SRI LANKAN GEOTECHNICAL SOCIETY
A Member Society of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

ANNUAL CONFERENCE 2023

Co-organized by
China Harbour Engineering Company (CHEC)

NEAR/OFFSHORE GEOTECHNICAL DESIGN AND CONSTRUCTION

23rd October 2023
At Galadari Hotel, Colombo, Sri Lanka

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SRI LANKAN GEOTECHNICAL SOCIETY

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MESSAGE FROM THE PRESIDENT

Eng. K. L. S. Sahabandu

It is with great pleasure and honour, indeed, that I send this message to the Annual Conference - 2023 of the Sri Lankan Geotechnical Society (SLGS) on the theme of **"Near / Offshore Geotechnical Design and Construction"**, which is conducted customarily preceding the Annual General Meeting.

SLGS, as you may have learned, was inaugurated nearly three and a half decades ago, under the visionary leadership of Prof. A. Thurairajah who returned to his motherland after an outstanding and influencing career at the University of Cambridge and was eventually considered as the father of Soil Mechanics in Sri Lanka.

Since its inception, SLGS has been playing an important role in the advancement of the discipline of Geotechnical Engineering in Sri Lanka as the professional body for practicing engineers and academia active in the field of geotechnical engineering and with an enhanced zeal with her becoming a Member of the *International Society for Soil Mechanics and Geotechnical Engineering* (ISSMGE).

Multi-faceted activities of SLGS included Conferences, Seminars, and Workshops conducted by world-renowned experts in the field as resource personnel, Geotechnical Forum – an evening lecture held monthly providing opportunities for the Sri Lankan Geotechnical Engineering Fraternity to learn from the experiences of the academia and the practitioners from local and overseas origins and Technical Visits to project sites of geotechnical relevance.

Moreover, SLGS conducted three major international conferences in 2007, 2015, and 2021 with the participation of a large number of internationally renowned academics and practitioners in the field of Geotechnical Engineering.

During, this tenure we have had Six Forums on contemporary topics in Geotechnical Engineering. As the final event of this tenure, we have planned our Annual Conference preceding the Annual General Meeting of SLGS as per its statutes. We have organized this conference in Hybrid mode to get the maximum benefits of both physical and virtual modes.

We have a great line of experienced resource persons today selected from the industry who have a thorough knowledge of practitioner's requirements to share their valuable perspectives under the theme of "Near / Offshore Geotechnical Designs and Construction". We are grateful to them for joining us on this knowledge-sharing journey with all their busy schedules. It would indeed be a thought-provoking conference of topical interest to all of us in the Geotechnical Engineering fraternity.

Ladies and Gentlemen, it is my privilege on behalf of the Society to welcome and appreciate Dr. Anil Joseph, President of the Indian Geotechnical Society, Dr. Jiaer Wu, R&D Manager and Lead Geotechnical Engineer of CHEC (USA) Ltd., Eng. Yin Changquan, Technical Director, CHEC South Asia Regional Office in Sri Lanka, Dr. Tang Qunyan, Geotechnical Expert of CHEC- Sri Lanka Division and Ir. Sridhar Krishnan, Technical Director, Marine and Coastal Engineering Division of COWI, Singapore accepting our invitation as Resource Personnel.

With immense gratitude, I welcome all distinguished Invitees including the SLGS Past Presidents and SLGS past and present Executive Committee Members for joining us by gracing this event.

I greatly appreciate the Members of China Harbour Engineering Company the Co-organiser of this event and our Sponsors, M/s Maga Engineering (Pvt.) Ltd., M/s Sanken Construction Pvt. Ltd., M/s Sierra Construction (Pvt.) Ltd., M/s ELS Construction (Pvt) Ltd., M/s Geotech (Pvt) Ltd and, M/s. Pile Test Consultants (Pvt.) Ltd. for their assistance and generous sponsorship of this event.

Also, I warmly welcome Prof. Athula Kulathilaka for accepting our invitation to Chair the Conference. I am grateful to all the members of the Organizing and the Executive Committees, particularly Prof. Udeni Nawagamuwa, Eng. Mahinda Rathnasiri, Dr. J.S.M. Fowze, Dr. Sampath Hewage, Eng. (Ms) Wathsala Galhena, and Eng. Janaka Priyantha for being with me together and offering their unstinting support to make this event successful.

I also wish to welcome all the participants who will make this event a remarkable one indeed. Undoubtedly, this Conference will be another exciting and informative event by SLGS and I wish everyone a very effective and fruitful Conference.

Thank you.

Eng. K.L.S. Sahabandu
B.Sc.Eng.(Hons.), Pg. Dip.(Hyd.Eng.), M.Sc.(Struct. Eng.), C.Eng.,
M.I.C.E.(London), M.Cons.E.(S.L.), F.I.E.(S.L.), Hon. F.S.S.E.(S.L.)
President-SLGS

SLGS ANNUAL CONFERENCE, 2023

"NEAR/OFFSHORE GEOTECHNICAL DESIGN AND CONSTRUCTION"

PROGRAMME

8:30 – 9:00	Registration of Participants / Morning Tea	
9:00 – 9:15	Ceremonial Opening	
9:15 – 9:25	Welcome Address	**Eng. K.L.S. Sahabandu** President - SLGS
9:25 – 10:10	**Keynote Address:** Pre-feasibility Studies, Design and Execution of Port Structures: Case Studies.	**Dr. Anil Joseph** President - Indian Geotechnical Society
10:10 – 10:30	Presentation by the Co-organizer, CHEC	
	Morning Session: Session Chair: Prof. Athula Kulathilaka	
10:30 – 11:15	A Case Study on Post-Stone Column Construction Marine Sediment Impact, Tibar Bay Deepwater Port.	**Dr. Jiaer Wu** R&D Manager and Lead Geotechnical Engineer, CHEC (USA) Ltd.
11:15 – 12:00	Application of New Vacuum Preloading Method in Dredging Ultra-Soft Soil.	**Eng. Yin Changquan** Technical Director, CHEC South Asia Regional Office Sri Lanka
12:00 – 13:00	Lunch Break	
	Afternoon Session: Session Chair: Prof. Athula Kulathilaka	
13:00 – 13:45	The Island and Tunnel Works of Hongkong – Zhuhai – Macao Bridge Project.	**Dr. Tang Qunyan** Geotechnical Expert, CHEC- Sri Lanka Division
13:45 – 14:30	Coastal Resilience through Urban Expansion: Lynetteholm Case Study.	**Ir. Sridhar Krishnan** Technical Director, Marine and Coastal Engineering COWI Singapore
14:30 – 14:40	Vote of Thanks	**Prof. Udeni Nawagamuwa** Hony. Secretary - SLGS
14:40 – 15:00	Evening Tea	
15.00 – 16.45	26th Annual General Meeting of SLGS	

SLGS ANNUAL CONFERENCE, 2023

"NEAR/OFFSHORE GEOTECHNICAL DESIGN AND CONSTRUCTION"

CONTENTS

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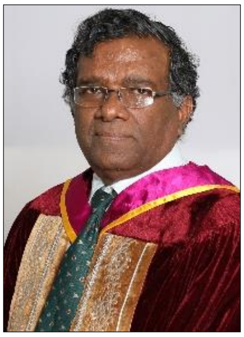


Prof. Athula Kulathilaka

Session Chair

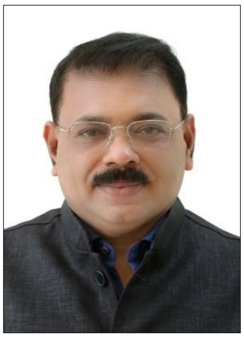
SLGS Annual Conference, 2023

Prof. Athula Kulathilaka, a prominent figure in the Department of Civil Engineering at the University of Moratuwa, has made enduring contributions to the Sri Lankan Geotechnical Society (SLGS) and the field of geotechnical engineering. Prof. Athula Kulathilaka's substantial contributions to the Sri Lankan Geotechnical Society extend beyond his role as a Past President and Secretary who contributed to organizing various activities throughout the past decades. He has been a dedicated representative of the SLGS on various international platforms, including international conferences, and has also played a significant role as a resource person for knowledge-sharing workshops.

As the representative of SLGS at international conferences, Prof. Kulathilaka has effectively showcased the achievements and capabilities of the Sri Lankan geotechnical community on the global stage. This involvement has not only expanded the society's international reach but has also fostered collaborations and knowledge exchange with experts around the world.

Prof. Kulathilaka also works as a resource person for knowledge-sharing workshops organized by SLGS and has been instrumental in disseminating geotechnical expertise. These workshops have provided a platform for professionals and students to gain valuable insights and skills, contributing to their growth and development in the field of geotechnical engineering.

His forthcoming role as a Session Chair for the SLGS's Annual Conference in 2023 is yet another testament to his continued commitment to advancing the Geotechnical Engineering community in Sri Lanka. As a Session Chair, he will be responsible for guiding discussions and proceedings, ensuring a productive exchange of knowledge and ideas among conference participants.



KEYNOTE ADDRESS:

"PRE-FEASIBILITY STUDIES, DESIGN AND EXECUTION OF PORT STRUCTURES: CASE STUDIES"

by
Dr. Anil Joseph,
President - Indian Geotechnical Society.

Dr. Anil Joseph had done his graduation and post-graduation in Civil Engineering from N.I.T. Surat. He was awarded the Ph.D. from NIT, Calicut, for his work on "Precompression of Soft Marine Clays".

He is the Managing Director of Geo-structurals (P) Ltd. a leading foundation & and structural consultancy firm based in Cochin. He has provided foundation and structural consultancy for more than 3000 High rise structures including many landmark multi-storeyed and Infrastructure projects in India and abroad in the last 30 years. His design of the Nippon Toyota showroom at Kalamassery, Platinum Mall at Maradu, Lulu Grand Hyatt Hotel & Convention Centre, Bolgatty Island, Kochi, Hyatt Regency Hotel and Convention Centre, Thrissur and Trine Green Lagoon Resort at Kochi has won the ICI-UltraTech Award for Outstanding Concrete Structure of Kerala in the building category in the year 2012, 2017 2018 and 2020 respectively. He is the Managing Director of CECONS (P) Ltd. a construction firm specializing in the execution of pile foundations and also the Director of Engineers Diagnostic Centre (P) Ltd a firm specializing in Geotechnical investigation and Retrofitting works.

He is the President of the Indian Geotechnical Society for the years 2023 and 2024 and is a National Council Member of the Institution of Engineers India in Civil Division for the term 2021 to 2025. He is the Governing Council member of the Indian Association of Structural Engineers. He is also the Chairman of the Indian Concrete Institute, Kochi Centre, Vice Chairman of the Builders Association of India, Cochin Chapter, and Governing Council member of the Builders Association of India. He was a National Executive Committee member of the Indian Geotechnical Society from 2012 to 2020 and is representing India in the International Technical Committee TC - 220 on "Field Monitoring in Geo Mechanics" and in the Asian Technical Committee AsRTC-14 on Smart Observation Methods. He is a member of State Committee in Institution of Engineers Civil Division, Executive Committee member of Institution of Engineers Kochi Local Centre, the Immediate past President of Association of Structural and Geotechnical Consultants, Kerala, Immediate Past State President of Graduate Association of Civil Engineers, the Past President of Association of Piling Specialists, Kerala, Honorary Secretary of Indian Geotechnical Society, Kochi Chapter, Member of Association of Contracting Engineers, Member of Deep Foundation Institute India, Managing Committee Member of Kerala Management Association and an adjunct faculty of Albertian Institute of Science & Technology, Cochin. He was also awarded a PhD from Open International University, for his work on "Prestressing and Hollow Shell Technology in Earth Quake Resistant Construction".

Dr. Anil Joseph is also involved in various social activities such as Vice President of the Regional Sports Centre, Kadavanthara, Cochin, He is an active Rotarian and was the Former Assistant Governor of Rotary District3201 and past president of Rotary Club of Cochin Downtown, etc. He was one of the top 10, Diamond Hall of Fame, New Age Icon Change Makers 2020. He is married to Rinna Anil, Director of Cecons Pvt Ltd, and is blessed with two children. Son, Akhil Anil who is doing his M.S. in Geotechnical Engineering from Texas A & M University U.S.A., and daughter Alina Anil doing her final year of Civil Engineering from VIT, Vellore.

Prefeasibility Studies Design & execution of Port Structures – Case Studies

Dr. Anil Joseph, President, Indian Geotechnical Society & Managing Director, Geostructurals Pvt Ltd
Email: aniljoseph@geostructurals.com

ABSTRACT: Offshore geotechnical engineering is a highly specialized field of civil engineering that deals with the study and analysis of the behavior of soils and rocks beneath the seabed. It plays a crucial role in various aspects of offshore construction, exploration, and resource development. Offshore geotechnical engineering is essential for the safe and successful development of offshore projects, from oil and gas exploration to renewable energy installations and infrastructure development. It ensures that structures are designed and constructed to withstand the unique challenges of the marine environment while minimizing environmental impact and maximizing cost-efficiency. Offshore development plays a major role in the growth of economy. Coastal areas comprise of very soft marine clays, which makes the construction in this area challenging. This paper deals with the few experiences of the author in handling various near/offshore projects. Case study 1 covers the feasibility study of Cochin Cruise Terminal, which is becoming leading cruise terminal of Kochi and includes a conceptual expansion, completed and operational phase of the project. Ground improvement in nearshore areas is essential to address the unique geotechnical challenges posed by coastal and marine environments. It not only supports safe and stable construction but also promotes the preservation of valuable ecosystems and helps safeguard against natural disasters and erosion, contributing to the overall sustainability of coastal regions. Case history of Ground improvement for Oil tank farm foundation at Willingdon Island, Cochin is presented in this paper. Beach restoration is essential for safeguarding coastal communities, protecting natural habitats, supporting local economies, and providing recreational and aesthetic benefits. A case study involving the process of restoration of Shanghumugham beach and road at Trivandrum is briefly described in this paper. Before constructing every port structures, feasibility studies are carried out. One such case for Development of Tadadi Port, Karnataka is mentioned in this study. The paper concludes with the case study of design of RO-RO Facility at Hazira, Gujarat. In summary, geotechnical studies in nearshore and offshore areas are crucial due to their unique challenges. These studies are essential for safe and sustainable construction and development in these environments.

1. INTRODUCTION

Offshore geotechnical engineering is a highly specialized field of civil engineering that deals with the study and analysis of the behavior of soils and rocks beneath the seabed. It plays a crucial role in various aspects of offshore construction, exploration, and resource development. These structures include submerged pipelines, oil platforms, and artificial islands. The applied loads and the weight of these constructions must not be too much for the seabed to support. Geohazards need to be considered as well. Because new fields are being developed farther offshore and in deeper water, requiring a commensurate adaptation of the offshore site studies, the necessity for offshore developments arises from the gradual depletion of hydrocarbon deposits onshore or near the coastlines. More than 7,000 offshore platforms are in operation today, with ocean depths reaching and surpassing 2000 metres.

India has a vast and diverse coastline that stretches over 7,500 kilometers (approximately 4,660 miles) along the Arabian Sea and the Bay of Bengal. This coastline is a significant geographic feature that plays a crucial role in India's social, economic, and environmental landscape. According to the Ministry of Ports, Shipping and Waterways, around 95 per cent of India's trading by volume and 68 per cent by value is done through maritime transport. It is serviced by 13 major ports (12 Government-owned and one private) and 187 notified minor and intermediate ports. India's major ports handled highest ever cargo of 795 million tonne in FY23. Government of India plans to build new greenfield ports and also built associated infrastructure such as railway lines through the 2015 established Sagarmala project, and National Maritime Development Programme.

Sri Lanka, an island nation located in the Indian Ocean, boasts a strategic and diverse coastline. Its extensive coastline, approximately 1,340 kilometers (around 833 miles) long, plays a significant role in the country's economy, trade, and culture. The coastline is home to numerous ports, each with its unique characteristics and contributions. Colombo Port, Hambantota Port and Galle Port are the major ports in Sri Lanka and there are few minor ports also. The coastal areas of Sri Lanka face various challenges, including coastal erosion, pollution, and habitat degradation. Climate change impacts, such as rising sea levels and extreme weather events, are major concerns. Conservation efforts and sustainable coastal management are ongoing to protect the environment and promote responsible tourism and development.

Near and offshore geotechnical engineering faces unique challenges due to the complex and dynamic nature of coastal and marine environments. The seabed can vary significantly, ranging from hard rock to soft sediments, which makes foundation design and stability assessments complex. Understanding the seabed's properties is essential for safe and efficient construction. These challenges include Wave and Wind Loading, Hydrodynamic Forces, Erosion and Sediment Transport, Corrosion and Marine Life, Environmental Impact, Siltation and Liquefaction, Rising Sea Levels, Cable and Pipeline Installation, Geohazards, etc. In summary, geotechnical studies in nearshore and offshore areas are crucial due to their unique challenges. These studies are essential for safe and sustainable construction and development in these environments.

2. DEVELOPMENT OF COCHIN CRUISE TERMINAL – CASE STUDY

2.1. About the Project

The Cochin Cruise Terminal, also known as the Cochin Port, is a prominent maritime gateway in the Indian state of Kerala. It plays a pivotal role in facilitating cruise tourism and trade activities in the region. The Cochin Cruise Terminal is situated in the city of Kochi, on the southwest coast of India, along the Arabian Sea. The port is strategically located, making it a vital entry point for international trade and tourism. In addition to attracting expanded air services to Kochi, the tourist traffic is also generating a new passenger cruise industry in southern India, with 55 cruise vessels calling in 2023-2024.

The project site includes about 10 to 15 acres of land and nearly 500 meters of shoreline to the south of Berth Q5. A plan was developed to leverage values that are present in the associated upland to support development costs of the cruise terminal. This plan includes a five-star hotel and a World Trade Complex consisting of an office tower and retail showcase. The cruise terminal is shown in **Fig.1**, itself is designed with modern wharf and passenger facilities, as well as organized loading and departure areas. Adjacent to the terminal, a portion of land is designated as a Kerala Village, featuring exhibits, kiosks, and performance areas. Based on this concept, a study was prepared of the business potential for each of four primary components; cruise passenger demand, hotel room rates, retail space rental, and office lease rates. In addition, the concept was developed into a detailed layout plan that included cost estimates, implementation schedule, operational requirements, and investment models. Finally, a detailed analysis of the financial feasibility was prepared to evaluate project viability.

Fig.1. Cochin International Cruise Terminal

2.2. Feasibility Study

The feasibility study for the Development of Cruise Terminal Cochin was awarded to MIR Projects, Cochin and Transystems, USA. The consortium entrusted Geostructurals Pvt Ltd, Kochi to carry out the geotechnical and structural design. Site area comprise of 15 acres of land, 500m of shoreline to south of Berth Q5. Cruise Terminal and Public Plaza including five star hotel, world trade complex comprising an office tower and retail showcase was envisaged for the project. The initial concept for the Cruise Terminal cum Public Plaza was for co-use of Ernakulam Wharf, Berth Q5 for cruise ships and for cargo ships as shown in **Fig.2.**

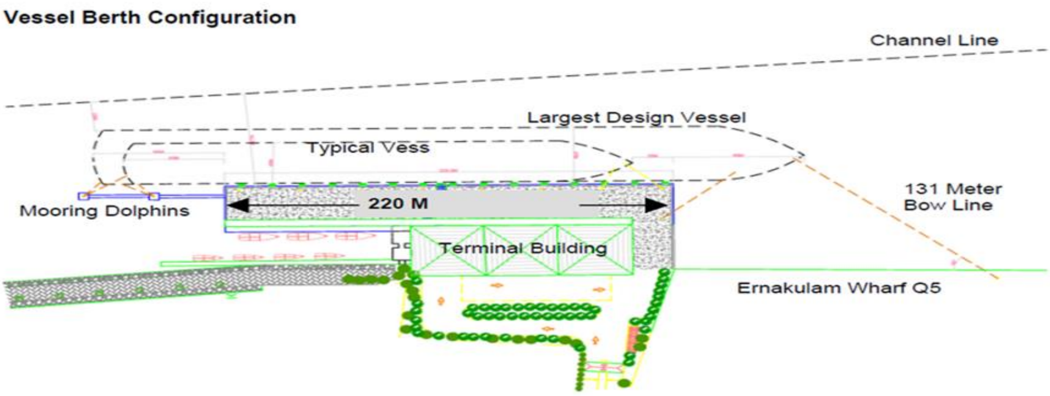


Fig.2. Initial Concept for Cruise Terminal cum Public Plaza

The **Fig.3** shows Design Plan & Terminal Layout, up on evaluation of this plan, and in consultation with marine experts and Port Trust management, the following issues were noted:

1. 150 meters would be inadequate berthing length and a portion of Q5 would be required
2. Vessel positioning along the berth would not be favorable
3. Significant reinforcement of the shoreline would be necessary
4. Excessive dredging costs would be encountered in maintaining the berth
5. Small boat activity and shoreline access would be impeded

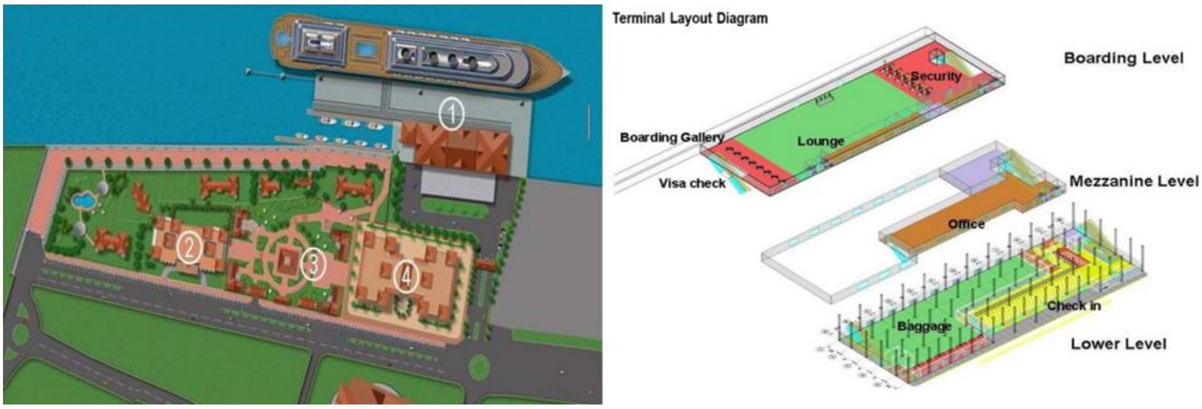


Fig.3. Design Plan and Terminal Layout

2.2.1. World Trade Complex - International Shopping Mall

This plan's shopping mall must operate as an easily accessible public destination that draws in cruise passengers, non-cruise passengers, and the local population while providing them with an excellent shopping experience. Nonetheless, the retail mall's main purpose will be to present premium Indian goods to the global market. In order to boost export and import opportunities in the Kerala region, the retail mall makes use of the trade opportunities that will be created by the new Container Transhipment Terminal and the foreign firms that will be housed in the World Trade Complex.

2.2.2. World Trade Complex – Office Tower

The office building is outfitted with contemporary amenities to serve as a top-tier business hub for global trading enterprises, including financial institutions, trading houses, shipping corporations, and banks. The interior specifications include fibre optic communication, centralised air conditioning, an executive exercise area, a business centre and office supply outlet, round-the-clock security, and other amenities that are expected by large international banking and trading companies in order to fulfil this function. Moreover, a friendly social and work environment is created by the nearby hotel restaurants, the outdoor plaza, and the shopping levels.

2.2.3. Hotel Tower

The planned hotel tower would include 238 rooms, international cuisine, and first-rate service, operating as a five-star establishment. Both corporate visitors to the World Trade Complex and tourists on home port cruises will be accommodated by the hotel. Up to 1,000 persons can have corporate events in one of the several conference halls and banquet rooms located on the lower levels. Simultaneously, plenty of room for parking is supplied in accordance with the current regulations, taking into account any possible overflow needs.

2.3 PROJECT VIABILITY

Project viability is worked out in such a manner that the three Revenue Centers have to be viable independently over a period of 15 years. Revenue Proposition of these major units worked out on an average of 15 years can be summarized as shown in **Fig.4.**

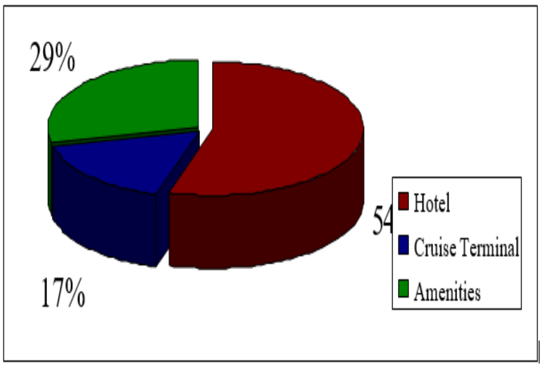


Fig.4. Revenue Preposition

2.4. SUMMARY

From the feasibility study, it is evident that the project is viable and the Government approved the sanctioning of the project. **Fig.5** shows the images of cruise terminal during construction and the completed image of the Phase I.

