજા કોઈ નામ થી વીજબીલ ચુકવણી અંગે નાં SMS મોકલવામાં આવતા નથી. તેમજ આવા છેતરામણા SMS દ્વારા માગવામાં આવેલ કોઇપણ પ્રકારની માહિતી કે OTP આપવા નહીં. તેમજ આવી બાબતોની જાણ નજીક ની MGVCL ની કચેરીને કરવી અથવા આ પ્રકારનાં SMS અવગણવા.

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