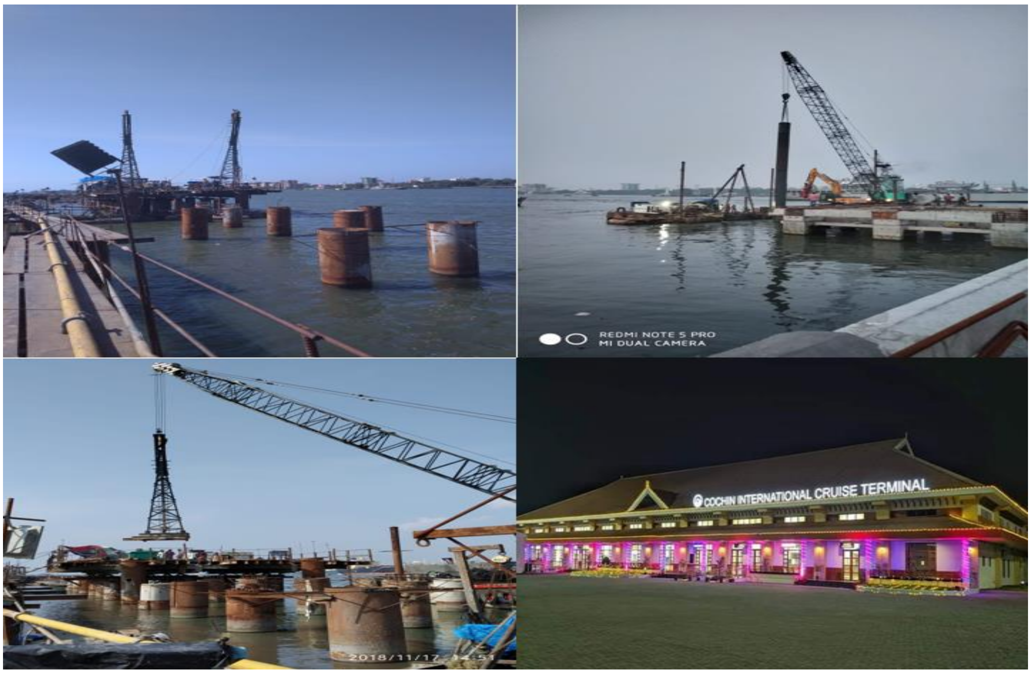


Fig.5 Images of cruise terminal Phase I during construction and after completion

Phase I of the project is completed and Shri Narendra Modi, Hon'ble Prime Minister inaugurated Sagarika International Cruise Terminal Phase-I at Cochin Port on 14.02.2021.

3. GROUND IMPROVEMENT FOR OIL TANK FARM FOUNDATION AT WILLINGDON ISLAND, COCHIN – CASE STUDY

3.1. ABOUT THE PROJECT

Sarthak Industries was planning to install Oil tank farm at Willingdon Island with a capacity of 40000 Tonnes of storage facility in about 8 tanks for storage of Edible and Petroleum oil to be imported from Middle east countries & Malaysia. Proposed project area comprise of recently dredged fill, below which very soft marine clay followed by medium dense sand is present. For the foundation of oil tanks, the initial design was to adopt DMC pile foundation terminating in dense sand strata at a depth of 45m. Since the cost component working out for foundation of the steel storage tanks within the leased area was much higher, it was decided to go for ground improvement technologies.

Due to extensive consolidation, soft marine clays can pose a number of construction-related issues and frequently compromise the performance and safety of the structures they support. In these cases, deep pile foundations are the best option, but they are also quite expensive and time-consuming. Therefore, in soft marine clays, ground improvement techniques are employed as an affordable solution to the issue. Solutions for soft clay deposits include band drains, stone columns, sand pilings, lime columns, and other ground improvement techniques. It is observed that the strength and settling qualities of soft clay deposits can be significantly enhanced by columnar intrusion used as a vertical drain through displacement techniques. The performance of a model bed without columnar intrusion, a single driven columnar intrusion, and a double driven columnar intrusion via displacement method are compared.

3.2. GROUND IMPROVEMENT

For some of the Initial tanks, rammed stone column in bored holes was adopted. But since the soil was extremely weak the lateral movement of material was quite high and there was about 250 to 300% of additional material consumption which was not working out to be economical. Ground improvement techniques using multiple driven fibre reinforced columnar intrusion technology was introduced. It was found that in case of very soft clay, if sand drain or coarse aggregate drain alone is provided, the diameter of the drain gets reduced and at certain times continuity of drain get broken, due to the necking action in the very soft clay. The schematic representation of Multiple driven fiber reinforced columnar intrusion is given in **Fig.6.**

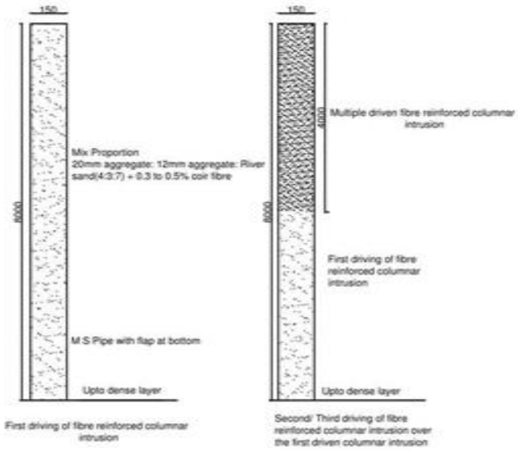


Fig.6. Schematic representation of Multiple driven fiber reinforced columnar intrusion

The clogging of well is also likely to be high in such cases. In order to reduce these effects and to increase the strength and maintain continuity characteristics of the drainage media a hybrid mix comprising of aggregate, sand and fiber reinforcements was proposed. A hybrid blend of aggregate, sand, and fibre reinforcements was suggested as a way to lessen these impacts, strengthen the drainage media, and preserve its continuity features. Sand and coarse aggregate with a size of 20 mm or less were combined with discrete coir fibres to create the hybrid mixture that was suggested. It was decided to go with a vertical drain using a hybrid mix of 20 mm aggregate, 12 mm aggregate, river sand, and coir fibre based on the results of the laboratory testing. In a hybrid mix, 0.3 to 0.5% (50 to 75 mm long) coir fibre was added to a mixture of 20 mm aggregate, 12 mm aggregate, and river sand in a 4:3:7 ratio.

In order to strengthen the ground using the displacement method, a 150 mm diameter hollow M.S. pipe with steel flap plates plugged in the bottom was originally driven to a depth of 8 metres. The hybrid mixture was layered into the pipe, compressed using rods at 1m intervals, and then the pipe was gradually removed. Following the installation of the first columnar intrusion, a second 150 mm diameter hollow M.S. pipe was driven using a displacement method to a depth of 4 m. This allowed the compacted hybrid drainage medium to be displaced into the first produced smear zone. The pipe's bottom was closed with steel flap plates. A third driving was also done at the same location if the soil was determined to be exceptionally weak. There were multiple driven sand columnar intrusions with 450 mm gap between them.

The drainage path is shortened and the smear zone caused by remolding is lessened as a result of the numerous driving processes that inject more hybrid mix with high shear strength into the weak soft clay layer. In theory, the drainage path shrinks from 150 mm to 119 mm in the second drive and then to 96 mm in the third driving, speeding up the consolidation process and strengthening the underlying strata. In real terms, it was seen that only roughly 60% of the material was eaten during the second driving, and only roughly 40% during the third. The injection of high strength material into the weak soil layer increased by 22 to 33% during double driving, resulting in a 44% reduction in the drainage channel. Higher shear strength, superior resistance to lateral deformation, and improved load distribution throughout the depth are all advantages of the hybrid mix.

3.3. CONCLUSION

Due to the double driving process as more hybrid mix which has got high shear strength is getting introduced into the weak soft clay layer the problem of smear zone formed due to remolding get reduces and the drainage path gets shorten. Hybrid mix has got the benefits of higher shear strength, excellent resistance to lateral deformation and better load distribution along the depth compared with sand or aggregate columnar intrusion alone. The benefit of providing the pre-stressed precast elements with R.C.C. beam to form the base mat over the conventional R.C.C. raft slab is that faster construction is possible due to prefabrication, the rigidity of the mat can be controlled. By adopting the new technologies of multiple driven fiber reinforced columnar intrusion, pre-stressed precast flexible mat foundation system and the preloading using hydro testing the installation time of the tank could be reduced to 55 days from 150 days and saving of 32% noted in the foundation cost. The performance of the foundation of oil tank farms over the improved ground is found to be satisfactory. The completed tank farm is shown in **Fig.7.**

Fig.7. Completed tank farm

4.0. SHANGHUMUGHAM BEACH AND ROAD RESTORATION, TRIVANDRUM – CASE STUDY

4.1. ABOUT THE PROJECT

Kerala has a long coast line and it has beautiful beaches. Over the years there has been a decline of the beaches and shore lines due to erosion. Many parts of the Kerala coast are subjected to erosion during the monsoon season. During such times the damages are widespread and some measures are taken to give aid and compensation for the people affected. The interim measures for protection of the coast include use of sand bags and depositing stones on the coast. In many sites along the shore line there were extensive beaches and over the years the beaches were partly or fully lost. When the valuable land, holdings and buildings are threatened, people adopt different protective measures against the waves with little effect. Shanghumugham Beach is located on the western side of Thiruvananthapuram and very near to Trivandrum International

Airport. **Fig.8** shows the location of the Shanghumugham beach and **Fig.9** shows the state of the beach in the past and present.

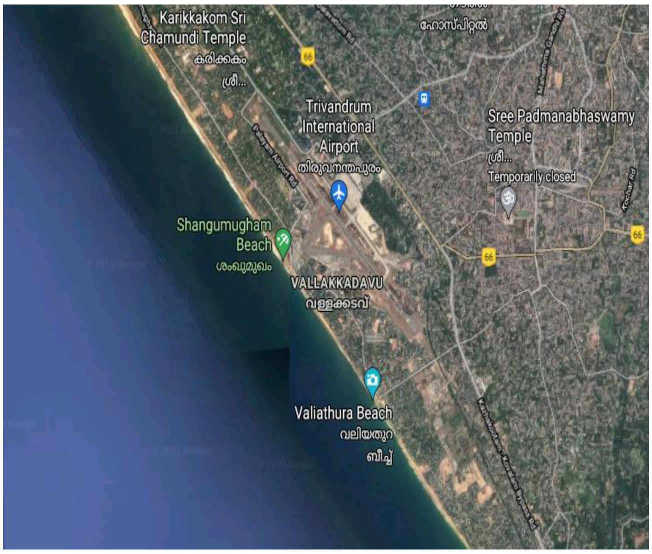


Fig.8: Location of the Shanghumugham beach, Trivandrum

Fig.9: Shanghumugham beach, Trivandrum (Past and Present)

4.2. EROSION OF THE SHANGHUMUGHAM BEACH

The stretch of the Shanghumugham coast is about 4.0km, up to the mouth of Veli estuary. Year after year the condition of the Shanghumugham beach is worsening due to severe sea erosion. The erosion is found to be at the peak during the months of June to August. The retaining wall constructed by Kerala road fund board was damaged and Airport- Shanghumugham road was washed off. The rough sea has almost swallowed the public amenities constructed by the Tourism department. Recent images of beach are shown in **Fig.10.**

Fig.10. Recent (2021) Images of the Shanghumugham beach

Sand up to a depth of 5m was also washed off in the heavy rain and the recurring shore erosion had affected the stability of the protection wall. **Fig. 11.** shows the decline of Shanghumugham beach and **Fig. 12.** shows the damaged road.

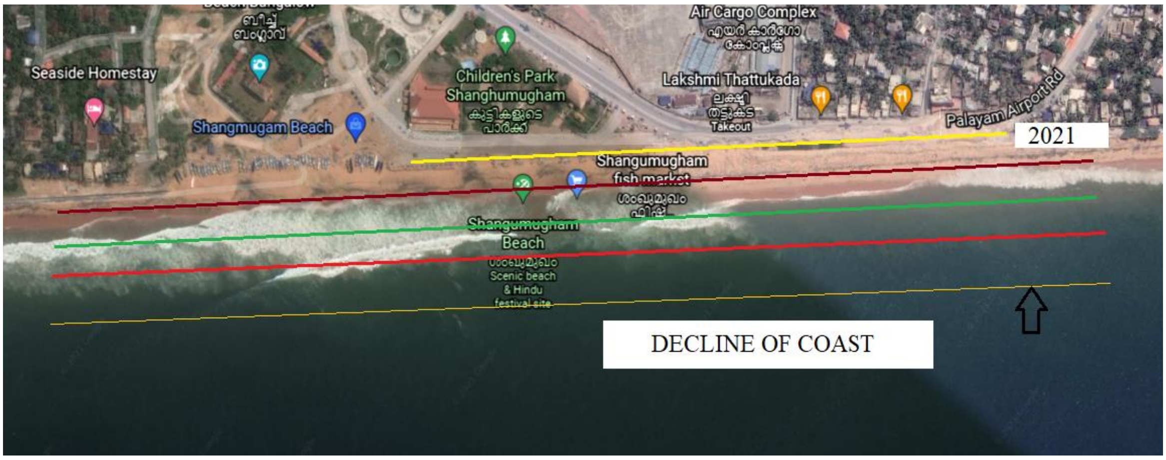


Fig.11. Line showing decline of Shanghumugham beach over years

Fig.12. Damaged road of Shanghumugham beach

There are numerous interruptions to littoral drift caused by manmade structures, dredging and sea level rise due to climate changes. Steepening of near shore are changing the long shore and cross shore sediment dynamics. Along the west coast of India, the open beaches typically erode by 30 to 50 meters during the south west monsoon (June-September). The beaches naturally accrete back to their original state during the early phase of dry season (October to December). It is noted that after the Ockhi Cyclone in 2017, the accretion process is not naturally happening during dry season. This report presents various coastal protection measures suitable for our coastal belts and the proposal to restore Shanghumugham beach for the protection of its coast from erosion due to extreme waves and storm surges. Conceptual layouts of the protection scheme for the coast has been suggested in this report.

4.3. SHANGHUMUGHAM BEACH RESTORATION MEASURES

The measures of protection of eroded road, restoration of 150m coastline and reduce the impact of waves on shore was studied. A diaphragm wall- which is a structural concrete wall constructed in a deep trench excavation as either cast in situ or using precast concrete components is suggested to protect the damaged road along with road alignment and multiple 250m length steel sheet pile/ Concrete Caisson system is proposed to be constructed at 150m into the sea from the diaphragm wall/ or existing shore wall protection which acts like a reef which eventually leads to the nourishment of beach. This nourishment will help to restore the Shanghumugham beach. The toe of the sheet pile/ Concrete Caisson sea walls needs to be protected by rubbles, tetrapod's etc depending upon the wave energy. A distance of 50m is suggested between the adjacent sheet pile system which helps the canoes to approach shore without any damages. Submerged geotubes at 150m into the sea from the sheet pile system is also suggested as a primary barrier to reduce the impact of waves on the shore. A brief idea of the proposal is shown in **Fig. 13 and 14.**

Fig. 13: Tentative plan for coastal protective measures suggested for Shanghumugham beach

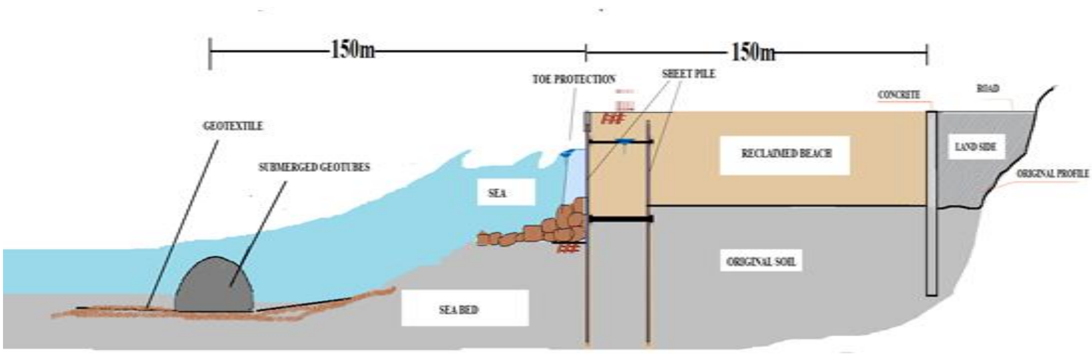


Fig.14. Tentative section for coastal protection

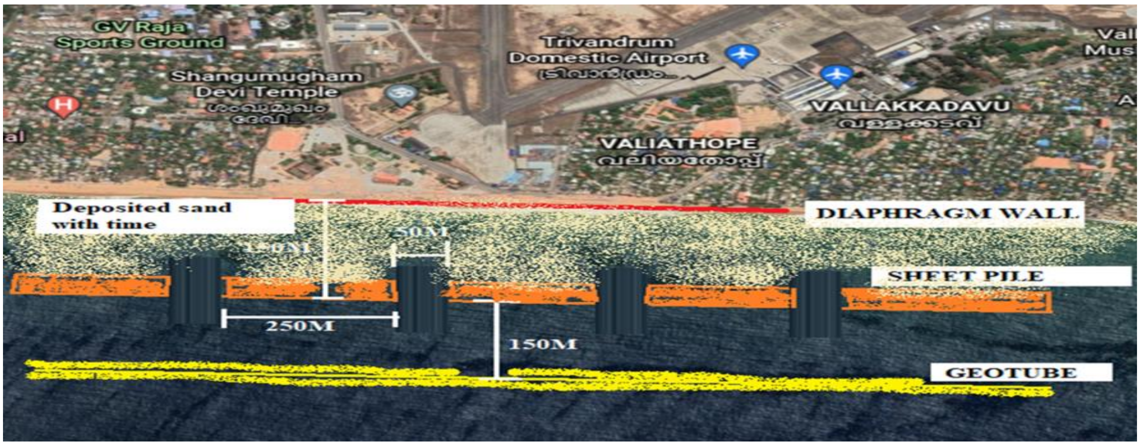


Fig. 15: Expected Shanghumugham beach over years after protective measures

If the breakwater is founded on sand, this can result in severe sand erosion and consequent breakwater instability. To prevent this, a geotextile needs to be installed across the top of the sand foundation prior to the construction of the breakwater. The geotextile acts as a filter preventing the erosion of the sand foundation through the breakwater. The expected outcome of these protective measures is shown in **Fig. 15.**

4.5 CONCLUSION

The coast of Shanghumugham beach is shrinking day by day, as the sea is continually pressing inland by a combination of climate-related and man-made causes.

1. There are many measures available for protection, a combination of diaphragm walls, steel sheet pile and Geotube breakwater would be effective for the restoration of the beach.
2. Diaphragm wall is suggested as a protective measure to restore the damaged road.
3. Sheet pile seawall / Concrete Caissons or artificial reefs at a distance of 150m into the sea from the existing road is suggested to restore eroded Shanghumugham beach.
4. Submerged Geotube breakwater is suggested at a distance of 150m into the sea from the sheet pile towards the sea as a primary barrier to reduce the impact of waves.
5. Conceptual layouts of the protection scheme for the coast has been suggested in this report. The final layout can be arrived at, after detailed engineering study.

5. FEASIBILITY STUDY OF DEVELOPMENT OF TADADI PORT, KARNATAKA - CASE STUDY

5.1. PROJECT DESCRIPTION

The Government of Karnataka has undertaken the development of the minor Port called Tadadi situated in the estuary of the Aghanashini River. The backwaters of the river Aghanashini has got vast water front at the existing Port which is currently being utilized for fishing activities alone. The Government of Karnataka sees a great potential in developing this Port with modern infrastructural facilities. Government of Karnataka decided to take up development of the port at Tadadi on a PPP framework with Karnataka State Industrial & Infrastructure Development Corporation (KSIIDC) as Nodal Agency. KSIIDC the nodal agency has appointed the Consortium consisting of the firms PROINTEC (Spain) and MIR PROJECTS AND CONSULTANTS (Kochi, India) as consultants for preparation of the detailed feasibility study. The consortium entrusted Geostructurals Pvt Ltd, Kochi to carry out the geotechnical and structural design, Topographic survey, Bathymetry & seismic survey, and Geotechnical Sub-surface investigations.

5.2. ENGINEERING SURVEYS & INVESTIGATIONS

The engineering surveys and investigations involves the Topographic survey, Bathymetry & seismic survey, Geotechnical Sub-surface investigations and Hydrodynamic and Coastal Process.

<u>Topographic Survey</u> – All existing features including project boundaries, bund lines along creak were noted during the survey which covered an area of 1723 acres. Digital drawing with 0.25m contours corresponding with the Indian National Grid were prepared as the part of the survey. Stretches involving construction of structures and their approaches were identified from the survey work and Permanent bench marks were established at site. The area was found to be primarily flat terrain. The stability of points was cross checked with the bench mark at Madanagiri Village and was cross verified.

<u>Bathymetry and Seismic survey</u> - Seismic survey was done to obtain continuous profiles of the sub-seabed around the proposed area in order to establish hard rock levels, sub-surface stratigraphy, individual stratigraphic units and their thickness and mark the interfaces using echo-sounding, high resolution shallow seismic profiling and side scan sonar surveys of the area under investigation. The nature of the surficial and sub-surface soils was also found out from the survey which was carried out in depths up to 30m below sea bed, using high-resolution sub-bottom profiler. **Fig 16.** Shows the equipment used for conducting survey.

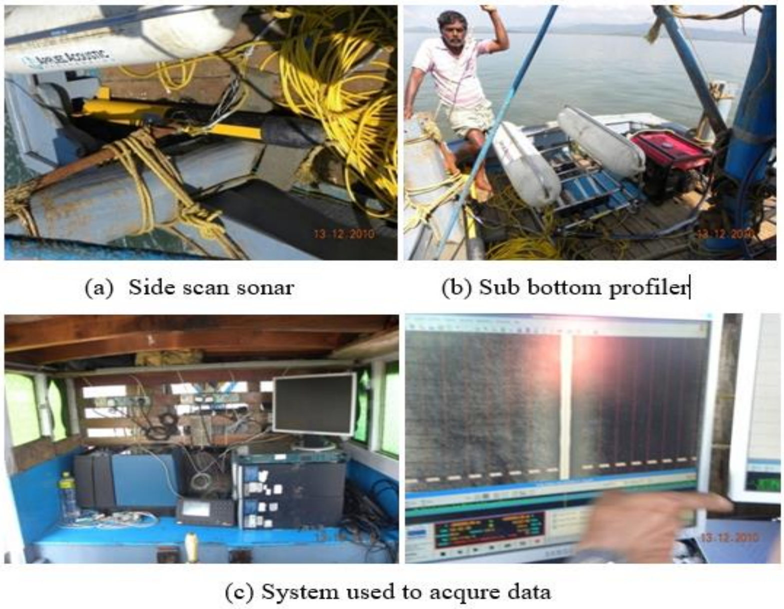


Fig 16. Equipment used for conducting survey

From the Bathymetric and Geophysical surveys, it is noted that the offshore area of the sea bottom is quite regular and flat with gentle slopes from 1:300 to 1:500: the -10 m depth is located approximately 3,000 m from the coastline and the -20 m is located at about 8,000m from coast line. The depth of the sea bed gradually varies from the 5m contour in the north eastern corner to 21m on the south western boundary of the offshore block. The maximum water depth of 21.4m is observed along the south western boundary while the minimum water depth of 3.9m is noted in the north eastern corner. Sea bed features include fine sediments, coarse to very coarse sediments, boulders, cobbles and scar marks depicted in **Fig.17**. A minor rocky patch is exposed on the sea bed in the north east corner of the block. At the river estuary there exist some shallow areas that emerges at the maximum low tides. The bathymetric contours of the inner block reveal a considerably steep slope along the channel of the Tadadi port, oriented northwest- southeast within the Tadadi creek. The sea bed exhibits a very gentle to negligible slope in the areas away from the channel on both sides. The depth of water column varies from negative values to 5m.

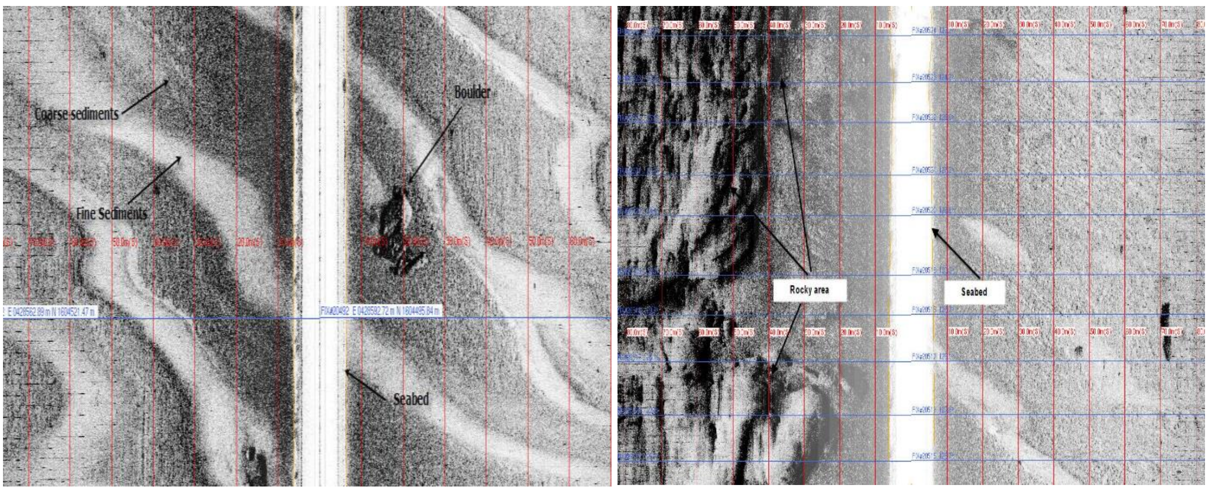


Fig.17. Extract of Side Scan Sonar and record depicting the scar marks on sea bed and rocky area

From the side scan sonar studies no visible rock patches was noted in the area proposed for the navigation channel. Studies of data from Sub bottom profiler also indicate that the hard rock strata is at a greater depth than the draft depth proposed for the channel.

Navigation Chanel:

The average natural draft at the area was found varying from 20.0m to 1.50m, Dredging is to be done up to a depth of 18m in the deep sea area and 16 m in the inner channel, Total quantity to dredged for channel is approximately 27000000 m^3.

Turning Circle & Berth

The average natural draft at the area of turning circle and berth is 1.5m. Dredging is to be done up to a depth of 16m and the total quantity to be dredged ~ 50 Million Cubic Meters.

5.3. GEOTECHNICAL INVESTIGATIONS

The principal objectives of the works were to investigate geotechnical properties and access the ground conditions / properties of soil/rock in subsurface of the site (including land and water) to undertake the preliminary design of foundations for berth structures, retaining structures, dredging, reclamation, accessory foundations, ground improvement techniques etc. 4 marine boreholes were taken in the outer block of the sea to understand the nature of soil required to be dredged to form the navigation channel and to work out the tentative quantity. 5 marine boreholes were taken in the inner block of the sea in order to work out the dredging quantity required in the turning basins and to work out the type of foundation required for the berth structures and 10 boreholes was taken in the back up areas of the berth in order to obtain representative soil profiles and to work out the design of foundation for various associated port structures proposed and to work out the ground improvements required for the storage facilities, road and rail systems.

From the study of boreholes, it was noted that the top 1.50m comprise of stiff clay followed by very soft clay having S.P.T value of 0 extending up to depth of 15.00m. Below this stiff clay was noted extending up to depth of 18.00m followed by disintegrated rock/hard rock extending up to depth of termination of borehole at 21.00m. Location of boreholes is shown in **Fig.18.a** and geotechnical investigation is shown in **Fig.18. b.**

Fig.18.a Location of boreholes, **b.** Geotechnical investigation

5.4. HYDRODYNAMIC AND COASTAL PROCESSES

A detailed Hydrodynamic studies including Historical evolution of the estuary (since 1854), analysis of the estuary processes, study of the sediment active zone, long-term estuary stability, littoral transport and need of maintenance dredging.

Historical evolution of the estuary (since 1854): The current estuary situation has been compared with the lay-out of the second half of the XIX Century. The pools and basin location and the sand barrier's overall length are fairly similar. The steep and rocky coast sited to North to the river mouth prevents the bar's progress and therefore the sand barrier position is just the same as in 1854 and the same it can be said for the coastline of the Northern and Southern beaches. Depths are very similar in both configurations. The main conclusion of the historical review is that the estuary is in equilibrium with no problems of sedimentation with the current physical system (without human interference). An Escoffier's analysis was carried out, and the result obtained was that the estuary is in equilibrium or almost in equilibrium as shown in **Fig.19.** The potential longshore transport has been calculated with three empirical models (CERC, Kamphuis and Van Rijn), and it has been concluded that the longshore transport is mainly from North towards South.

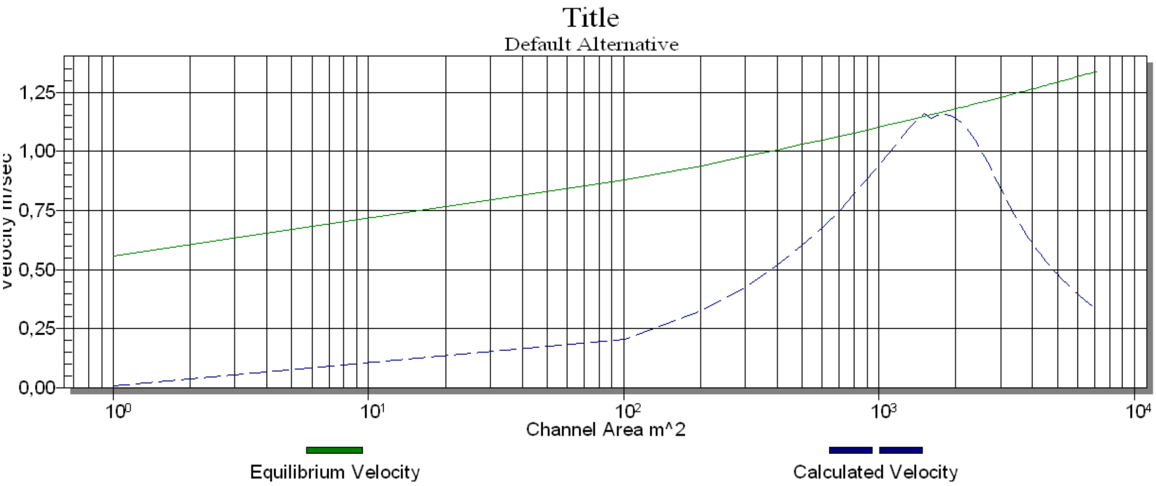


Fig.19. Escoffier's analysis result

Estuary Process: Tidal inlets (Aghanashini river estuary) are characterized by large sand bodies that are deposited and shaped by tidal and wave currents. A simplified morphological model of a natural ebb-tidal delta can include several components such as a main channel scoured by the ebb jets, linear bars flanking the main channel as a result of the wave –tidal current interaction, Terminal lobe located seaward at the end of the ebb channel, Swash platforms - sand sheets located between the main ebb channel and the adjacent barrier islands, Swash bars that dorm and migrate across the swash platforms because of currents (the swash) generated by breaking waves, Marginal flood channels flank both up drift and drown drift barriers.

Sediment Active Zone: In order to know the transport across and along the surf zone it is also necessary to demarcate the zones of active sediment movement regions, transport rates. The depth of closure for a given or characteristic time interval is the depth seaward of which there is neither significant change in the bottom elevation nor significant net sediment exchange between the near shore and the offshore areas. Thus, the closure depth separates the area where both longshore and onshore-offshore sediment transport take place from a non-active offshore area and therefore it is an important parameter in coastal engineering projects. The active depth for a given or characteristic time interval is the depth seaward of which there does not exist longshore sediment transport and only onshore-offshore sediment transport takes place and where the change in the bottom elevation are less important. Thus, the active depth separates the active near shore area from a less active offshore area and therefore it is also an important parameter. These depths can be determined empirically by examining seafloor elevation changes measured by repetitive surveys and identifying the depth at which changes became negligible.

Littoral Transport: To determine longshore sediment transport three empirical models such as CERC, Kamphuis, Van Rijn were used. The results obtained with Van Rijn formula match better with the transport modelling results and are those closest to the model results. Therefore, this formula seems to be the most reliable. The net longshore transport to the south of the Aghanashini river is 415,000 m^3/year. It is important to point out that the calculated transport rates correspond to potential (not real) values, because the real littoral drift depends on the sediment availability. Numerical modelling of Littoral transport includes wind characterization, waves characterization at deep waters, waves numerical propagation, Currents numerical modelling (due to breaking waves & river discharges & tides), sediment transport numerical modelling (due to waves & currents).

Tides Characterization:

According to the information from the Navigation Chart number 216 the main tide levels AT Tadadi are as follows:

MHHW (Mean Highest High Water): + 1.8 m above CD (Chart Datum)
MLHW (Mean Lowest High Water): + 1.7 m above CD
MSL (Mean Sea Level): + 1.2 m above CD
MHLW (Mean Highest Low Water): + 1.0 m above CD
MLLW (Mean Lowest Low Water): + 0.4 m above CD
This agrees with the information from the Karwar Port tide gauge.

Wind Characterization: The wind data used are the hindcast wind provided by the company BUOYWEATHER at a point located in front of Tadadi. The Annual Average Cumulative Probability Functions were also obtained from the data. It is also possible to distinguish four seasons as shown in **Fig.20.**

i) April and May

ii) SW-monsoon

iii) October and November

iv) NE-monsoon season

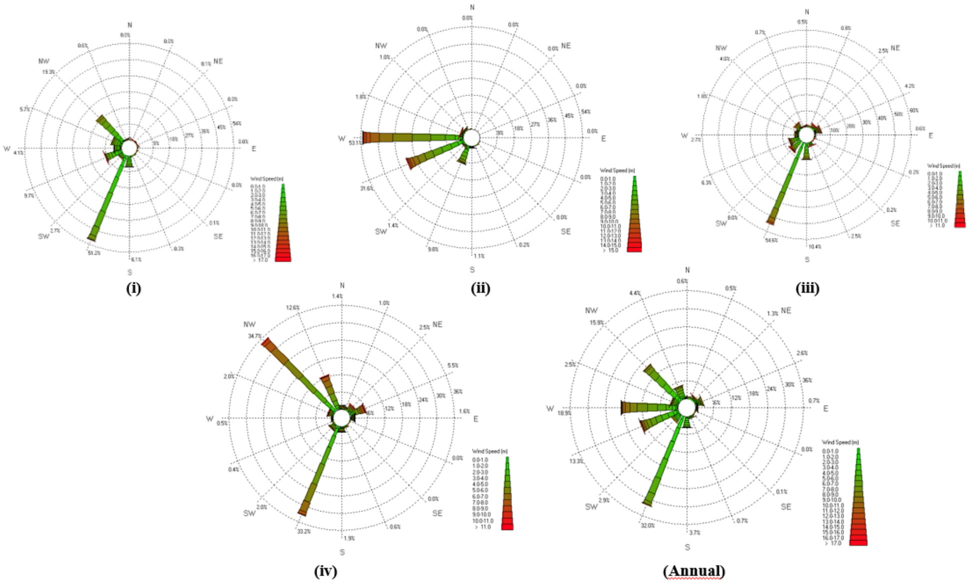


Fig.20. Wind Characterization for all seasons

<u>Waves propagation numerical study:</u> The numerical study of the waves propagation from deep water to the site has been performed with the program MIKE21-PMS. Different regional grids were used for each direction and finally a common local grid was implemented.

<u>Currents numerical study:</u> The numerical study of the currents has been performed with the program MIKE21-HD shown in **Fig. 21.** Three origins such as waves breaking, tides, river discharges for the currents were considered. The main conclusions of the current modelling can be summarized as follows:

- Near the river mouth, the longshore current direction is mainly towards the South. Currents under the action of SSW waves are the main exception.
- Longshore currents directions is always towards north at the north half of the headland located in the river Aghanashini mouth. As a result of flood and ebb tides, currents flow is towards upstream and downstream respectively.

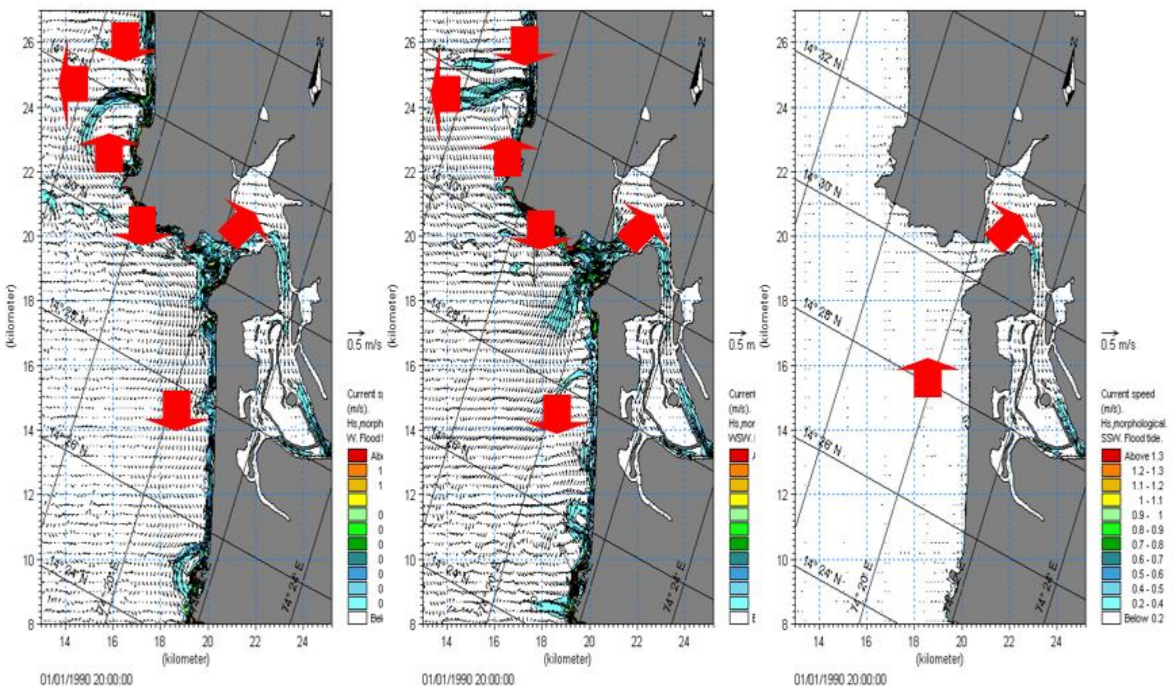


Fig.21. Currents pattern due to mean tide (flood tide) + mean river flow + W waves (left), WSW waves (center) and SSW waves (right)

Sediment Transport Numerical Modelling: The numerical study of the sediment transport has been performed with the program MIKE21-ST, considering both waves and currents. Transport sediment pattern due to mean tide (flood tide) + mean river flow + WSW waves (up & left), W waves (up & right), SSW waves (down & left) and NW waves (down & right). The net sediment transport rates have been calculated at several cross profiles and is depicted in **Fig.22.**

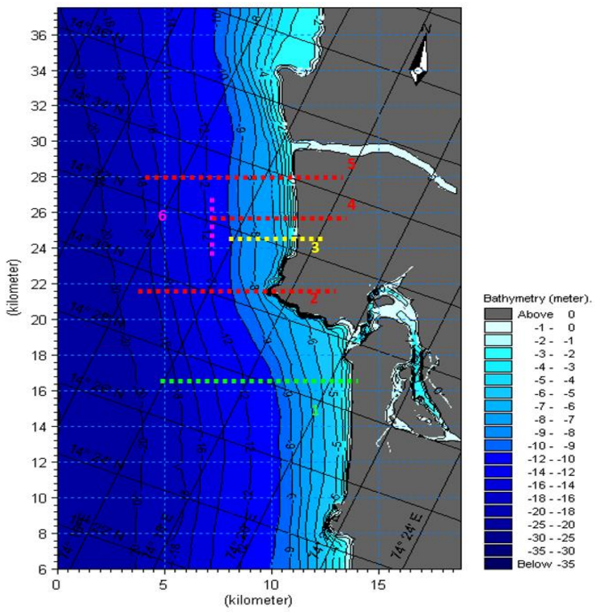


Fig. 22. The net sediment transport rates at several cross profiles

5.5 NEED OF MAINTENANCE DREDGINGS

To keep depths in the future Tadadi Port navigation channel trench at the active area (i.e. where the longshore sediment transport is significant) due to the action of waves and currents. From the study, it is noted that it will be necessary dredging at least 3,35,000 m3/year (coming from river) + 1,85,000 m3/year (coming from the South) + 95,000 m3/year (coming from the North) = 6,15,000 m3 per year as shown in **Fig.23**. To keep depths in the future Tadadi Port navigation channel trench at the active area (i.e. where the longshore sediment transport is significant) due to the action of waves and currents. To define the sediment accumulation out of the active area, an analysis of the maintenance dredging at the Ports of Mormugao, New Mangalore and Cochin was performed, concluding with a quite constant sediment accumulation ratio of 2.65 m3/m2/year = 2.65 m/year.

For the outer channel projected at Tadadi (16,83,000 m2) it means 44,75,000 m3/year.

Therefore, the total maintenance dredging for Tadadi Port will be 50,90,000 m3/year.

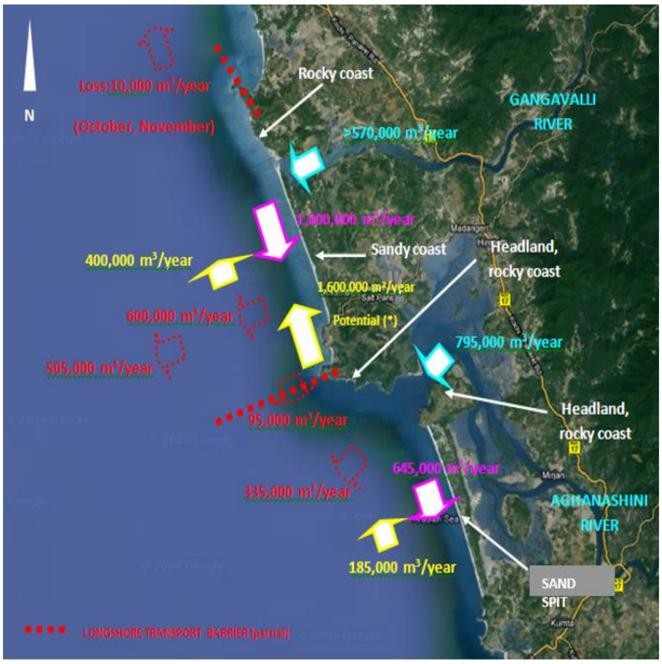


Fig.23 Quantification of maintenance dredging

5.6. MASTER PLAN OF THE PORT

Master plan of the project include navigation channel, turning circle, currents/wave protection: need of a breakwater, berths, On shore layout, handling equipment.

Navigation Chanel: The Aghanashini river has at this area a direction from SSE to NNW and suddenly makes a closed 135º turn so that its mouth has an alignment from NNE to SSW. At the northern part of this turn an estuary has been developed where two creeks can be found. The existing boundaries of these creeks define the northern limit of the port basin. At this area the navigation channel, turning circles and berthing area is proposed to be developed. In order to define the geometric dimensions of the navigation channel, three different methods were analysed. The first method is based on Indian Standard 4651 ("Code of practice for planning and design of ports and harbours. Part V: Layout and functional requirements") whereas the second and third methods are based on the Report of the PIANC-IAPH Working Group II-30 ("Approach channels. A guide for design"). Based upon the studies by the different methods it is decided to provide one – lane navigational channel with the following dimensions.

Outer Channel:

Width = 180 m, Depth = (-)18 mCD

Inner Channel (straight section):

Width = 200 m, Depth = (-)16 mCD

[Dredging slopes: 1V: 5H]

Width = 217.50 m, Depth = (-)16 mCD, Turning radius = 2,800 m

Turning Circle: According to the Indian Standard 4651, part V, in the case of vessels that turn by free interplay of the propeller and rudder assisted by tugs the minimum diameter of the turning basin or circle should be 1.70 to 2.00 times the length of the largest vessel to be turned (1.70 for protected locations and 2.0 for exposed locations). Where no tug assistance is available the diameter of the turning circle may be as large as 4 times the length of the design ship. In the case of the Tadadi Port due to the existing geometrical limitations and for minimizing the volume of material to be dredged it is recommended to reduce the turning circle dimensions as much as possible. Therefore, the assistance of tugs has been considered and a diameter of 2 times the length of the largest ship has been applied. From the table mentioning the ship sizes it can be checked that the largest vessel available to enter at the new port is a container vessel 290 m long. Thus, a turning circle with a diameter of 580 m has been considered. A second turning circle has been included to allow the turning of the vessels that berth at the second wharf alignment. At this quay the largest ships capable to berth are 40,000 DWT general cargo vessels, with a length over all of 210 m. Therefore, the diameter of this second turning circle is reduced to 420 m. For both turning circles the depth will be -16 m (below Zero), this is, the same depth that at the inner channel. **Fig. 24**. Shows the layout of turning channel.

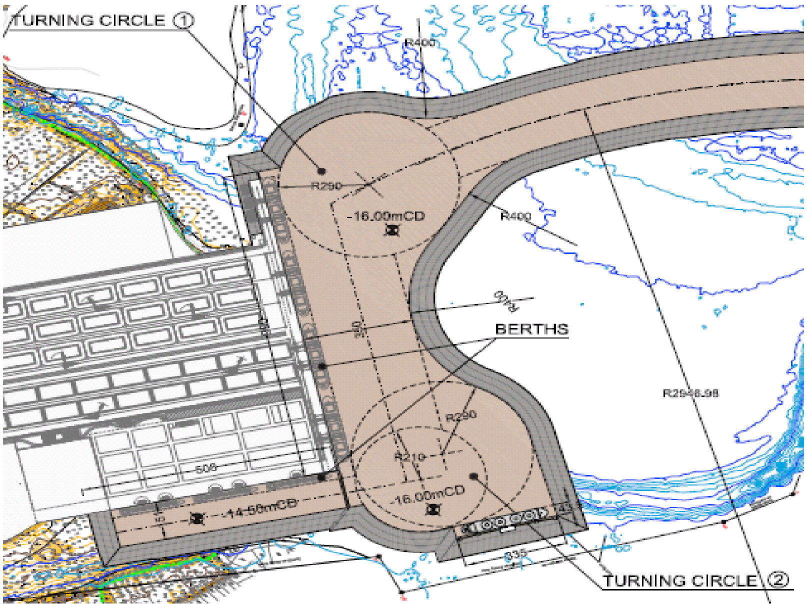


Fig.24. Layout of turning channel

Currents/Wave Protection: Need of a Breakwater Analysis: The existing conditions at the port entrance with limited depths produce an important protection effect as the wave breaking is induced, and therefore the wave heights are reduced drastically. The dredging of the new navigation channel will increase considerably the depth at the estuary mouth, which could allow the waves penetration into the estuary towards the new port. An agitation numerical model study has been performed to obtain the local values of Hs at the future berths. In the case a (piled berth) the values are lower than the maximum accepted (40 hours/year). This allows us to conclude that the natural protection of the Tadadi port is enough and additional breakwater is not necessary for this purpose. In the case b (vertical quay) the values for the southern berths are higher than the maximum accepted (40 hours/year), so that the natural protection of the Tadadi port is not enough and additional breakwater should be constructed. Therefore, the piled berth technology is recommended for the wharves.

Berths: The different types of structural systems for the berth structure studied for arriving at the most optimum arrangement are listed below. The main categories are, solid berth structure (i.e. gravity wall & sheet pile wall) wherein the fill extends right out to the berth front where a vertical front wall is constructed to resist the horizontal load from the fill. Open berth structure (a piled berth) wherein a load bearing slab is constructed supported on beams and piles resting with adequate anchorage on hard rock. Of the two, open berth structure is adopted for this port considering the relative shallow depth at which hard rock is encountered. "L" shaped berth which can accommodate 5 no of vessels (3 number of 100000DWT ore/ coal vessels and 2 no of 40000 DWT general cargo vessels) is proposed as shown in **Fig.25**. The width of the berth is 50m. 990m length is provided for berthing 3 no of 100000 DWT and 550m length is provided for berthing 2 number of 40000DWT vessel. However, considering the future expansion for the general cargo, an additional two berths are proposed as an extension of the multipurpose terminal on the North western side. Even though the envisaged size of vessels is of 40000 DWT for the multipurpose terminal, considering the future expansion all the berth structures has been designed to accommodate 100000 DWT vessels. Based on the soil investigation the foundation recommended for berth structures is bored cast in situ piles with adequate anchorage in to the hard rock. Load test are essential for accurate determination of pile capacity. From the study, 3 or 4 tugs will be necessary for the approach, berthing and un-berthing vessel operations.

Fig.25. Layout of the proposed berths

5.7. DEVELOPMENT IN PHASES

For the iron ore terminal, it has been considered that the Phase 1 has to include both yard stock yards, and all the berths and the necessary handling equipments. For the coal terminal it has been considered that the Phase 1 (shown in **Fig. 26**) has to include 2 of the 3 planned stock yards, plus all the berths and the necessary handling equipments. For the multipurpose terminal it has been considered that the Phase 1 has to include 2/3 rd parts of the berth length and the paved back area. The berth nº 5 will correspond to the general cargo and container traffic and the berth nº 6 to the steel products.

Phase 1-A **Phase 1-B** **Phase 2**

Fig.26. Development Phases

It must be mentioned that this Phase 1 can be constructed into two sub-phases, as the iron ore terminal should start its operation in 2015-16 (Phase 1-A), whereas the coal and the multipurpose terminal does not start its operation until 2020-21 (Phase 1-B). The Phase 2 should start its operation in 2031-32, when two third of the coal and steel traffic will have been completed.

5.8. PRELIMINARY DESIGNS

Designs for each berthing structures were obtained using various load combinations. The berth is designed for the worst effect of the individual or any combination of the load. A uniform vertical loading of 60 KN/m2. Further the berth structure is designed for operation of cranes and for Class AA and 70 R Loadings, Crane loading, Railway loading, Impact loading and special load like pipe line, conveyor belt and surcharge load. The truck loading that are likely to be handled on the berth considered for analysis in accordance with IS 4651 (Part III) : 1974 is IRC 70R wheel loading. The dead load comprises of the weight of all the beams and the slab of the deck.

All forces acting horizontally or along the line facing water is considered to be the horizontal load. All horizontal loads are calculated as per IS: 4651 part III code of practice for the planning and design of port and harbors. The main horizontal loads considered for the design is Berthing Force/ Berthing Energy, Mooring load, Bollard Pull, Wave force, Current force, Wind force and Seismic force.

5.9. CONCLUSION

Adequate land is available for the project. Major connectivity for the port are Konkan Railway at the edge of the land, NH 63 & 206 connected to the area, Hubli – Ankola Railway line and possess minimum Social Impact. From the study, the project is technically feasible and is waiting for the sanction from the authority as it is a green port project, there is a delay.

6. DESIGN OF RO-RO FACILITY AT HAZIRA, GUJARAT – CASE STUDY

6.1 ABOUT THE PROJECT

Deendayal Port Trust has entrusted the ‘Construction and Maintenance of Ro-Pax facility at Hazira, Gujarat for Gujarat Marine Board to BMS Projects - Marymatha JV, Surat. The EPC appointed M/s. Geo Structurals Pvt Ltd, Kochi, Kerala as their Structural Engineering Consultant for providing Structural Engineering & Design Services as per the Scope of Services for the proposed project.

Hazira is commercial green field port located on the eastern side of Gujarat on the shores of the Gulf of Khambhat. The present Ro-Pax facility is aimed at bridging Surat to Bhavnagar. The travel distance which is at present close to 400kms which takes a travel time of more than 12 hours will be cut short to 4 hours as shown in **Fig. 27**. The project once developed will be a big step towards the vision of mobilizing the waterways and incorporating them with the economic development of India.

Fig.27. Travel distance from Surat to Bhavnagar

6.2 CHALLENGES IN THE PROJECT

Tidal variation of about 7m is noted in the project area and the biggest challenge is to have a floating pontoon connected with link span bridge for the smooth movements of the vehicles from approach platform/ turning platform to the vessel in the berthing structure. Tidal variation in a day in the project site is shown in **Fig. 28.**

Fig.28. Tidal variation in Hazira Port area

6.3 SUB SOIL INVESTIGATION

Subsoil investigation was carried out at site in order to understand the properties of soil beneath the proposed structure and for recommendation of the foundation for the port facility. 5 number of bore hole were taken in the site up to a depth of 50m from the ground level. Ground water was encountered up to a depth of 3 to 5m below the ground level during the time of investigation. From the study of boreholes, it was noted that in areas of BH-1, predominantly top 13.50 m of soil strata are of black medium dense to medium dense fine to coarse sand followed by yellow hard plastic silty clay strata. Strata thereafter are black very dense medium to coarse sand. In BH-2, top 10.50 m of soil strata are of black medium dense to dense fine to coarse sand followed by black dense gravel strata. Strata thereafter are yellow hard plastic silty clay strata. Strata thereafter are black dense gravel followed by black dense medium to coarse sand strata. In BH-4 & 5 the top 10. 50 m soil strata are of black loose to medium dense sand followed by black medium to coarse sand strata. For Jetty structure (BH-1 & 2), pile foundation is recommended.

6.4. MODELLING OF THE STRUCTURE AND LOAD COMBINATION

Fig.29. 3D rendered view of approach way, turning platform and berthing platform

The berthing structures were modelled using STAAD Pro as per relevant IS Codes and the approach way, turning platform and berthing platform are depicted in the **Fig.29** and the various load combinations used for the modelling are shown in **Fig.30. Fig.31.** shows the Layout and details of piles and upper deck of berthing jetty. Layout of Approach and Turning Platform and details of link span erection is shown in **Fig.32.**

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PROJECT: *CONSTRUCTION & MAINTENANCE OF RO-PAX FACILITY AT HAZIRA, GUJARAT*	**PAGE** 16 **OF** 27
SUBJECT: *DESIGN BASIS REPORT – OFF-SHORE STRUCTURES*	**REVISION:** R2

Load combinations considered for design as per *IS 4651-(Part IV)-2014, Table-1* are:

	Limit state of Serviceability
1.	DL + LL + WL+/- WLX
2.	DL + LL + WL+/- WLZ
3.	1 DL+ 0.5 LL + 1 TL
4.	1 DL+ 0.5 LL -1 TL
5.	1DL + 1LL+ WL+/-EQX
6.	1DL + 1LL + 1WL +/- EQZ
	Limit state of Collapse
7.	1.5 DL + 1.5 LL + 1.2 WL +/- 1 WLX
8.	1.5 DL + 1.5 LL + 1.2 WL +/- 1 WLZ
9.	1.5 DL+ 1.5 LL
10.	1.2 DL+ 1.2 LL +/- 1.2 EQX
11.	1.2 DL+ 1.2 LL +/- 1.2 EQZ
12.	1.5 DL +/- 1.5 EQX
13.	1.5 DL +/- 1.5 EQZ
14.	1.2 DL + 1.2 LL + 1WL
15.	1.2 DL+ 1.2 LL+ 1 WL+/- 1.2 EQX
16.	1.2 DL+ 1.2 LL+ 1 WL+/- 1.2 EQZ
17.	1.2 DL+ 1.2 LL +1 WL +/- 1.2 WLX
18.	1.2 DL+ 1.2 LL +1 WL +/- 1.2 WLZ
19.	0.9 DL + 0.9 LL + 1 WL
20.	0.9 DL + 0.9 LL +1 WL +/- 1.5 WLX
21.	0.9 DL + 0.9 LL +1 WL +/- 1.5 WLZ
22.	0.9 DL + 0.9 LL + 1 WL+/- 1.5 EQX
23.	0.9 DL + 0.9 LL + 1 WL +/- 1.5 EQZ

Fig.30. Load combinations used for the modelling

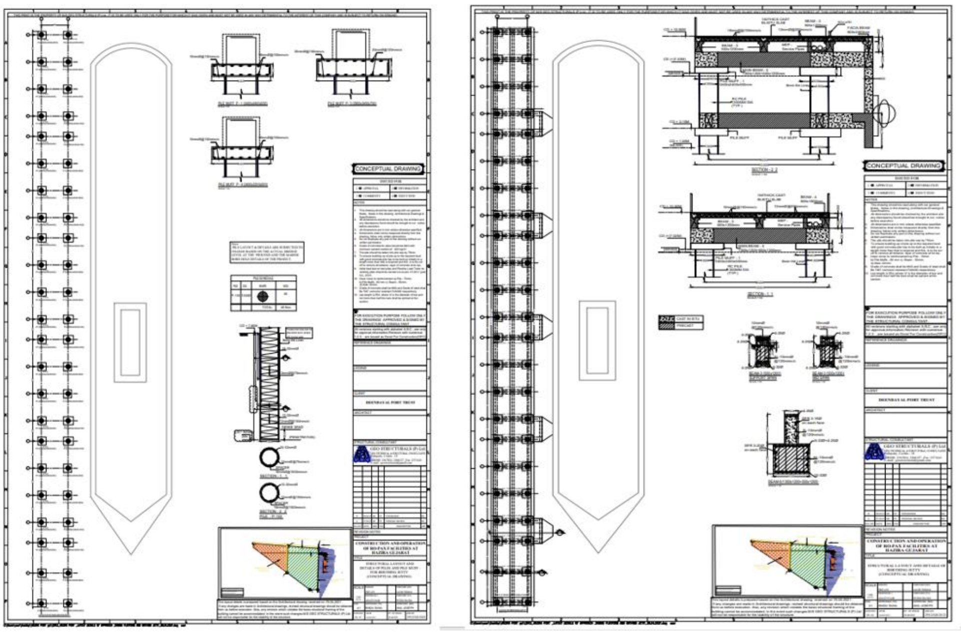

Fig.31. Layout and details of piles and upper deck of berthing jetty

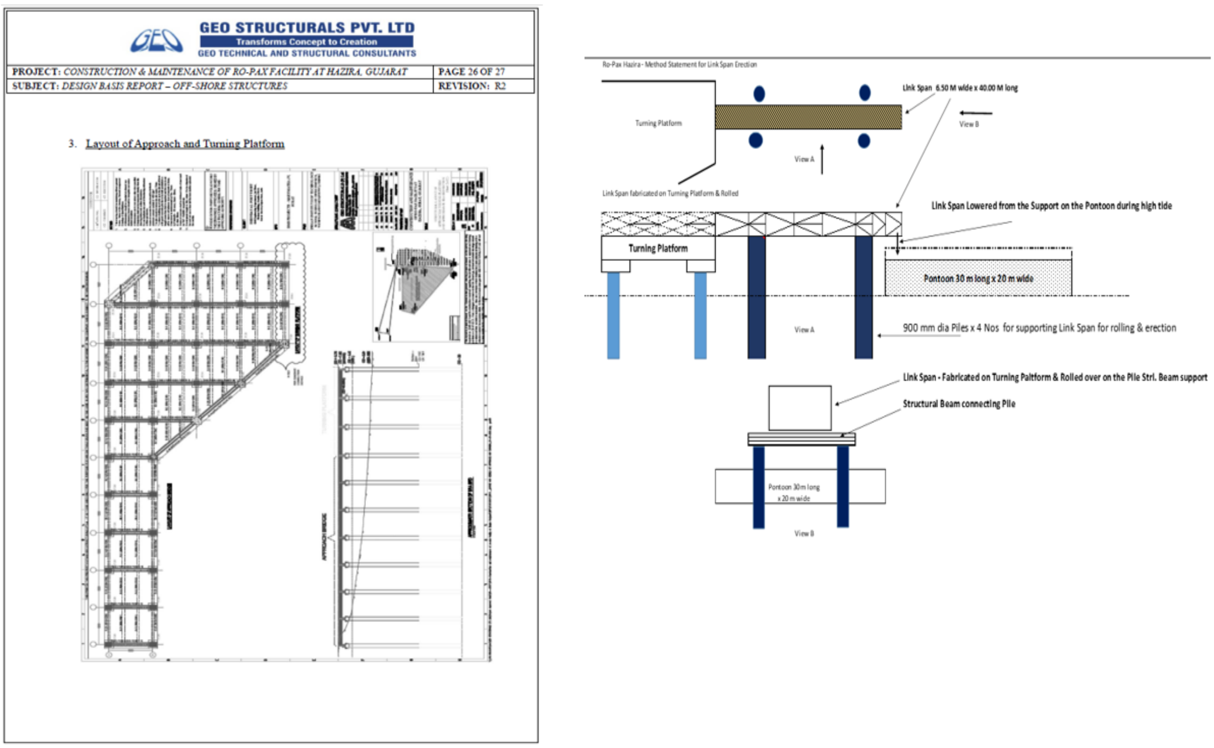

Fig.32. Layout of Approach and Turning Platform and details of link span erection

6.5. CONSTRUCTION PHASES

Different stages of construction of the Facility is shown in **Fig.33 .** Completed structure is shown in **Fig.34.**

Fig.33. Stages of construction of RO-RO Facility at Hazira

Fig.34. Completed images of RO-RO Facility at Hazira

7 CONCLUSION

In nutshell, the field of near and offshore geotechnical engineering is a dynamic and vital discipline that addresses the unique challenges posed by coastal and marine environments.

Diverse Challenges: Near and offshore environments present a wide array of challenges, from varying seabed conditions to dynamic forces, geohazards, and environmental considerations.

Safety and Sustainability: The primary goal of offshore geotechnical engineering is to ensure the safety and sustainability of projects and involves careful planning, innovative solutions, and strict adherence to regulatory and environmental standards.

Economic Significance: Coastal and marine areas are economically significant, supporting trade, tourism, and resource exploration. Effective geotechnical engineering is essential to harness the potential of these regions while mitigating risks.

Environmental Protection, Innovation and Adaptation: Balancing economic development with environmental protection is a paramount concern. Sustainable practices and environmental impact assessments are integral to responsible development. Engineers and researchers continually strive to develop innovative solutions and adapt to changing conditions. These efforts are essential for the successful execution of projects in dynamic marine environments.

In summary, near and offshore geotechnical engineering is a critical field that underpins the safety, sustainability, and success of projects in coastal and marine regions. It requires a deep understanding of the unique challenges these environments pose and the ability to develop creative solutions that safeguard both human activities and the natural world. As we continue to explore, develop, and protect our coastal and marine areas, the role of geotechnical engineering remains pivotal in shaping a sustainable and resilient future.


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"A CASE STUDY ON POST-STONE COLUMN CONSTRUCTION MARINE SEDIMENT IMPACT, TIBAR BAY DEEPWATER PORT"

by
Dr. Jiaer Wu
R&D Manager and Lead Geotechnical Engineer,
CHEC (USA) Ltd.

Jerry is the Lead Geotechnical Engineer and R&D manager at CHEC USA Inc., a wholly-owned subsidiary of China Harbour Engineering Company. He is a licensed Professional Engineer and Geotechnical Engineer in California, with more than 20 years of research and engineering experience.

Jerry received a B. Eng and a B. Econ from Tsinghua University in Beijing, an M.S.C.E. from the University of Kentucky, and a Ph.D. from the University of California at Berkeley. He conducted his doctoral research on the post-liquefaction deformation behaviour of sands under the advisement of Professor Raymond Seed.

After graduation, Jerry joined the legacy geotechnical firm Woodward-Clyde Consultants, which later became URS and AECOM. During his tenure at URS, he worked on numerous transportation and water resources projects including San Francisco International Airport Seismic Assessment and Panama Canal Expansion, and became a certified Project Manager. He has been with CHEC since the establishment of CHEC USA in 2013. In his current role, he leads the geotechnical practice of CHEC USA and provides design management, technical consultation, and support for infrastructure projects from around the world.

Jerry is well experienced in subsurface soil and site characterization, foundation design, seismic hazard assessment, soil-structure interaction analysis, and ground improvement. He has particularly strong expertise in soil liquefaction susceptibility evaluation, advanced laboratory testing techniques, seismic analysis of slopes, earth structures, maritime facilities, constructability, and design optimization (value engineering).

He has an extensive publication history including more than 40 technical papers and reports. He has papers featured in the official ASCE Civil Engineering Magazine September 2010 issue and won the Outstanding Paper Award at the 2014 USSD annual conference. He is passionate about geotechnical engineering and actively engages in professional activities. In addition to being a member of ASCE, DFI, USSD, and GEER, he was the past President of the Overseas Chinese Civil Engineering Association (OCCSEA) which has more than 100 active members in Northern California. He welcomes connection at linkedin.com/in/jerry-wu-6a11a99.

A Case Study on Post-Stone Column Construction Marine Sediment Impact Tibar Bay Deepwater Port

海洋沉积物对碎石桩施工影响的分析
东帝汶帝巴湾新集装箱码头项目

SLGS Annual Conference 2023

October 23rd, 2023

CHEC USA (Inc.)

Presenter Bios

JIAER (JERRY) WU, PhD, PE, GE
R&D Manager, Lead Geotechnical Engineer

Jerry is a California-based geotechnical engineer with over 20 years of academia and industrial experience. He received his PhD degree from UC Berkeley under Professor Ray Seed for his research on post-liquefaction deformation behavior of sands.

He started his consulting career at Woodward-Clyde Consultant (URS) in Oakland, California where he contributed to multiple large scale infrastructure projects in the US and internationally. He is well experienced in geotechnical investigations, analysis and project management.

Since 2013 he leads the geotechnical practice of CHEC USA, a subsidiary of China Harbour, and is responsible for overseeing and supporting the design and construction of numerous projects from around the world.

Presentation Outline

- ❑ Project introduction 项目简介
- ❑ Recap of marine sediment issue 对护岸沉积问题的简要回顾
- ❑ Field and laboratory investigations 对护岸沉积物的勘察和试验
- ❑ Assessment of sediment properties and impact 对沉积物性质和影响的分析
- ❑ Final assessment and lesson learned 最终结论和经验总结
- ❑ Acknowledgement 鸣谢

1. Project Introduction 项目简介

Tibar Bay Port project is by-far the most ambitious infrastructure project of East Timor (Timor-Leste), a young nation that won independent in 2002. The port, located just 12km from the capital Dili, is a major $490 million public-private partnership (PPP) greenfield project that aims to become a regional transshipment hub with up to 1 million containers throughput annual capacity. Conceived in 2012, the PPP project initiated in 2016 when the government reached a 30-yr concession agreement with French company Bollore Group. China Harbour Engineering Company (CHEC) was later awarded the contract to build the new port. The construction started in July 2019 and after overcoming numerous difficulties including Covid-19 caused delays, the port was officially inaugurated in November 2022.

1. Project Introduction

1. Project Introduction

1. Project Introduction

- 12km west of Capital city Dili
- First PPP project in Timor-Leste
- RFQ in 2013, FEED in 2016
- 30-yr concession term
- 490M USD total investment
- From 350K to 1M TEU annually
- Construction started in 2019

1. Project Introduction

Project Team

- International Finance Corporation 【PPP Advisor】
- Government of Timor-Leste 【Public Partner】
- Timor Port S.A., Bollore Group 【Private Partner, Operator】
- WorleyParsons 【FEED Study and Project Management Consultant】
- China Harbour Engineering Company 【EPC Contractor】
- CCCC 4th Harbor Institute 【EPC Designer】
- ZPMC 【Equipment Manufacture】

1. Project Introduction

Project Scope

MAJOR CONSTRUCTION ITEMS

NO.	ITEM	UNIT	TOTAL	PHASE 1A	PHASE 1B	REMARK
1	CONTAINER BERTH	m	630	325	305	2 BERTHS
2	SERVICE TUG BERTH	m	80	80	/	2 BERTHS
3	SLOPE PROTECTION	m	362.2	164.3	197.9	
4	TURNING BASIN	ITEM	1			DIAMETER=600m
5	APPROACH CHANNEL	ITEM	1			WIDTH=250m
6	LAND AREA	ha	19	13.2	5.8	

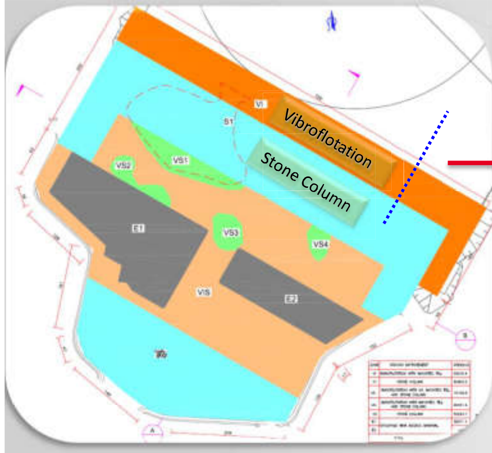
1、Dredging（-16CD）

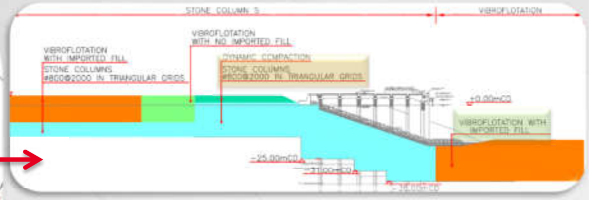
2、Container yard and building zone

3、630m long wharf with two berths

1. Project Introduction

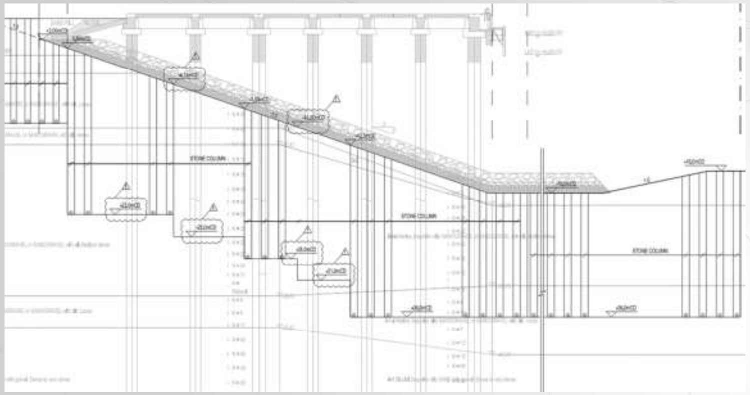
Ground Improvement Scheme





1. Project Introduction

Quay Wall Revetment Design Section and Construction Sequence



Revetment Construction Sequence

→ Dredging and reclamation

→ Stone columns installation

→ Steel pipe piles driving

→ Placement of surface layers including filter, underlayer and armor rocks

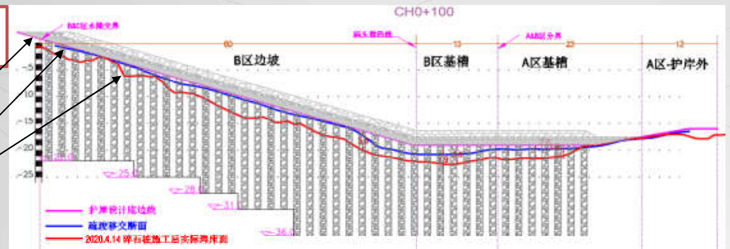
2. Recap of Marine Sediment Issue

Revetment Slope Design Cross Section with Stone Columns

[purple] Design slope

[blue] Reclaimed slope

[red] Slope after S-C installation

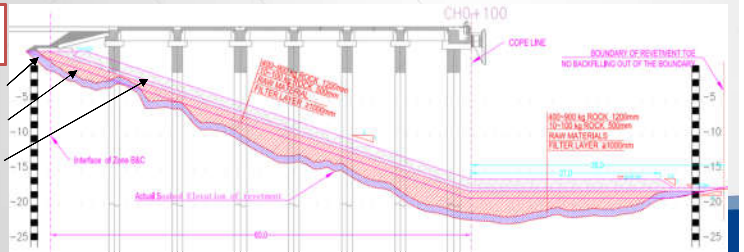


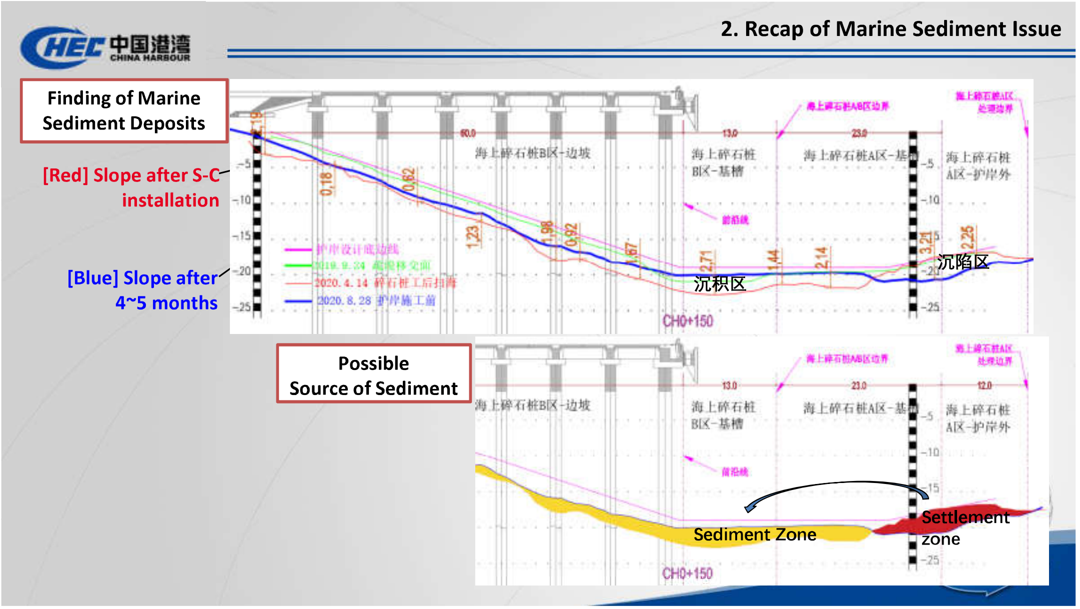
Revised Revetment Slope Design Cross Section

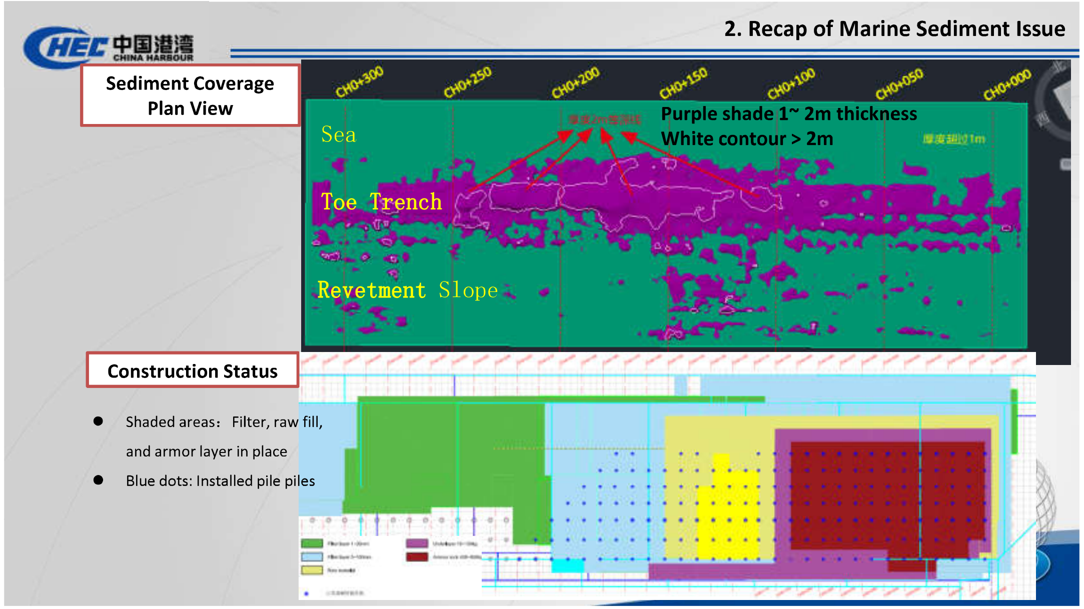
Lowered filter layer

Extra raw material

Armor rocks





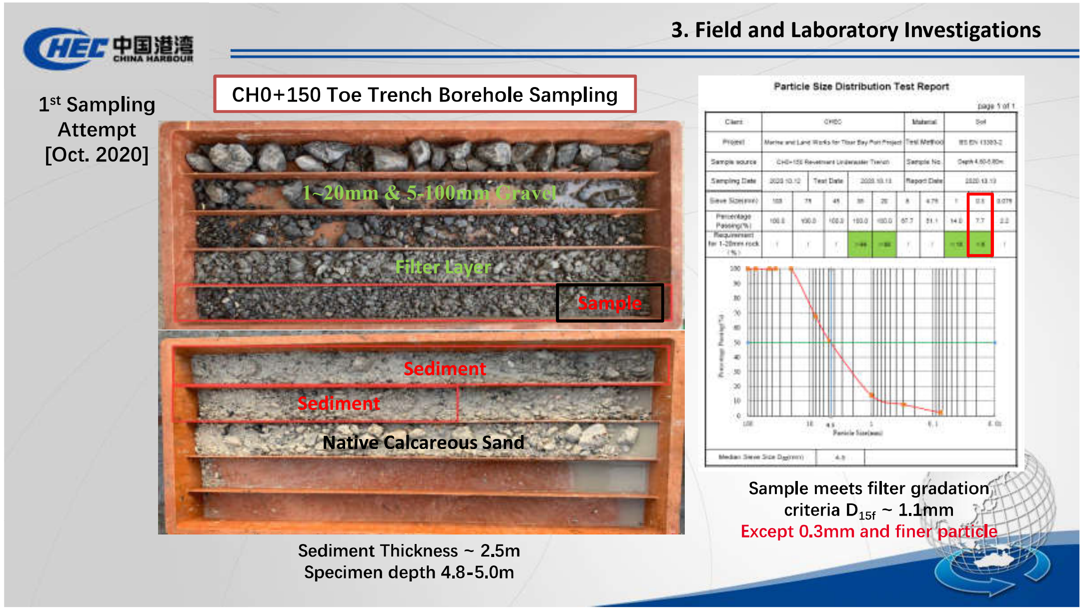


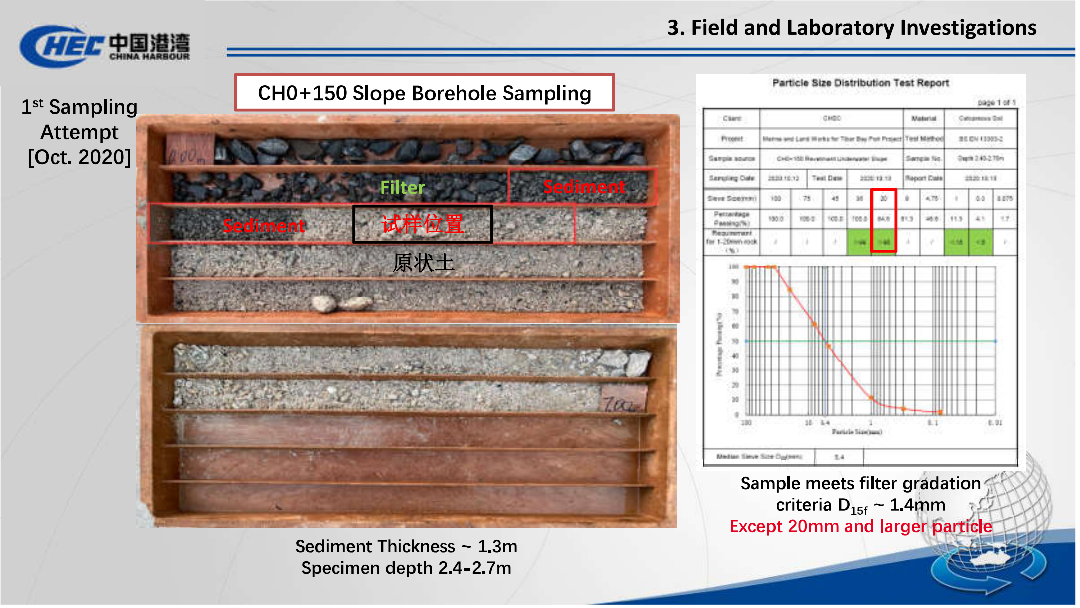
2. Recap of Marine Sediment Issue

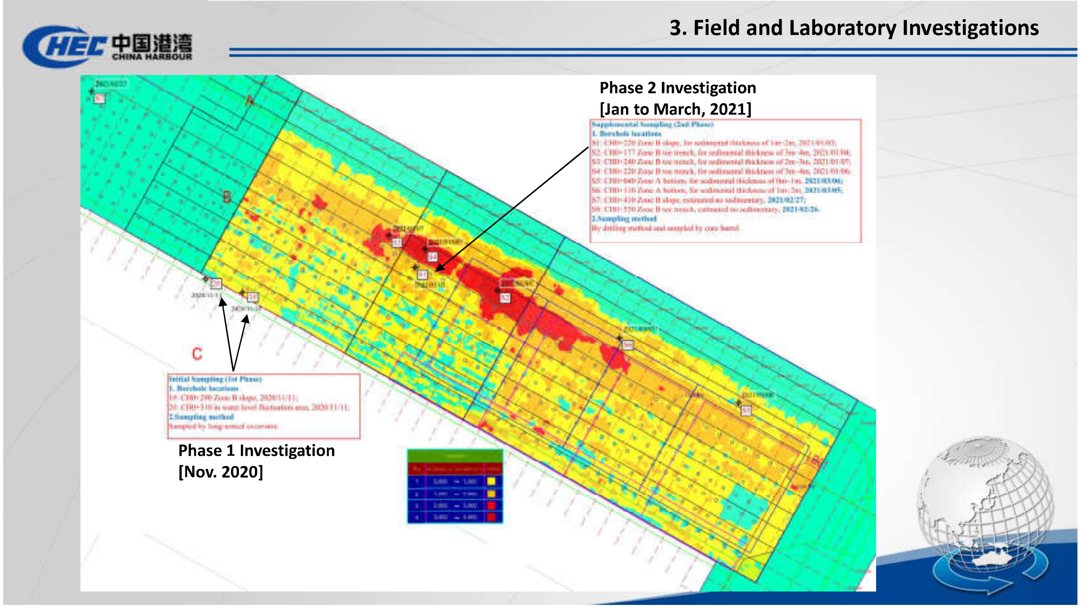
Statistics of sediment thickness in stone-column treated areas

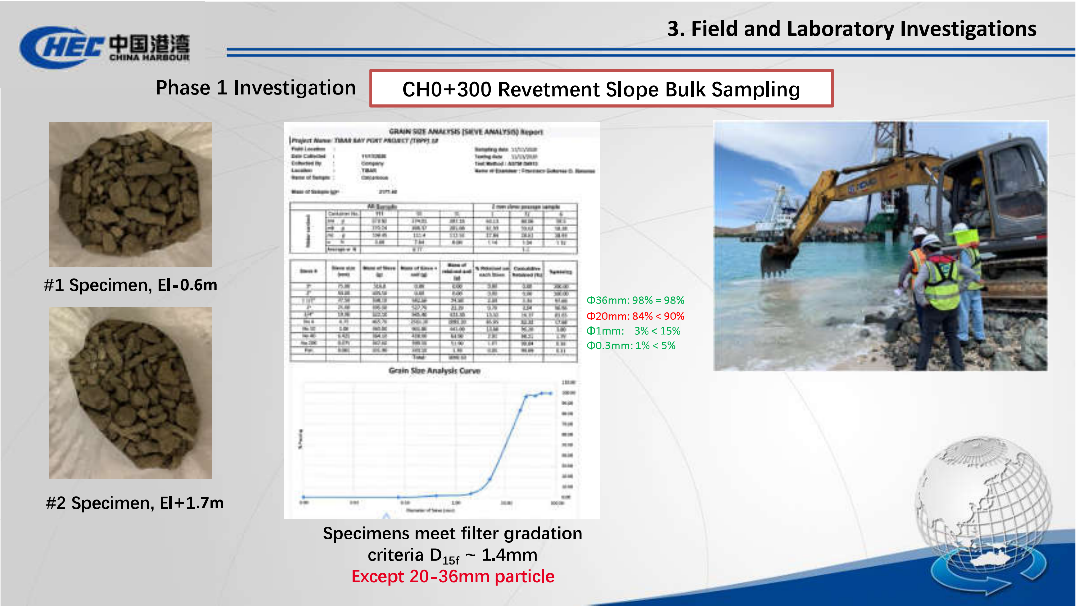
Sediment thickness (m)	Coverage area (m^2)	Percentage of SC treated area	Percentage of sediment area
< 0 (erosion)	9,124	25.8%	-
0 ~ 1	17,182	48.5%	65.3%
1 ~ 2	7,498	21.2%	28.5%
>2	1,620	4.6%	6.2%
Sum	35,424	100%	100%

- Average sediment thickness ~ 0.9m
- Total sediment volume estimate ~ 24,000m^3
- Total coverage area estimate ~ 26,300m^2




3. Field and Laboratory Investigations
1st Sampling Attempt [Oct. 2020]
CH0+150 Slope Borehole Sampling
Filter
Sediment
Sediment
试样位置
原状土
Particle Size Distribution Test Report
Sample meets filter gradation criteria D15f ~ 1.4mm
Except 20mm and larger particle
Sediment Thickness ~ 1.3m
Specimen depth 2.4-2.7m


3. Field and Laboratory Investigations
Phase 2 Investigation
[Jan to March, 2021]
B
C
Phase 1 Investigation
[Nov. 2020]


3. Field and Laboratory Investigations
Phase 1 Investigation
CH0+300 Revetment Slope Bulk Sampling
#1 Specimen, El-0.6m
#2 Specimen, El+1.7m
GRAIN SIZE ANALYSIS (SIEVE ANALYSIS) Report
Grain Size Analysis Curve
Φ36mm: 98% ≈ 98%
Φ20mm: 84% < 90%
Φ1mm: 3% < 15%
Φ0.3mm: 1% < 5%
Specimens meet filter gradation criteria D15f ~ 1.4mm
Except 20-36mm particle


3. Field and Laboratory Investigations
Phase 2 Investigation
Overwater Drilling and Core Sampling
Depth 0 - 2m
Raw fill and rock filter
2 - 5.5m
Sediment
5.5 – 6.7m
Treated soil

3. Field and Laboratory Investigations

Phase 2 Investigation | Overwater Drilling and Core Sampling

Depth 0 – 0.6m
Rock filter

0.6 – 3.1m
Sediment

3.1 – 4.5m
Treated soil

3. Field and Laboratory Investigations

Phase 2 Investigation | Sediment Specimens Laboratory Testing

(b) Specimen of S3 depth 1.50m-3.00m

4. Assessment of Sediment Properties and Impact

Three Primary Engineering Concerns with Marine Sediment

#1. Sediment may violate the filter design criteria for revetment slope

#2. Sediment may be susceptible to liquefaction during earthquake

#3. Sediment may impede stone-column pore pressure dissipation/drainage

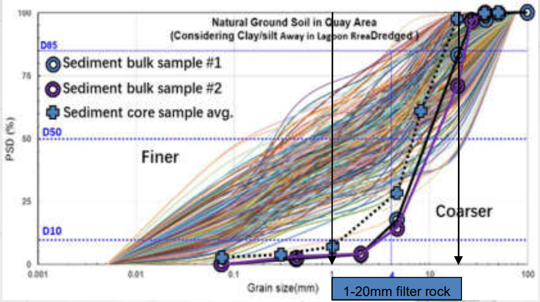
4. Assessment of Sediment Properties and Impact

Characteristics of Sediment Material

	Soil Sample		Fines Content (%)	Gravel Content (%)	D_{10} (mm)	D_{50} (mm)	Permeability K (cm/s)
Design phase	In-situ Soils	3-1-a	19	N.A.	0.01~0.13	0.2~6	0.06
		3-1-b	19	N.A.	0.01~0.2	0.2~10	0.02
Construction phase – initial sampling	Revetment Sediment (bulk sample)	#1	0.1	82.3	~3	~10	0.15
		#2	0.1	85.4	~3	~12	0.11
Construction phase – supplemental sampling	Revetment Sediment (core sample)	S1	2.1	75.5	1.6	7.8	N.A.
		S2	3.3	69.0	1.2	6.6	0.21
		S3	2.2	61.5	1.1	6.1	0.17
		S4	2.9	77.1	1.7	7.2	0.21
		S5	3.1	73.4	1.4	7.8	N.A.
		S6	2.5	66.5	1.2	6.9	N.A.

- ❑ Fines content < 3.5%
- ❑ Gravel content > 65%
- ❑ D_{10} between 1 ~ 2mm, D_{50} between 6 ~ 8mm
- ❑ 90%+ particles between 1 - 20mm (finer filter)
- ❑ Permeability coefficient 0.1~0.2cm/s

Sediment and in-situ Soils Gradation Comparison



4. Assessment of Sediment Properties and Impact

Concern #1. Sediment may violate the filter design criteria for revetment slope

Sediment and Filter Rock Gradation Comparison

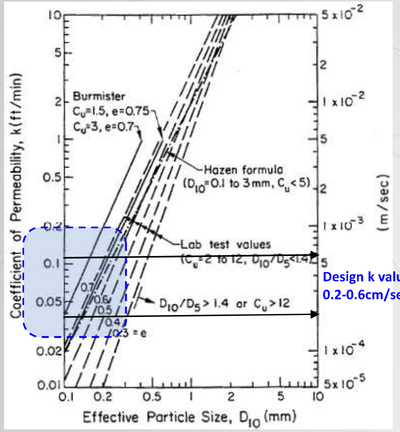
Material	ELL (<5%) D_5 (mm)	NLL (<10%) D_{10} (mm)	NUL (>90%) D_{90} (mm)	EUL (>98%) D_{98} (mm)	Median Size D_{50} (mm)
Revetment sediment (bulk samples)	~2	~3	~22	~36	~11
Revetment sediment (core samples)	~0.4	~1.3	~16	~22	~7
1-20mm filter rock	0.3	1	20	36	5
Results	Satisfactory	Satisfactory	Satisfactory	Satisfactory	-

Filter Compatibility Check

Material	Base material	D_{15f} (mm)	D_{85b} (mm)	D_{15f}/D_{85b}	Results
Revetment sediment (bulk sample)	In-situ soil	4.5	1.5	3	< 5, satisfactory
Revetment sediment (core sample)	In-situ soil	1.9	1.5	1.3	< 5, satisfactory
1-20mm filter rock	Revetment sediment (bulk sample)	1	20	0.05	< 5, satisfactory
1-20mm filter rock	Revetment sediment (core sample)	1	14	0.07	< 5, satisfactory
5-100mm filter rock	1-20mm filter rock	5	16	0.31	< 5, satisfactory

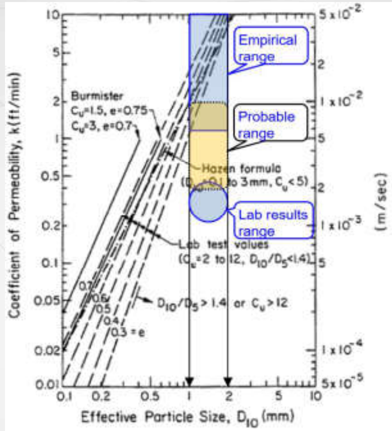
4. Assessment of Sediment Properties and Impact

Permeability of In-situ Calcareous Soils

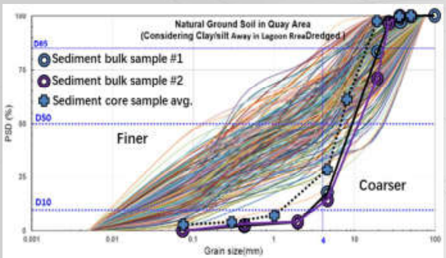


Permeability data from Liao et al. (2020)

Permeability of Sediment

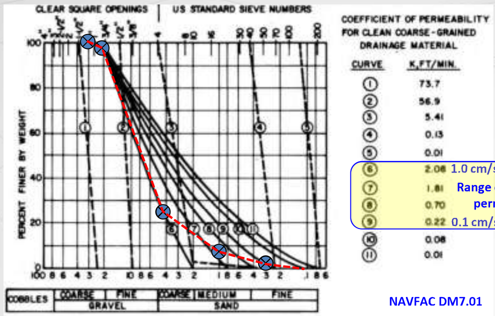


4. Assessment of Sediment Properties and Impact



Soil	Coefficient of Permeability, k (m/sec)	Relative Permeability
gravel	> 10^{-3}	high
sandy gravel, clean sand, fine sand	10^{-3} to 10^{-5}	medium
sand, dirty sand, silty sand	10^{-5} to 10^{-7}	low
silt, silty clay	10^{-7} to 10^{-9}	very low
clay	< 10^{-9}	practically impermeable

Terzaghi and Peck

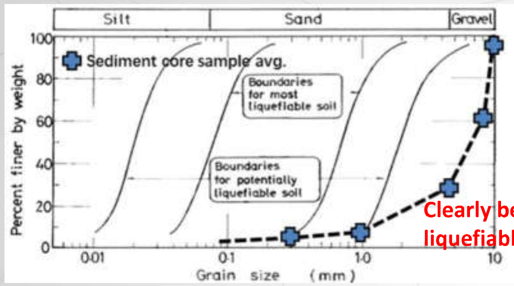


NAVFAC DM7.01

4. Assessment of Sediment Properties and Impact

Concern #2. Sediment may be susceptible to liquefaction during earthquake

Liquefaction Potential Screening of Sediment



Boundaries of liquefiable soil (Tsuchida, 1970)

4. Assessment of Sediment Properties and Impact

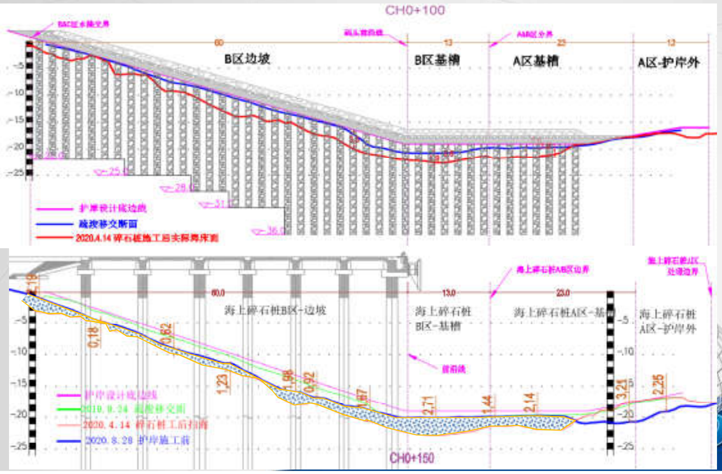
Concern #3. Sediment may impede stone-column pore pressure dissipation

Design Cross Section

Material	Permeability Coefficient (cm/s)
In-situ Soil	0.064
Stone Column	10.0

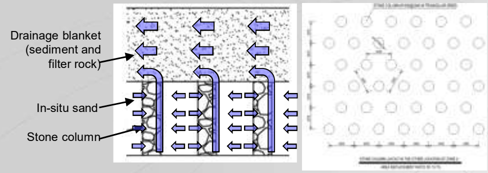
As-Built Cross Section

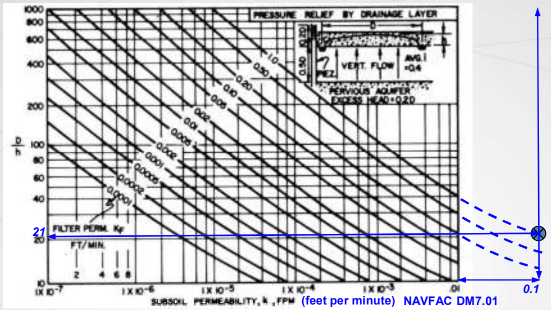
Material	Permeability Coefficient (cm/s)
In-situ Soil	0.064
Stone Column	10.0
Sediment	0.1 ~ 0.2



4. Assessment of Sediment Properties and Impact

Concern #3. Sediment may impede stone-column pore pressure dissipation





As-Built Cross Section

Material	Permeability Coefficient (cm/s)
In-situ Soil	0.05 0.1 FPM
Stone Column	10.0 20 FPM
Sediment/Blanket	0.1 ~ 1 1 FPM

- Neglecting stone-column, assume vertical drainage from soil into blanket
- Use measured in-situ soil and sediment permeability coefficients
- Obtain design D/h ratio of 21 per NAVFAC recommendation
- Assume avg. thickness of blanket (filter + sediment) h = 2.5m
- Blanket is adequate for width D=53m (> revetment sediment coverage width)

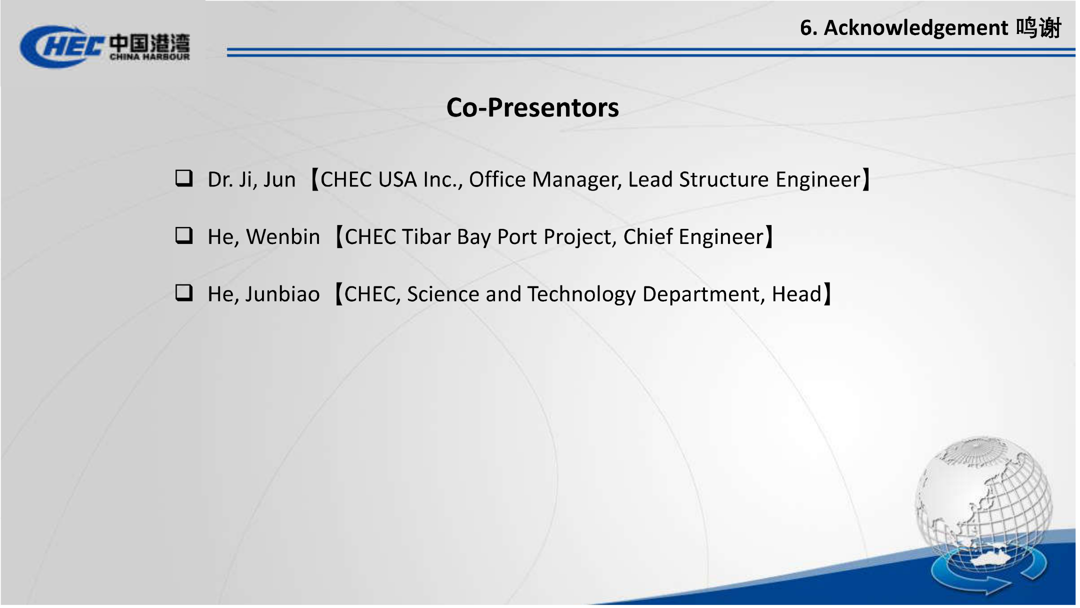
5. Final assessment and lesson learned

- ➢ Our final assessment is that the presence of marine sediment on the revetment slope does not pose significant performance hazards (filter, liquefaction, drainage) to the revetment structure.
- ➢ The assessment was reviewed and approved by EMP and ENG in April 2021. Marine sediment issue closed. Revetment and quay construction continued successfully.
- ➢ Port officially opened in November 2022.

5. Final assessment and lesson learned

Lessons Learned

- ➢ Unexpected site conditions can occur as a result of construction activities. In this case study, revetment slope subsidence occurred during the installation of stone-columns, and significant marine sediment accumulated on slope due to delay in construction. Placement of filter and armor layers over sediment deposit further complicated the situation and restrained available remedial options.
- ➢ As with most geotechnical issues, the first order of business is to investigate conditions and geomaterials involved. Investigation reliability and quality ought to be of highest importance.
- ➢ Through careful investigation and engineering assessment, the impact of sediment was found insignificant and the sediment issue was successfully resolved without a need for costly remedial measures.


6. Acknowledgement 鸣谢
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The End

"APPLICATION OF NEW VACUUM PRELOADING METHOD IN DREDGING ULTRA-SOFT SOIL"

by
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He has obtained more than 20 national patents (first inventor), such as "a soft soil filling combined with vacuum consolidation enclosure structure", and "a monitoring system for vertical deformation of the riverbed".

He Participated in the compilation of 2 technical industry standards of the People's Republic of China "Construction Specifications for Breakwaters and Bank Revetments" JTS208-2020, and the CCCC Enterprise Standard "Technical Specifications for Vacuum Preloading Reinforcement of Super-Soft Foundation Shallow Surface Sand-Free Cushion".

He has published 10 scientific papers, such as "Research on the Deformation Characteristics of the Coil with New Silt-Sand Interlayer Structure", and "Instability Mechanism and Measures to Control the Slope of a Silt-Containing Layer".

He undertook a number of scientific research projects, and won the provincial and ministerial level scientific and technological progress 6 awards, such as "Research and Application of Key Technologies for the Construction of Artificial Islands in Colombo Port City under Strong Surge Environment" China Water Transport Construction Industry Association First Prize, "Research on the Coupling Mechanism of Primary and Secondary Consolidation of Typical Soft Soils Based on Soil Structural Properties" and its engineering application", won the second prize of China Water Transport Construction Industry Association.

Application of New Vacuum Preloading Method in Dredging Ultra-Soft Soil

Eng. Yin Changquan
Technical Director
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1. Classification and characteristics of Ultra Soft Clay



With the rapid development of the economy in China's coastal areas, land resources are becoming limited. The use of dredged soil from ports and waterways for land reclamation and vacuum preloading ground treatment has become the main means of alleviating land resource scarcity.

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1. Classification and characteristics of Ultra Soft Clay

The dredged soils is gradually sorted and deposited under hydraulic action, forming a soft soil with high moisture content, high compressibility, and extremely low strength. It is crucial to classify this type of soil reasonably and select appropriate ground treatment method.

Boundary parameters of soft clay in China

index / name	Density(kN/m^3)	Water content (%)	Moisture content Liquid limit	Unconfined compressive strength (kPa)
Soft clay	≥16	≤70	≤1.4	≥5
Ultra soft clay	< 16	> 70	> 1.4	< 5

- Vacuum preloading method
- surcharge preloading method

1. Classification and characteristics of Ultra Soft Clay

Basic situation for soft clay in China:

Ground treatment projects in Tianjin, Lianyungang, Ningbo and other city, mainly consist of silt and muddy soil in the shallow surface layer, with a moisture content mostly concentrated between 50% and 90%. Some parts have a moisture content more than 100%.

For ultra soft foundations, they often undergo 3-5 months of preloading, and even after groud treament, often exceeding the theoretical settlement value by evaluation, the strength vane shear strength is only 5-12kPa, which cannot meet the requirements or expectations of the foundation's use.

Further improvement of ground will be needed to gain required strength & settlement limits, and this will acquire more time & money.

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2. Traditional vacuum preloading method

2.1 Development History

In 1952, Swedish W. Kjellman proposed the method of vacuum preloading to strengthen soft soil foundation and preliminarily explained its mechanism.

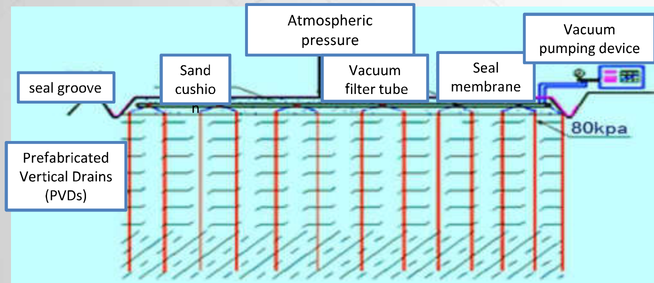
In 1958, the Philadelphia Airport in the United States completed the expansion project of the aircraft runway by combining vacuum well point precipitation with drainage sand wells. The highest vacuum level in the entire region was 380mmHg (51kPa).

In the 1980s, vacuum preloading was used to in Tianjin china, resulting in a vacuum degree of 600mm Hg (81kPa) under the membrane. This pumping system was rapidly developed in China.

2. Traditional vacuum preloading method

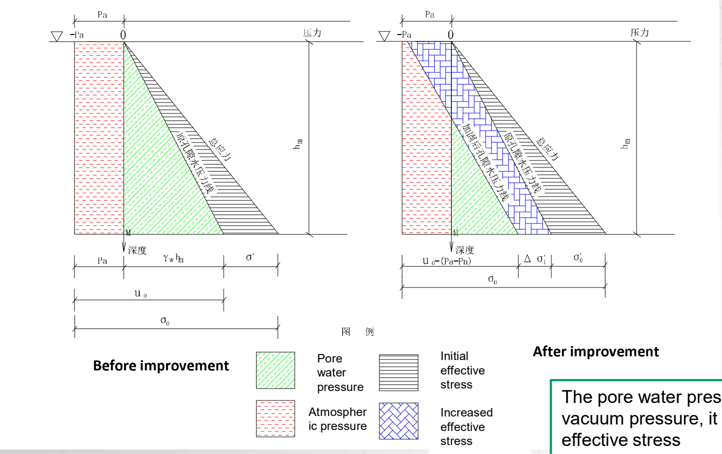
2.2 Principle of vacuum preloading method

Use vacuum equipment to extract air and water under the sealing membrane, negative pressure is formed in the foundation and transmitted to the deep layer, resulting in consolidation settlement and improvement of the foundation soil.



2. Traditional vacuum preloading method

Stress distribution in soil layer



The pore water pressure reduce due to vacuum pressure, it results increase of effective stress

2. Traditional vacuum preloading method

2.3 Scope of application

Soft soil grounds mainly composed of cohesive soil such as silty clay, clay, and hydraulic fill clay. When there is a permeable soil layer such as sand, treatment measures need to be taken.

2. Traditional vacuum preloading method

2.4 Surface Treatment

There are several methods commonly used for Surface Treatment:

- **2.4.1 Air drying**

The surface layer of the foundation naturally forms a certain thickness of hard-shell layer with a working platform with bearing capacity before the treatment of the foundation. It requires a long time of air drying, usually one to three years, depending on the soil conditions.

- **2.4.2 Laying bamboo rafts**

On the surface of the foundation, lay bamboo raft and fill with a certain thickness of sand to form a working platform to gain bearing capacity.

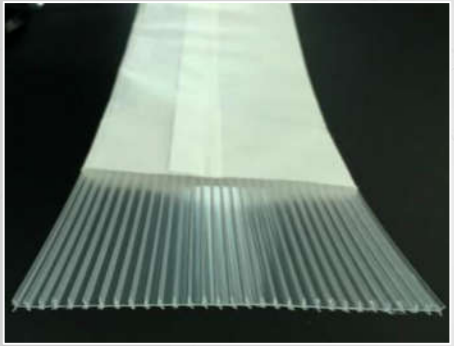
2. Traditional vacuum preloading method

2.5 Drainage channels

- Vertical: PVD, sand wells, etc;
- Horizontal: sand cushion, three-dimensional geogrid, etc.

2. Traditional vacuum preloading method

2.5.1 Prefabricated Vertical Drains (PVDs)



2. Traditional vacuum preloading method

Performance parameters of commonly used PVDs

project/ model	model	A-type	B-type	C-type	D-type	test conditions
setting depth(m)		⩽15	⩽25	⩽35	⩽50	
Longitudinal Flow rate (Cm^3/S)		⩾15	⩾25	⩾40	⩾55	lateral confining pressure 350KPa
permeability coefficient of filter membrane(Cm/S)		⩾$5x10^{-4}$				Soak the specimen in water for 24h
Equivalent pore size of filter membrane(mm)		0.05 ~ 0.12				Calculated as O_{95}
Tensile strength of plastic drainage board(KN/10cm)		⩾1.0	⩾1.30	⩾1.5	⩾1.8	when the elongation is 10%
Tensile strength of filter membrane(N/cm)	dry state	⩾15	⩾25	⩾30	⩾37	when the elongation is 10%
	wet state	⩾10	⩾20	⩾25	⩾32	when the elongation is 15%, the specimen is immersed in water for 24h

Precaution:

During the installation process of PVD, it is strictly prohibited to kink, break, or damage the filter membrane of the PVD.

2. Traditional vacuum preloading method

2.5.2 Horizontal drainage channels

- **Sand cushion:** Materials with a permeability coefficient generally not less than 10^{-3}cm/s and can also play a certain role in reverse filtration. Usually, well graded medium or coarse sand is used, with a clay content of no more than 3%. Thickness of the onshore drainage cushion is more than or equal 0.5m, and the thickness of the underwater cushion is more than or equal 1.0m.
- **Three-dimensional geogrid**:

2. Traditional vacuum preloading method

2.6 Vacuum pump

The vacuum pumping has undergone the development of vertical pumps, horizontal pumps, and new technology pumps, and its efficiency has gradually improved.

2. Traditional vacuum preloading method

2.7 Sealing system

2.7.1 Horizontal:

The sealing membrane generally adopts 2-3 layers ,thickness is 0.12mm~0.10mm.

Minimum tensile strength (MPa)		Minimum elongation at break (%)	Minimum right angle tear strength	thickness
direction	transverse		(kN/m)	(mm)
18.5	16.5	220	40	0.12～0.16


2. Traditional vacuum preloading method

2.7.2 Vertical:

When there is a deep and strong permeable layer and breathable layer , it is advisable to reduce its permeability and breathability by setting up clay sealing walls or vertically laying membrane.

2. Traditional vacuum preloading method

2.8 Loading

- In the early stage of vacuum pumping, the high-pressure difference between the inside and outside of PVD will cause small soil particles to collect on the surface and surrounding areas of the PVD, forming a "mud seal layer" .To avoid the occurrence of a "mud seal layer" , vacuum load shall be increased gradually ,especially for ultra soft clay.
- In vacuum preloading, the soil undergoes inward displacement, therefore, compared to surcharge preloading, vacuum preloading has less impact on surrounding buildings.
- When the vacuum load is less than the design preload load, it can be combined with surcharge preloading.

2. Traditional vacuum preloading method

2.9 Quality Inspection

soil test： Lab tests of soil samples, vane shear tests on-site, and plate load tests before and after treatment,

The monitoring data : vacuum degree, ground settlement, settlements at deep layers, horizontal displacement, and pore water pressure .

For example: The degree of consolidation and residual settlement of ground can be evaluate through settlement monitoring data.

$$S_\infty = \frac{S_3(S_2 - S_1) - S_2(S_3 - S_2)}{(S_2 - S_1) - (S_3 - S_2)}$$

$$S_\infty = S_0 + \frac{1}{\beta}$$

$$S_t = S_0 + \frac{t}{\alpha + \beta t}$$

2. Traditional vacuum preloading method

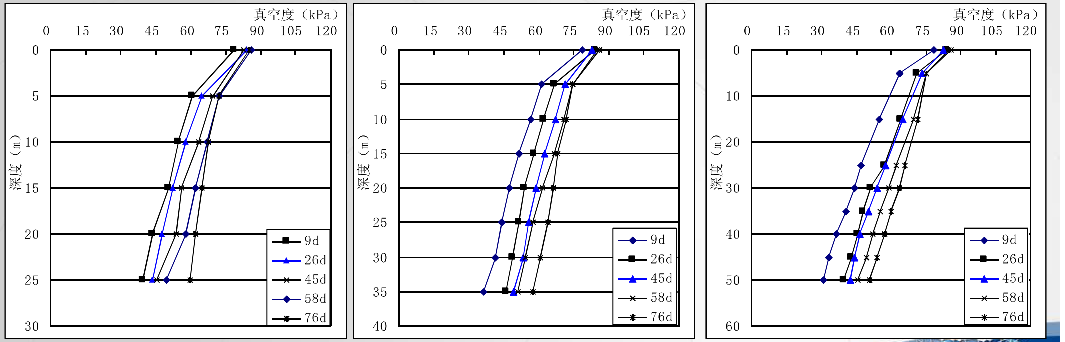
2.10 Issues related to vacuum preloading

- Depth of improvement
- Sealing effect
- Selection of vertical drainage channels
- Low strength gain at surface after treatment

2. Traditional vacuum preloading method

2.10.1 Depth of improvement

A large number of practical engineering practices have shown that the effective depth of improvement has exceeded 50 meters.



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2. Traditional vacuum preloading method

Project Name	Depth of plastic drainage board(m)	Spacing(m)
Reinforcement of soft foundation behind the C-C section of the pier on the south side of Tianjin Port East Pie	23	0.65
Soft Foundation Reinforcement of Container yard behind Shantou Multipurpose berth	24.9	1
Soft Foundation Reinforcement of the Rear Storage yard of shenzhen Mawan Port 3# Pier	17	1
zhuhai power plant foundation treatment project	20~21.5	1
Tianjin Port Coal Terminal Soft Foundation treatment project	20.5	0.9~1.3
Reinforcement of soft foundation in the rear yard of Tianjin Port nanjiang general Bulk Cargo Berth	19.7	0.8
Treatment and reinforcement of soft foundation behind the nansha Seawall revetment Project in Panyu	16.2 ~ 32.7	1
Foundation Treatment of Zone C in the East Area of Tianjin Port Container Logistics center	19.5	1
Foundation Treatment of Lianyungang port zhongyun logistics park	10 ~ 19	0.8
Foundation Treatment of Tianjin Port North Port Container terminal phase Ⅲ project	20.7	0.8

2. Traditional vacuum preloading method

2.10.2 Sealing effect

2.10.2.1 Surface sealing

- Insufficient depth of the sealing groove or incomplete sealing clay material;
- quality issues of the sealing membrane.

2.10.2.2 Vertical sealing

- The permeable layer is located on the surface - the depth of the seal groove exceeds the permeable layer;
- The permeable layer is in the upper and middle depths - clay sealing walls, vertical membrane;
- The permeable layer is at the bottom - the PVD should generally not reach the permeable layer.

Traditional vacuum preloading method

2.10.2.3 The clay sealing wall

- Before the construction of the clay sealing wall, drilling and probing should be carried out at every 50-100m . The depth of the sealing wall should be 50-100cm into the impermeable layer;
- The clay particle content of the clay sealing wall should generally be greater than 15%; The permeability coefficient of clay sealing wall should be less than 1 × 10-5cm/s;
- The clay sealing wall should be mixed in two rows, and the thickness should not be less than 1.2m.

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2. Traditional vacuum preloading method

2.10.3 Selection of vertical drainage channels

Loss in the vertical drainage channel when transmitting vacuum,

Resistance along the sand cusion.

Sand cusion thickness 、 sand permeability

Resistance along the vertical drainage channel.

PVD type 、 installation

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2. Traditional vacuum preloading method

2.10.4 Low surface strength after treatment

After 3 to 5 months of preloading,

- The post constrution settlement is often exceeding the theoretical calculation settlement.
- The strength gain from the ground treatment is small and sometimes cannot achieve the requirements for the use of the foundation.

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2. Traditional vacuum preloading method

There are two measures to solve this problem:

1) The secondary treatment method is adopted. Firstly, the ultra soft clay on the surface should be preliminarily treated, and then deep treatment should be carried out .

2) The second method is to reduce the spacing between PVD or use a combination of long and short PVD for conventional vacuum preloading treatment.

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3. Secondary Vacuum Preloading

3. Secondary Vacuum Preloading method

Treated the shallow soft clay layer by manually inserting PVD;

At the same time of treatment, a certain thickness of sand is backfilled on the sealing membrane, usually 80cm. The working platform is formed by combining shallow treated layer and sand to meet the requirements of mechanical construction, and deep vacuum preloading is carried out.



3. Secondary Vacuum Preloading

The secondary treatment method is a highly effective treatment for super soft soil where the surface hydraulic fill soil is thick, has high moisture content, low strength, and difficult for personnel and construction machinery to work with. When compared to the primary treatment,secondary treatment using sand has several advantages:

- Working Platform (Cushion Layer) Optimization
- Enhanced Structural Integrity
- Moisture Control and Stability
- Settlement Reduction and Quality Improvement
- Optimized Drainage and Transmission

3. Secondary Vacuum Preloading

It has been found that for disturbed soil in the liquid state, reducing the spacing between PVD can effectively improve the shear strength of the soil, The use of PVD spacing of 40cm, 50cm, and 60cm, can quickly and effectively improve the foundation bearing capacity, eliminate settlement, and meet the design requirements.



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4. Vacuum Preloading without sand Cushion

Preloading without Sand Cushion

Due to the shortage of sand sources and soaring prices, mining is strictly prohibited in china, this method is introduced.

Vacuum preloading process method using the without sand cushion developed four methods :

- 4.1 Self sealing vacuum preloading,
- 4.2 Direct suction without sand cushion vacuum preloading,
- 4.2 mud sealing vacuum preloading,
- 4.4 Replacing sand cushion with three-dimensional composite geotextile drainage network.

4. Vacuum Preloading without sand Cushion

4.1 Self sealing vacuum preloading process

- This is a new process using in-situ surface clay as a sealing layer and filter pipes as drainage channels. This process binds the filter tube and PVD together, and while inserting the PVD, steps the filter tube into the mud for 30-50cm.
- This process is suitable for newly filled high moisture content dredged soil.
- Compared with the traditional vacuum preloading scheme, it eliminates the use of geotextile, horizontal sand cushion (geosynthetic drainage cushion), and sealing film as construction cushion.

Note: The soil used for self sealing on the surface needs has to be treated separately.

4. Vacuum Preloading without sand Cushion

4.2 Vacuum preloading process for direct suction without sand cushion

- This process is suitable for both shallow and deep layers or a combination of the two for secondary treatment.
- Shallow process: Bind the PVD onto the filter tube, manually set the PVD while pressing the filter tube into the mud for 30cm~50cm, and lay two layers of sealing film at once;
- Deep process: Mechanical installation of PVDs, excavation of filter pipe trenches at each row of PVDs, with a depth of 30cm~50cm, binding of PVD filter pipes, embedding of filter pipe trenches, and laying of two layers of sealing film at once.
- This process reduces the drying time of hydraulic fill clay and effectively shortens the treatment period; The process is simple, and the construction quality is easy to control.

4. Vacuum Preloading without sand Cushion


天津市工程建设标准 DB
DB/T29-263-2019
备案号：J14651-2019
天津市无砂法真空预压加固软基技术标准
Technical Standard of Non-sand Blanket Vacuum Preloading Method for Soft Soil Improvement in Tianjin
2019-04-26 发布 2019-07-01 实施
天津市住房和城乡建设委员会 发布

4. Vacuum Preloading without sand Cushion

4.3 Vacuum preloading technology for mud film sealing of without sand cushion

- This is a new type that uses filter pipes as drainage channels and rear filled clay as sealing layers.
- This process involves laying PVD on the surface of the foundation, connecting the PVD and filter pipes, and filling approximately 80cm thick clay on the surface of the treatment area as a sealing layer.

4. Vacuum Preloading without sand Cushion

4.4 Composite 3D Geotextile drainage network replacing sand cushion

- A shallow vacuum preloading process using a recyclable three-dimensional composite geotextile drainage network instead of a sand cushion as a horizontal drainage cushion.
- The three-dimensional composite geotextile drainage network adopts the overall form of three-dimensional geotextile network with non-woven fabric sheets on both sides.
- This can be directly laid on ultra soft foundations, can be reused, and effectively reduces the loss of negative pressure in the sand cushion process.

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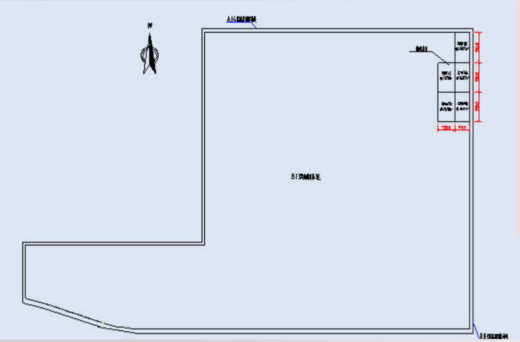
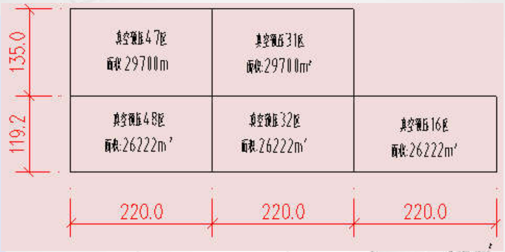
5. Case Studies

Engineering Cases

The trail area is located in Tianjin Nangang Industrial Zone, with a treatment area of 138000 m^2.

There are five zones in total, including 47 zone, 48 zone, 31 zone, 32 zone, and 16 zone.





5. Case Studies

The hydraulic fill clay within a depth range of 5m on the shallow surface of the trail area has a moisture content of 103% to 124%, and a clay content of about 40%, belonging to the flow mud with a high content of fine particles.

The average vane shear strength of the five trail areas is less than 2KPa, indicating that the bearing capacity of the foundation is very low, making it difficult for people and machinery to access.

reinforced area	Particle composition (mm)			physical properties of soil			Limit water content		
	fine sand	silt	Clay particle	moisture content	wet density	Void ratio	liquid limit	plasticity index	Liquidity index
	0.25~0.075	0.075~0.005	<0.005	ω	ρ	e	ω_L	I_P	I_L
	%	%	%	%	g/cm^3	–	%	–	–
16zone	0.6	60.2	39.2	112.3	1.41	3.16	39.9	19.7	4.69
31zone	0.9	51.7	36.1	103.6	1.44	2.9	40	19.8	4.23
32zone	0.5	60.2	39.3	122.6	1.38	3.43	39.9	19.7	5.24
47zone	0.4	60.1	39.5	124.9	1.38	3.5	39.2	19.2	5.46
48zone	0.5	58.9	40.6	104.7	1.43	2.94	39.9	19.7	4.3

Statistics on physical indicators of surface fill soil

	16zone	31zone	32zone	47zone	48zone
average value (kPa)	1.6	1.7	1.8	1.6	1.7
Maximum value (kPa)	2.7	2.4	2.2	2.2	2.2
minimum value (kPa)	0.9	1.2	1.4	1	1

Statistics of vane shear strength values of the cross plate before treatment

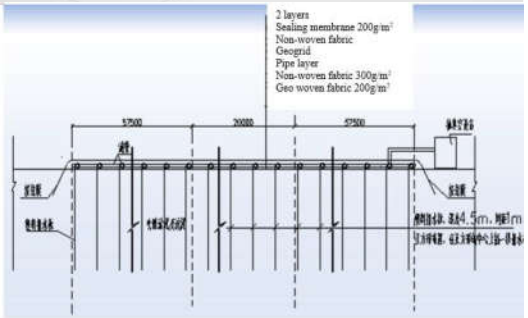
5. Case Studies

5.1 Shallow layer vaccum prelongding

a layer of woven fabric and a layer of non-woven fabric were laid on the hydraulic fill flow mud, and PVDs were manually installed.

The depth of the PVD is 4.5m, and the exposed length is 0.4m. Each row of PVDs is equipped with a filter tube, with a spacing of 50cm between the filter tubes. Connect the PVD and filter tube, wrap the PVD around the filter tube for one and a half circles, and fix it with a self-locking device.

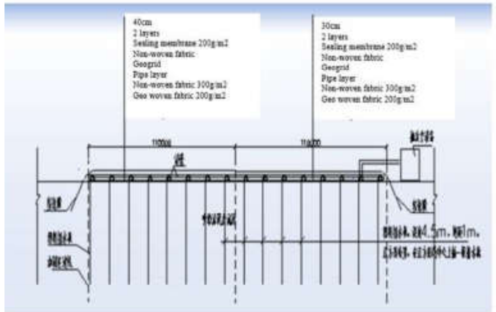
Lay one layer of geogrid, one layer of non-woven fabric, and lay two layers of sealing film at once.

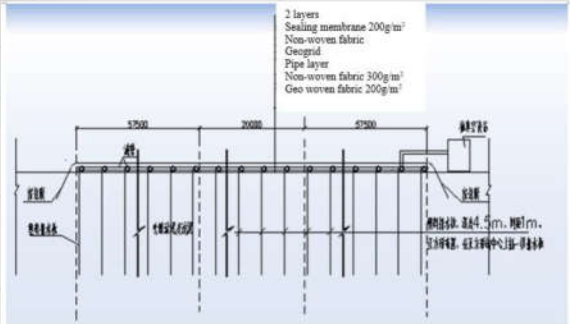


Case 1 :In areas 47, 31, and 16 of the trail,

The spacing between PVDs is 1.0m and arranged in a square shape, with one PVD installed at the centroid of every four adjacent PVDs; The actual spacing between PVDs is about 0.7m.

5. Case Studies





Case 2: After the shallow treatment and unloading of Zone 47 in the trail, black sand was filled with half of the area of 30cm and half of the area of 40cm.

Case 3: Test zones 48 and 32, with PVDs spaced 0.8m apart and arranged in a square shape. At the same time, one PVD is installed at the centroid of every four adjacent PVDs; The actual spacing between PVDs is 56cm

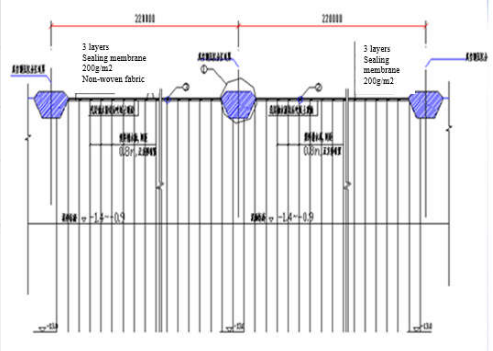
5. Case Studies

5.2 Deep vacuum preloading

The spacing between PVDs in all 5 trail areas is 0.8m, arranged in a square shape. The bottom elevation of the PVD is -13.0m, and the exposed area is 0.4m. Lay a filter tube at the position of each row of PVDs, and the PVD filter tubes are connected through self-locking.Deep vacuum preloading requires a vacuum degree under the membrane of not less than 85kPa. The estimated vacuum dead load time is 100 days.

Case 1: Directly lay sealing membrane on the filter tubes in Zone 16, Zone 48, and Zone 31 of the trail,

Case 2: Cover the filter tubes in Zone 32 and Zone 47 with medium to coarse sand. The thickness of the sand layer on the top of the filter tube should not be less than 10cm, and then lay the sealing membrane.



5. Case Studies

5.3 unloading Standards

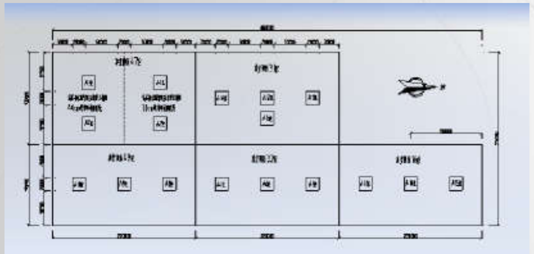
Shallow layer pumping consolidation unloading standard: On the 20th day of formal pumping, on-site vane testing will be conducted, with 8 points in each area. The average vane shear strength within the range of 2m~3m in the shallow layer is required to be greater than 10kPa. If it cannot be achieved, continue to vacuum and conduct a vane inspection every 10 days until the foundation strength meets the requirements.

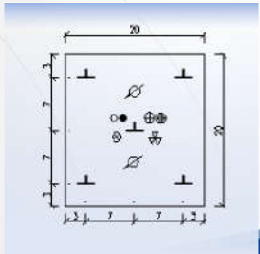
Deep vacuum preloading unloading standard: The consolidation degree calculated based on the measured settlement curve shall not be less than 90%, and the average surface settlement rate measured for 5 consecutive days shall not exceed 2.5mm/d.

5. Case Studies

5.4 Design of inspection and monitoring plan

To reduce experimental errors, the inspection and monitoring items of each trail area were placed in a 20m * 20m inspection and monitoring area, and a total of 17 inspection areas were set up for the trail. The test items mainly include surface settlement, membrane vacuum, pore water pressure, layered settlement monitoring, on-site vane shear strength before 、after and during treatment, on-site soil sampling before and after treatment, indoor geotechnical tests, etc.





5. Case Studies

5.5 treatment time

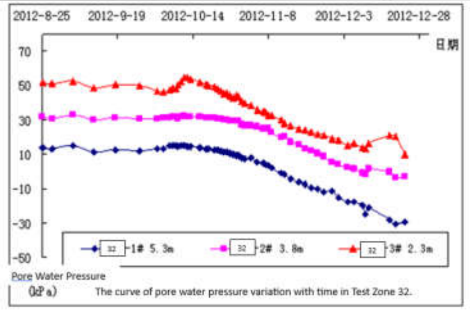
The actual treatment time for the shallow layer is 40 days, and the deep layer treatment time is 100 days.

5. Case Studies

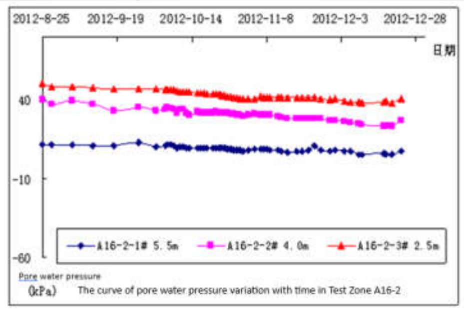
5.6 shallow treatment result

5.6.1 Shallow pore pressure dissipation

The pore pressure dissipation values of zones 32 and 48 with a drainage spacing of 56cm are 35.05kPa and 30.70kPa, while the pore pressure dissipation values of zones 16, 31, and 47 with a drainage plate spacing of 70cm are 10.63kPa, 11.77kPa, and 14.11kPa. The treatment area with small spacing between PVDs has a high dissipation value of pore pressure.



The curve of pore water pressure variation with time in Test Zone 32.



The curve of pore water pressure variation with time in Test Zone A16-2

5. Case Studies

5.6.2 Shallow surface settlement

The total settlement of shallow treatment in zones 32 and 48 of the trail is 1734.5mm and 1763.7mm, while in zones 47, 31, and 16 of the trail, it is 1267.32mm, 1097.3mm, and 1525.2mm.

Estimation of consolidationg deree: The consolidation degree of Zone 32 and Zone 48 is 70%~75%, and the residual settlement is 35cm~50cm.The consolidation degree of zones 47, 16, and 31 is 50%~65%, and the residual settlement is 60cm~70cm;

Test Area	Average settlement (mm)	Average sedimentation rate (mm/d)	Settlement during insertion period (mm)	total settlement (mm)
47zone	1078.9	5	188.3	1267.2
32zone	1339.7	4.4	394.8	1734.5
48zone	1173.1	3.7	590.6	1763.7
31zone	937.5	2.3	159.8	1097.3
16zone	1158.4	3.2	366.8	1525.2

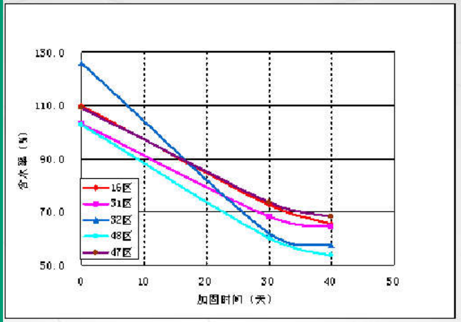
5. Case Studies

5.6.3 Changes in soil quality before and after shallow treatment

The moisture content rapidly decreases during the treatment process, and the reduction speed in the early stage of treatment is faster than in the later stage.

The average water content of 32 and 48 zones with a spacing of 56cm is 62.0% and 60.1% in 30 days, and 57.3% and 53.9% in 40 days; The 30-day average moisture content of zones 31, 16, and 47 with a spacing of 70cm is 68.2%, 72.6%, and 74.1%, while the 40-day average moisture content is 64.5%, 65.5%, and 68.1%.

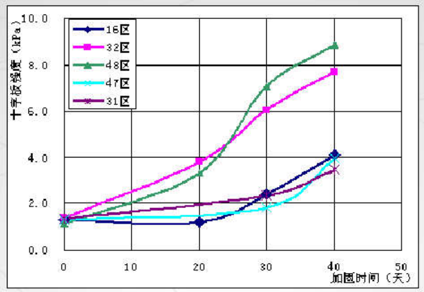
Reducing the spacing of PVDs can accelerate the reduction rate of soil moisture content in the treatment area.



5. Case Studies

5.6.4 Shallow vane shear strength

After 40 days of vacuum preloading treatment, the average vane shear strength in zones 16 and 47 is about 4kPa, the average vane shear strength in zone 31 is 3.5kPa, and the average vane shear strength in zones 32 and 48 is 7.7kPa and 8.8kPa.



reinforced area	Vane shear strength (kPa)			
	Before reinforcement	20days	30days	40days
16zone	1.3	1.2	2.4	4.1
47zone	1.3		1.8	4
31zone	1.4		2.4	3.5
32zone	1.4	3.8	6.1	7.7
48zone	1.2	3.3	7.1	8.8

5. Case Studies

5.6.5 Shallow layer unloading standard

Based on a large number of in-situ vane tests, data statistics, composite soil pressure tests, and actual working conditions of the insertion machine on site, it is recommended to use the in-situ vane shear test of soil as the recommended detection method.

It is recommended to use lightweight plug-in machinery for PVD construction, and the unloading standards for shallow treatment without sand cusion are as follows:

Within the depth range of 0.2~0.5m of the surface layer, the average vane shear strength shall not be less than 7kPa, and the average shear strength of the vane within the treatment range below 0.5m shall not be less than 4kPa.

It is recommended to use heavy-duty plug-in machinery for PVD construction, and the unloading standards forshallow treatment without sand cusion are as follows:

Within the depth range of 0.2~0.5m of the surface layer, the average vane shear strength shall not be less than 10kPa, and vane shear strength within the teatment range below 0.5m shall not be less than 7.5kPa.

5. Case Studies

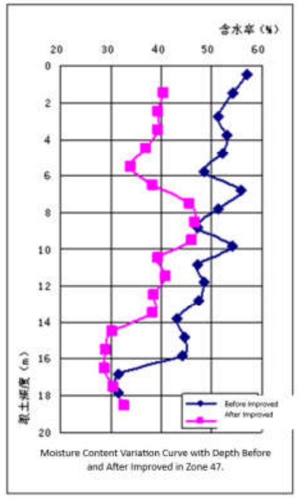
5.7 deep treatment result

5.7.1 Changes in soil quality before and after deep treatment

There is no significant difference in the effect of between traditionanl and without sand cusion method on the water content.

The average moisture content of the trail area before deep treatment is 41.2%~54.6%. After treatment, there is no significant difference in the average moisture content of each area, ranging from 35% to 38%, and the moisture content decreases by 10.2%~34.9%.

	16zone	31zone	32zone	47zone	48zone
Moisture content before reinforcement (%)	54.6	49.9	48.8	47.1	41.2
Moisture content after reinforcement (%)	35.5	36.9	38.3	35.7	37
reduce (%)	34.9	25.9	21.6	24.2	10.2



Moisture Content Variation Curve with Depth Before and After Improved in Zone 47.

5. Case Studies

5.7.2 Surface settlement

After deep treatment, the total settlement of the trail area is 3217mm~3658mm, and the estimated consolidation degree is between 90.2%~90.7%, meeting the design requirements.

During the deep vacuum pumping process, the settlement of the soil below the original mud surface is 281mm~455mm, accounting for 32% of the total thickness of the hydraulic fill layer, resulting in 51.4%~69.2% settlement.

Settlement mainly occurs in the hydraulic fill layer.

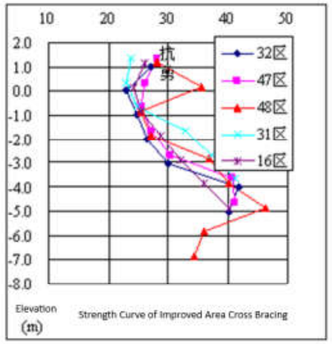
Settlement statistics	16zone	31zone	32zone	47zone	48zone
Shallow settlement (mm)	1160	938	1340	1079	1173
Settlement during insertion period (mm)	1296	1404	1240	1329	1280
Deep vacuum settlement (m)	1072	1037	1078	911	764
total settlement	3528	3379	3658	3319	3217
Settlement rate for 5 consecutive days (mm/d)	1.4	1.6	1.6	1.7	1.3
Degree of consolidation (%)	90.3	90.7	90.2	90.4	90.3

5. Case Studies

5.7.3 Strength change

Vane shear strength change:

The surface soil around 8 meters is filled with hydraulic fill soil. After shallow and deep vacuum preloading treatment, the vane shear strength increases significantly, and the vane shear strength increases from less than 2 kPa to 23.1 kPa~41.2 kPa.



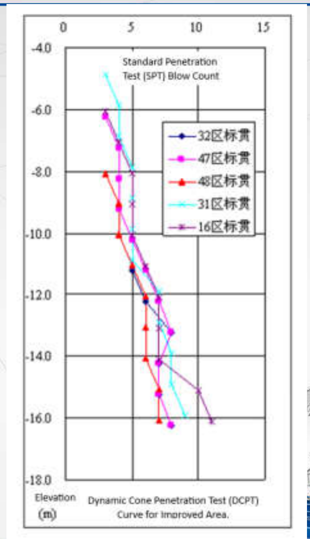
Strength Curve of Improved Area Cross Bracing

reinforced area	47area	32area	48area	31area	16area
Shear strength of surface 0m~6m cross plate (kPa)	29.7	28.9	32.2	30.5	28.8
10m～18mStandard Penetration Blows (Blows)	6.2	6	5.3	6.6	7

5. Case Studies

SPT change:

Before treatment the soil layer below the original mud surface, the vane shear strength is 4 kPa to 35.9 kPa, and the blow number of SPT for the last layer of soil is 5-6. After treatment, it is difficult to take vane shear strength test below the original soil layer, and the blow number of SPT is 3-11.



5. Case Studies

5.8 Compared with traditional vacuum preloading

5.8.1 Process differences (taking zones 32 and 48 as examples)

The adjacent treatment areas of the trail area are Zone 15, Zone 29, and Zone 46, which use traditional sand secondary treatment technology.

Main differences:

- shallow treatment:The spacing in Zone 32 and Zone 48 of the trail area is 56cm, and the spacing between PVDs in adjacent areas with sand treatment is 70cm; The treatment time for the trail area is 40 days, and for the area with sand treatment is 30 days;
- deep treatment:Before the deep treatment of the trail area, there was no fine sand cushion layer; Zone 32 of the trail is covered with medium coarse sand on the filter tube, while the traditional zone is completely covered with 80cm thick medium coarse sand on the filter tube.

5. Case Studies

5.8.2 Comparison of results

Vacuum preloading reinforcement process	Sand free process in the experimental area					Traditional craftsmanship in adjacent reinforcement areas		
	16zone	31zone	47zone	32zone	48zone	15zone	30zone	46zone
Moisture content before reinforcement（%）	54.6	49.9	47.1	48.8	41.2	55.5	48.9	48.4
Moisture content after reinforcement（%）	35.5	36.9	35.7	38.3	37	37.7	37.6	36.8
reduce（%）	34.9	25.9	24.2	21.6	10.2	32	23.1	24

Vacuum preloading reinforcement process	Adjacent to the traditional treatment area			without sand cusion test area				
	15zone	30zone	46zone	16zone	31zone	32zone	47zone	48zone
Strength of reinforced cross plate (kPa)	20.2	23.6	24.7	27.3	28.4	26.3	27.5	30.6

5. Case Studies

5.8.3 Economic comparison

According to prices in Tianjin, the secondary treatment of vacuum preloading without sand cushion reduces the cost by 8.91 china yuan per square meter compared to the traditional sand process in 2017.

	name	unit	Traditional craft	Sand free process	Difference	Industry unit price (yuan)	total
1	Shallow drainage board	Linear meter	104290	153797	49507	2	99014
2	Shallow filter tube	Linear meter	7326	24899.2	17573.2	1.8	31631.76
3	Construction period of shallow vacuum preloading	day.10000 square meters	30	40	10	2100	21000
4	Deep drainage board	Linear meter	327045.6	322282.8	-4762.8	2	-9525.6
5	Deep filter tube	Linear meter	6448	12697.6	6249.6	2.8	17498.88
6	black sand	cubic meter	8000	0	-8000	45	-360000
7	medium coarse sand	cubic meter	515.8		-515.8	110	-56738
8	Filling the elevation with dredged silt	cubic meter	0	12000	12000	14	168000
Total (yuan)							-89118.96
Increase in unit price per square meter for sand free process (yuan/square meter)							-8.91


CHEC 中国港湾
Thank you!
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"THE ISLAND AND TUNNEL WORKS OF HONGKONG – ZHUHAI – MACAO BRIDGE PROJECT"

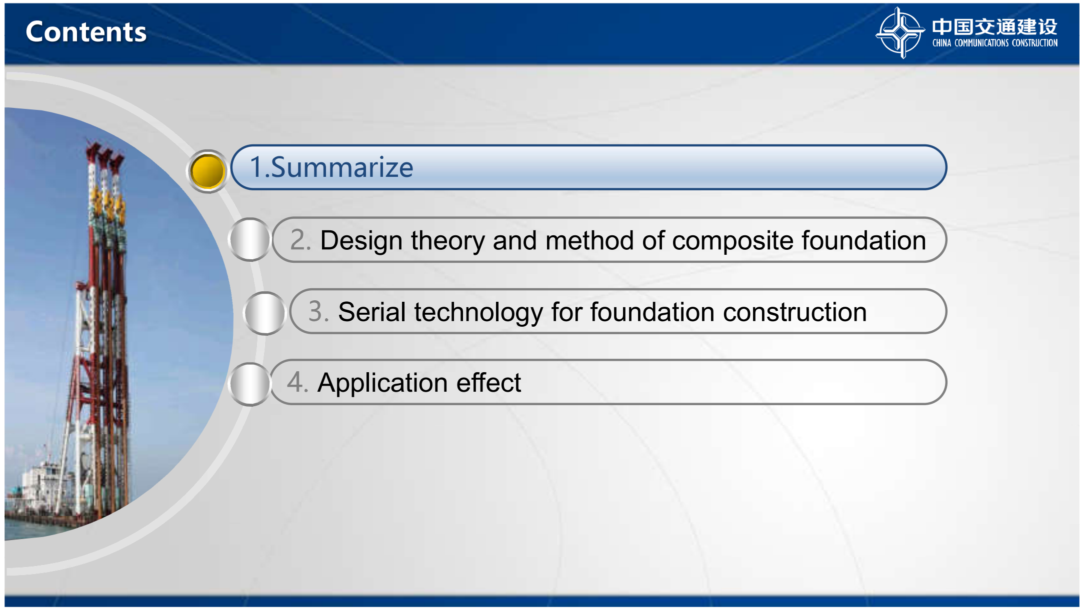
by
Dr. Tang Qunyan,
Geotechnical Expert, CHEC- Sri Lanka Division

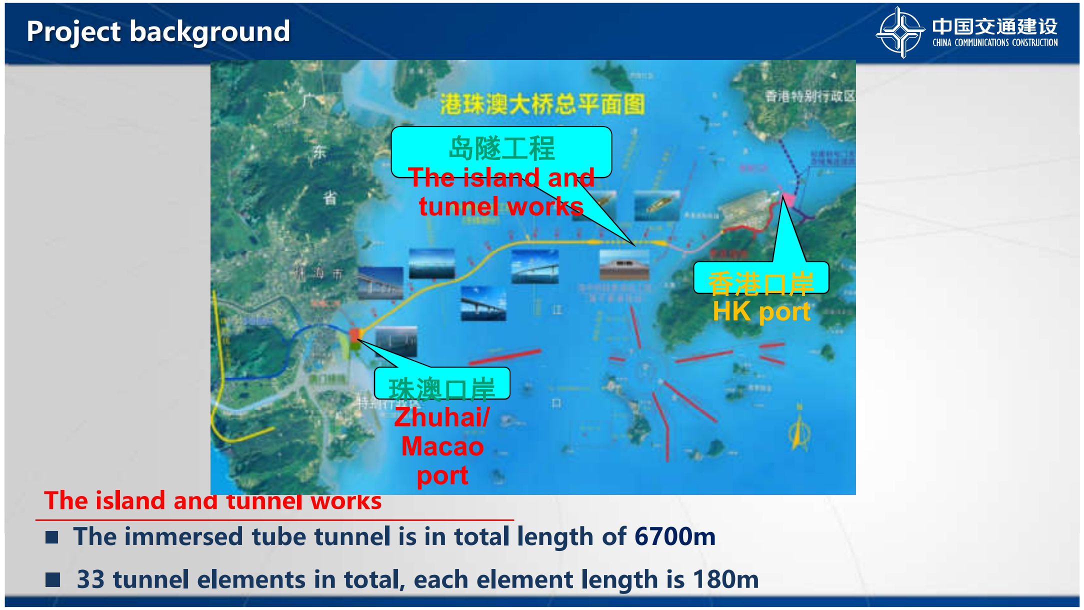
Dr. Tang Qunyan is a registered civil engineer (Geotechnical Engineering) of the People's Republic of China with over 14 years of experience in land reclamation, soft foundation treatment, slope engineering and geological investigation, etc. She graduated from Tongji University at the end of 2009 and obtained her doctorate in 2010. At the end of 2009, she started her work at CCC-FHDI Engineering Co., Ltd and has worked in this company for 14 years. She has participated in hundreds of projects, including dozens of projects as the lead geotechnical designer, dozens of projects as the chief engineer of geological investigation, 4 as the design project manager, 1 as the deputy project manager, and 1 as the survey project manager.

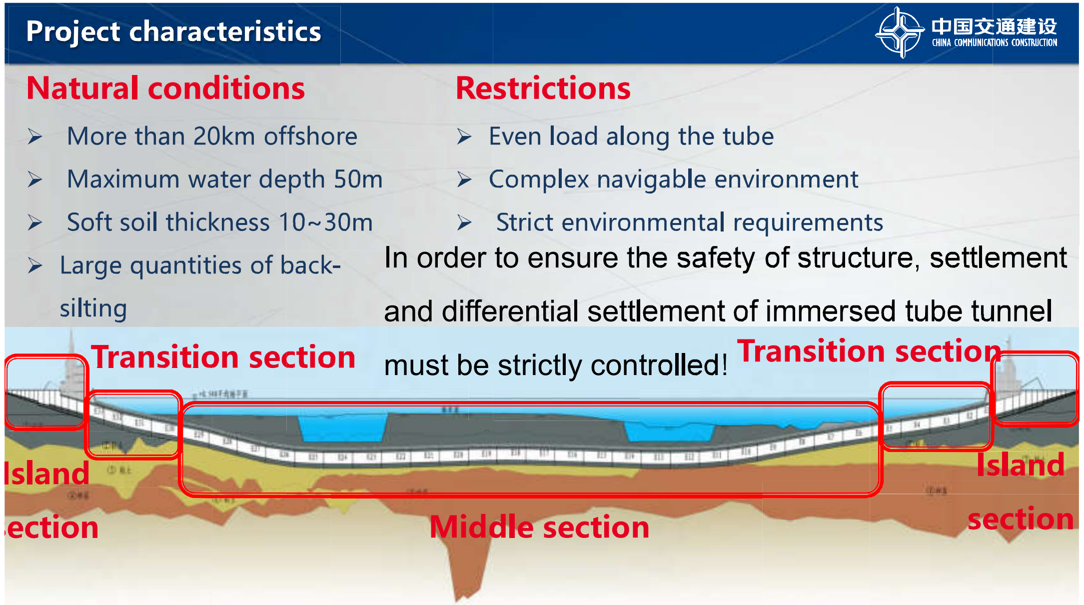
These projects include the design of Sri Lanka's Colombo South Port project, the detailed design of Angola's Lobito Refinery offshore project, the Hong Kong-Zhuhai-Macao Bridge Tunnel and man-made island project, the East Timor container project, the Pasay-SM reclamation project in the Philippines and Kyaukpyu Project in Myanmar, etc.

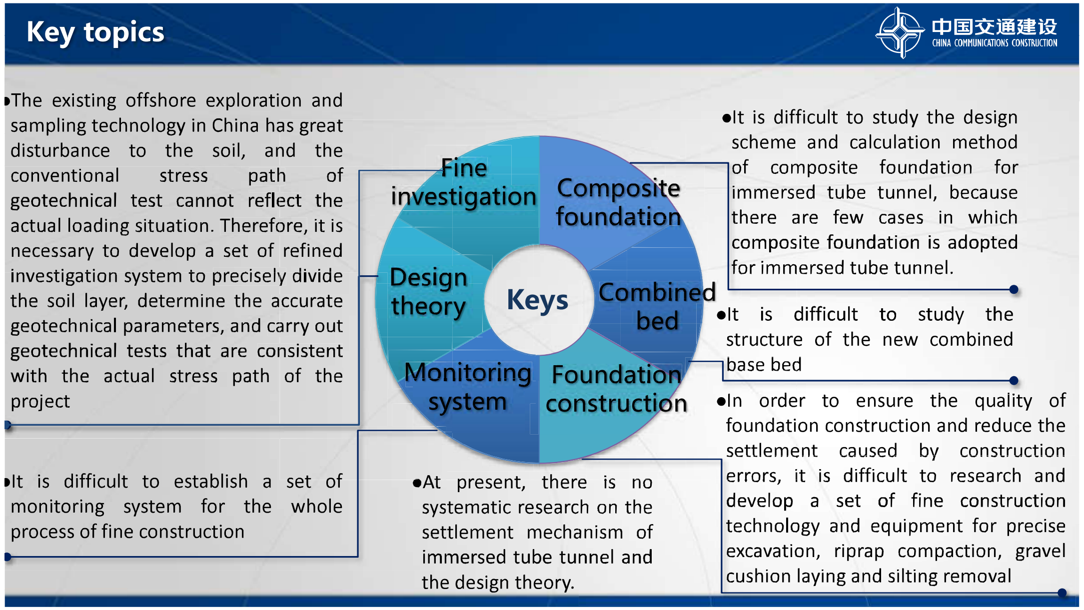
She has published several papers in journals. In 2022, she published her professional book "Guidelines for Geotechnical Investigation of Harbor Engineering" based on domestic and foreign Standards. She also has been invited to participate in several Chinese Standards Writing including the recently published Chinese Standard – *JTS/T 242-2020 Technical Specification for Cone Penetration Testing in Port and Waterway Engineering.*

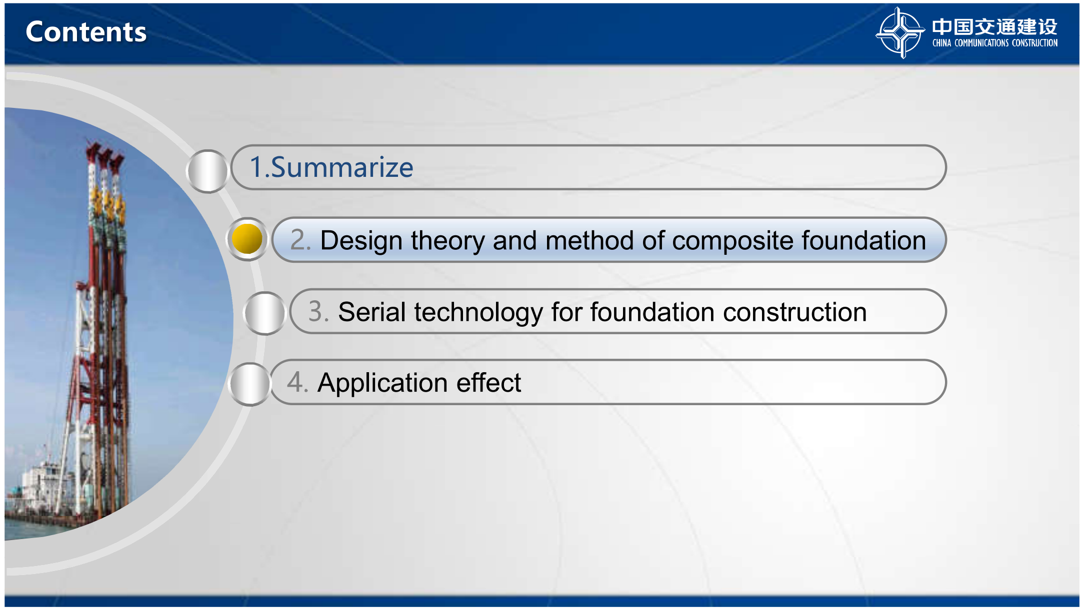

The island and tunnel works of Hongkong-Zhuhai-Macao Bridge Project
Design and construction technology of composite foundation for offshore deep soft soil immersed tube tunnel
Tang Qunyan


Contents
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
1.Summarize
2. Design theory and method of composite foundation
3. Serial technology for foundation construction
4. Application effect


Project background
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
港珠澳大桥总平面图
岛隧工程
The island and tunnel works
香港口岸
HK port
珠澳口岸
Zhuhai/ Macao port
The island and tunnel works
The immersed tube tunnel is in total length of 6700m
33 tunnel elements in total, each element length is 180m


Project characteristics
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Natural conditions
More than 20km offshore
Maximum water depth 50m
Soft soil thickness 10~30m
Large quantities of back-silting
Restrictions
Even load along the tube
Complex navigable environment
Strict environmental requirements
In order to ensure the safety of structure, settlement and differential settlement of immersed tube tunnel must be strictly controlled!
Transition section
Transition section
Island ection
Middle section
Island section


Key topics
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
•The existing offshore exploration and sampling technology in China has great disturbance to the soil, and the conventional stress path of geotechnical test cannot reflect the actual loading situation. Therefore, it is necessary to develop a set of refined investigation system to precisely divide the soil layer, determine the accurate geotechnical parameters, and carry out geotechnical tests that are consistent with the actual stress path of the project
Fine investigation
Composite foundation
Design theory
Keys
Combined bed
Monitoring system
Foundation construction
•It is difficult to study the design scheme and calculation method of composite foundation for immersed tube tunnel, because there are few cases in which composite foundation is adopted for immersed tube tunnel.
•It is difficult to study the structure of the new combined base bed
•In order to ensure the quality of foundation construction and reduce the settlement caused by construction errors, it is difficult to research and develop a set of fine construction technology and equipment for precise excavation, riprap compaction, gravel cushion laying and silting removal
•It is difficult to establish a set of monitoring system for the whole process of fine construction
•At present, there is no systematic research on the settlement mechanism of immersed tube tunnel and the design theory.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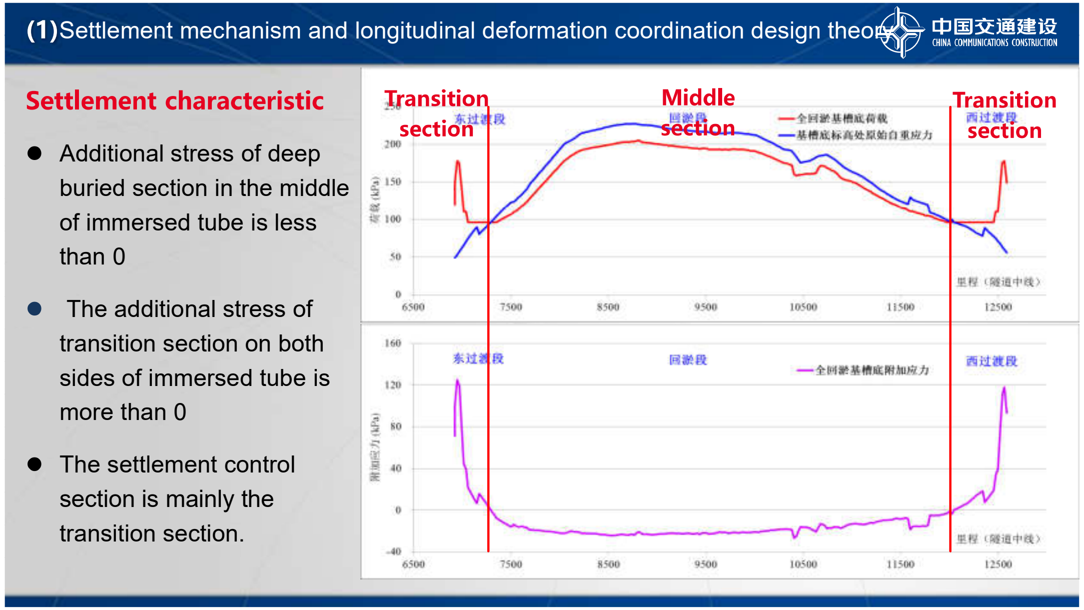
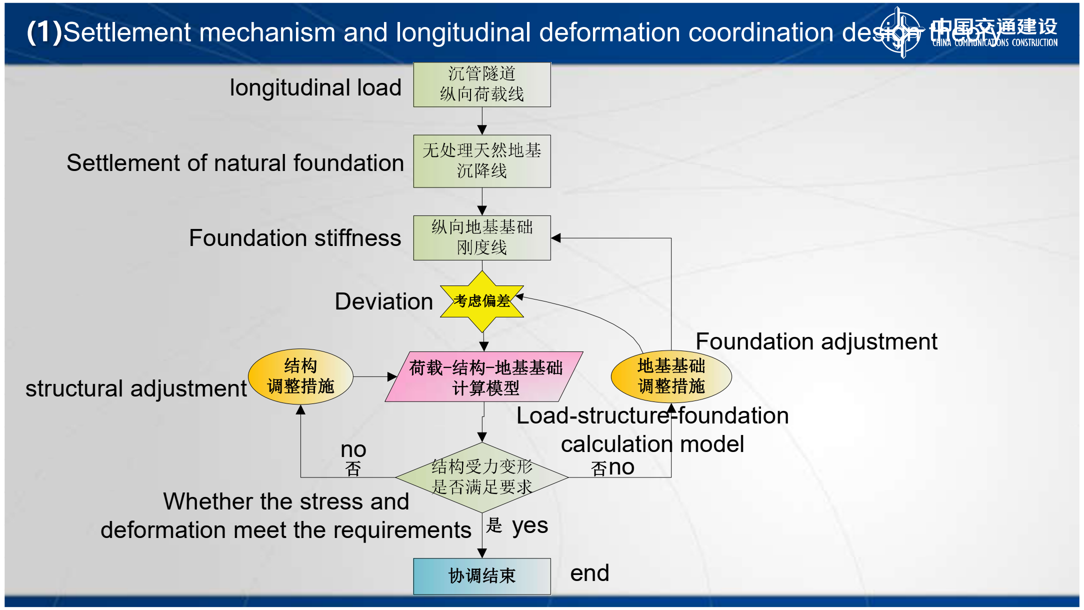
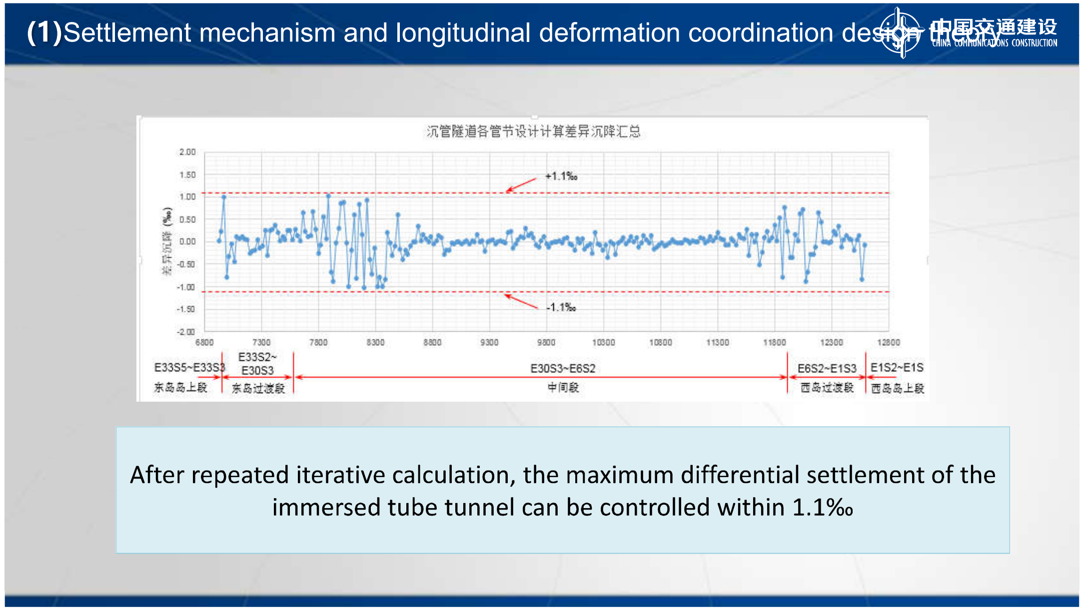
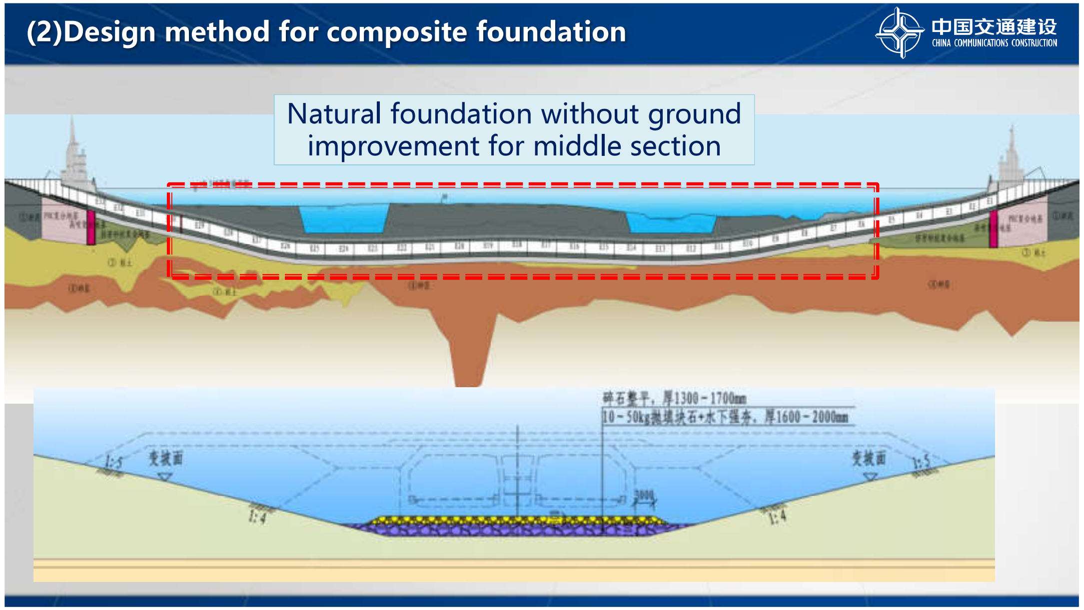
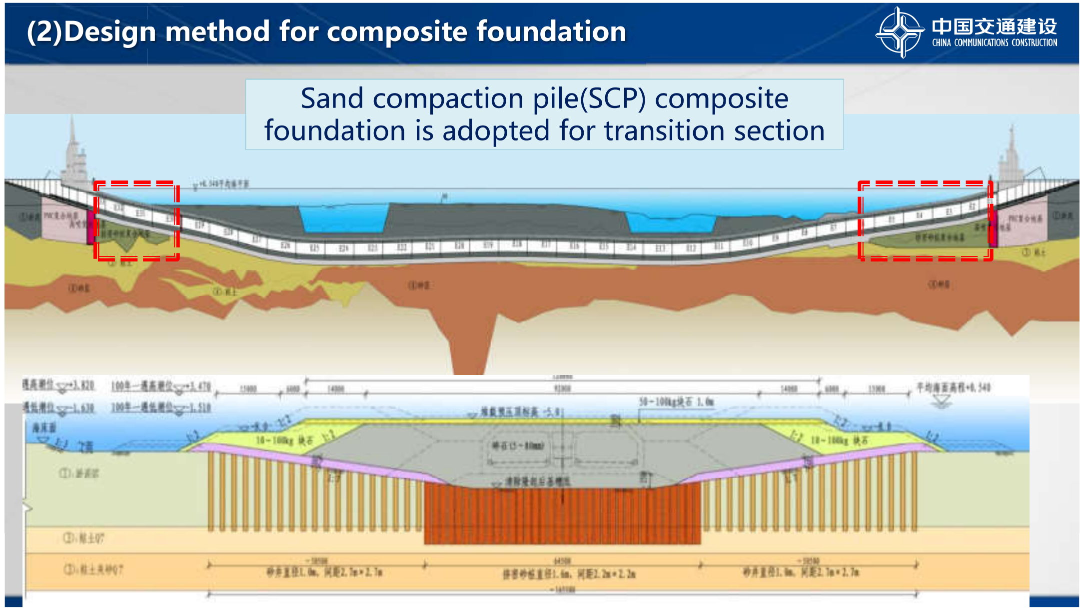
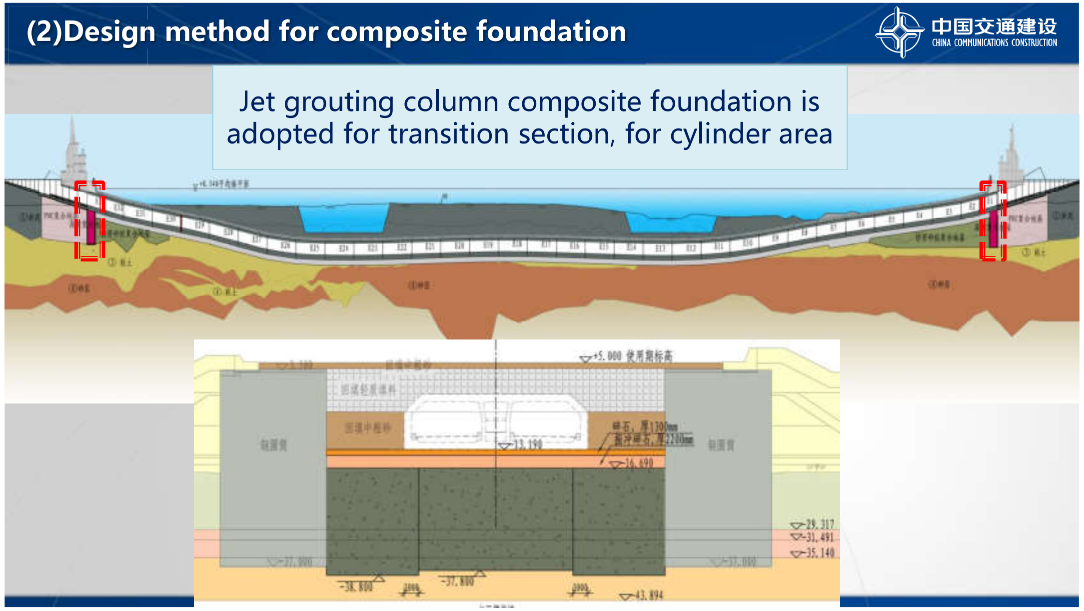

(1)Settlement mechanism and longitudinal deformation coordination design theory
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Settlement characteristic
● Additional stress of deep buried section in the middle of immersed tube is less than 0
● The additional stress of transition section on both sides of immersed tube is more than 0
● The settlement control section is mainly the transition section.
Transition section
东过渡段
Middle section
回淤段
Transition section
西过渡段
全回淤基槽底荷载
基槽底标高处原始自重应力
全回淤基槽底附加应力
里程（隧道中线）
6500
7500
8500
9500
10500
11500
12500


(1)Settlement mechanism and longitudinal deformation coordination design theory
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
longitudinal load
沉管隧道纵向荷载线
Settlement of natural foundation
无处理天然地基沉降线
Foundation stiffness
纵向地基基础刚度线
Deviation
考虑偏差
structural adjustment
结构调整措施
荷载-结构-地基基础计算模型
地基基础调整措施
Foundation adjustment
Load-structure-foundation calculation model
no
否
结构受力变形是否满足要求
否no
Whether the stress and deformation meet the requirements
是 yes
协调结束
end


(1)Settlement mechanism and longitudinal deformation coordination design theory
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
沉管隧道各管节设计计算差异沉降汇总
+1.1‰
-1.1‰
E33S5~E33S3
E33S2~E30S3
E30S3~E6S2
E6S2~E1S3
E1S2~E1S
东岛岛上段
东岛过渡段
中间段
西岛过渡段
西岛岛上段
After repeated iterative calculation, the maximum differential settlement of the immersed tube tunnel can be controlled within 1.1‰

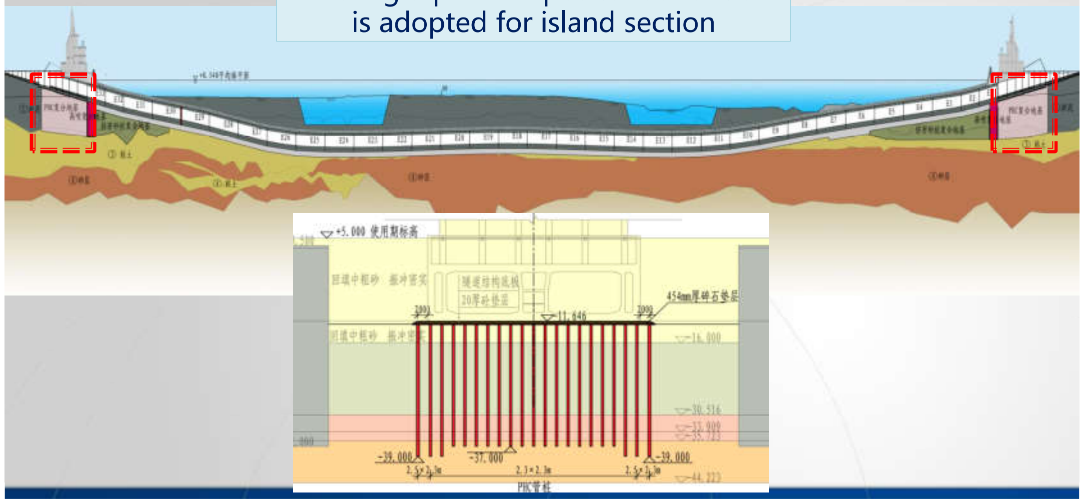

(2)Design method for composite foundation
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Natural foundation without ground improvement for middle section


(2)Design method for composite foundation
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Sand compaction pile(SCP) composite foundation is adopted for transition section


(2)Design method for composite foundation
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Jet grouting column composite foundation is adopted for transition section, for cylinder area

(2)Design method for composite foundation

PHC Rigid pile composite foundation is adopted for island section



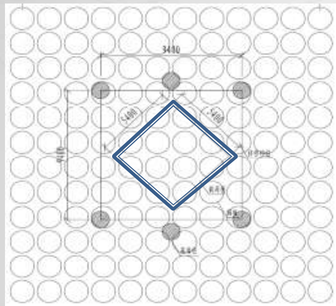
(2)Design method for composite foundation

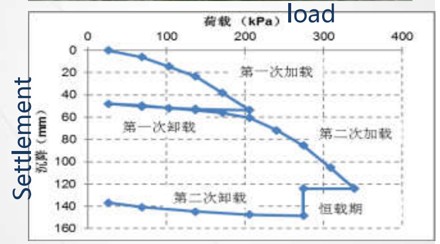
Plate loading test for SCP composite foundation

Layout

- Replacement ratio:60%
- Size

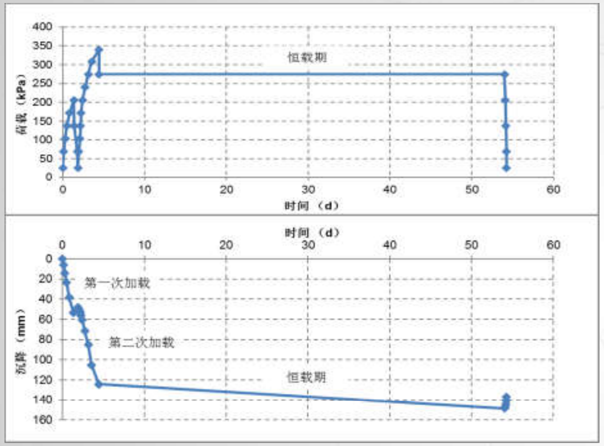
  5.4m×5.4m





(2)Design method for composite foundation

Plate loading test for SCP composite foundation

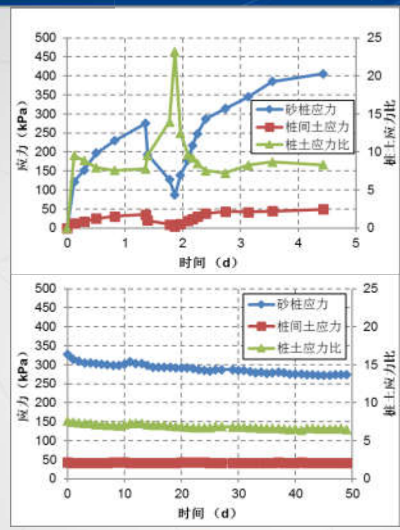


(2)Design method for composite foundation

Plate loading test for SCP composite foundation

Result

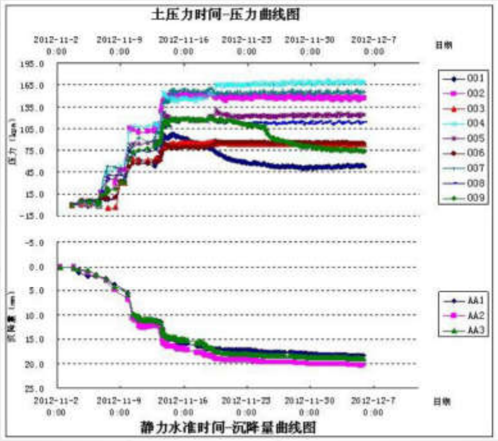
- The pile-soil stress ratio of 6~8 gradually decreases with time
- The pile-soil stress ratio increases suddenly at unloading
- When the pile-soil stress ratio is 7.5, the calculated value of settlement is close to the theoretical value with a deviation of 6%



(2)Design method for composite foundation

中国交通建设 CHINA COMMUNICATIONS CONSTRUCTION

Plate load test for jet grouting composite foundation



(2)Design method for composite foundation

中国交通建设 CHINA COMMUNICATIONS CONSTRUCTION

Plate load test for PHC rigid pile composite foundation

Layout

- The loading tests of natural foundation, single pile composite foundation and multi-pile composite foundation were carried out
- Multi-pile loading tests with 30cm, 50cm and 70cm gravel cushion thickness were carried out respectively

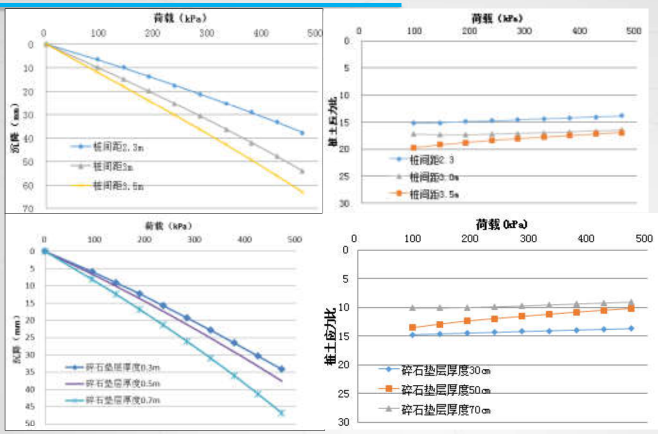
(2)Design method for composite foundation

中国交通建设 CHINA COMMUNICATIONS CONSTRUCTION

Plate load test for PHC rigid pile composite foundation

Result

- Settlement and pile-soil stress ratio increase with the increase of pile spacing
- The settlement increases and the pile-soil stress ratio decreases with the increase of cushion thickness



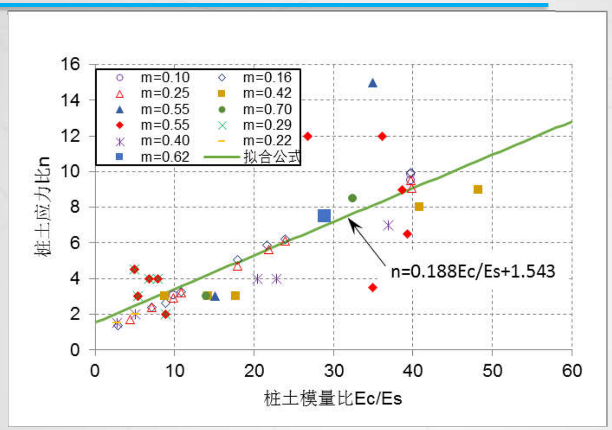
(2)Design method for composite foundation

中国交通建设 CHINA COMMUNICATIONS CONSTRUCTION

Settlement calculation of sand compacted pile composite foundation

- It is recommended to use stress correction method to calculate the settlement of sand compacted pile composite foundation, and the formula of pile-soil stress ratio in settlement calculation is proposed for the first time.

$$n=0.188E_c/E_s+1.543$$

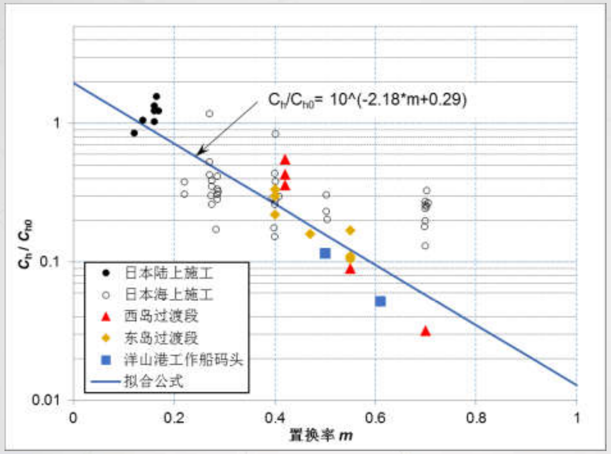


(2)Design method for composite foundation

Settlement calculation of sand compacted pile composite foundation

- The calculation method of consolidation delay of sand compacted pile is put forward, and the following consolidation delay calculation formula is proposed

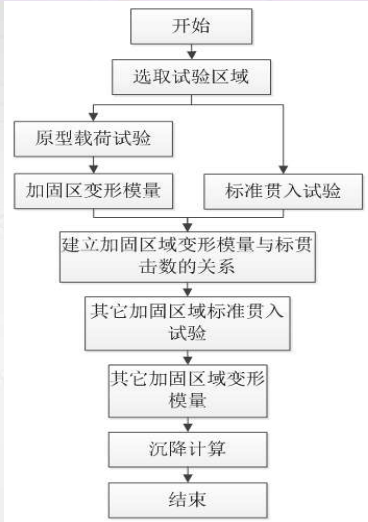
C_h/C_{h0}= 10^(-2.18*m+0.29)



(2)Design method for composite foundation

Settlement calculation for jet grouting composite foundation

For important projects with strict settlement control, it is recommended to use preloading test and standard penetration test for settlement calculation.

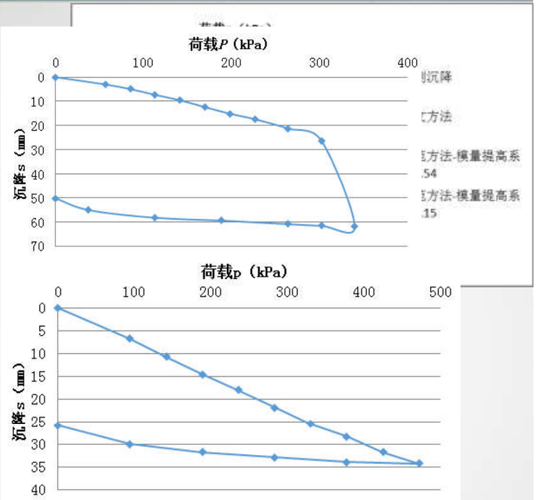


(2)Design method for composite foundation

Settlement calculation for PHC composite foundation

A new method for calculating the increase coefficient of composite modulus is presented.

$$E_{sp} = \xi E_s = \frac{f_{spk}}{f_{sk}} E_s \qquad \xi = \frac{f_{spk}}{f_{sk}} = \frac{E_{spk}}{E_{sk}} = \frac{p_{spk} b_{spk} / s_{spk}}{p_{sk} b_{sk} / s_{sk}}$$



(3)Combined bed

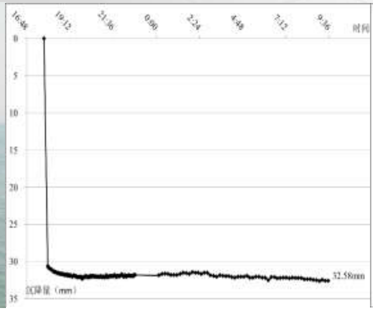
Trials for stone cushion

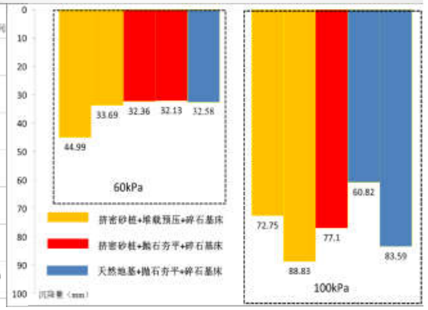
- From December 2010 to November 2011, 25 tests were carried out in five batches to study the size, grading, fine content and sensitivity of the gravel layer.
- The settlement was basically stable after loading, and the deformation was mainly caused by compaction deformation and lateral deformation.

(3)Combined bed

Plate load test underwater

- The test site is located at E4, E6 and E12 element of immersed tube tunnel .
- 95% settlement completed in the first 15 minutes, and the instantaneous settlement is directly related to the load 。

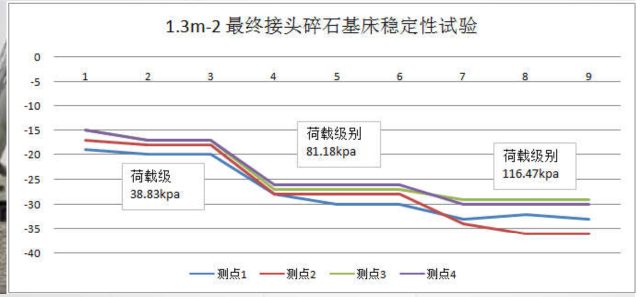




(3)Combined bed

Plate load test

- From January 2017 to February 2017, nine groups of 36 onshore foundation bed load plate tests were.
- Settlement is mainly composed of instantaneous settlement.

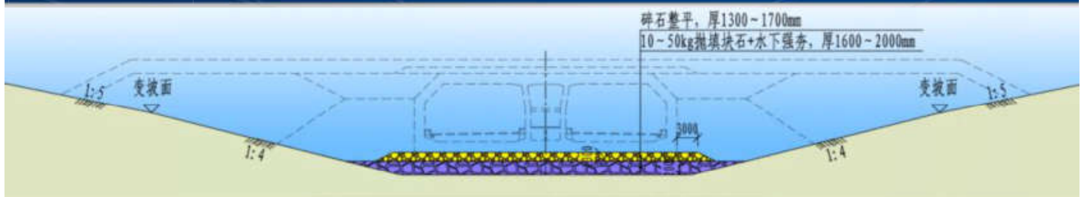


(3)Combined bed

Model tests for sediment carring capacity

- There is a close relationship between the sediment carrying capacity of gravel layer and the thickness of gravel layer, gradation and groove size.

(3)Combined bed



Composition of combined bed

- Crushed stone 1.3m,rock block 2.0m
- The top wide of groove is 1.8m, the wide of groove is 1.05m
- 10~100kg block stone, compaction ratio of 10%~15%, compaction time of 45~55s

(3)Combined bed

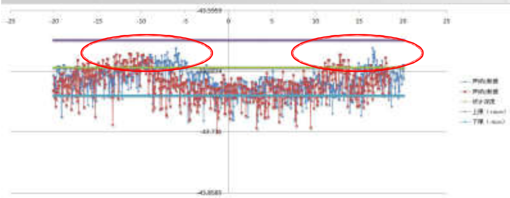
Benefits

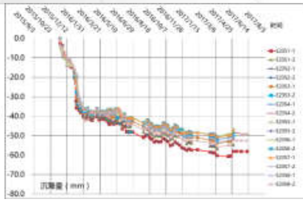
- Improves the settlement coordination ability
- Improves the fault tolerance of excavation
- The rock block layer provides the ability of sedimentation, and the crushed stone layer creates better conditions for sediment removal

(3)Combined bed

Research results

- The settlement is mainly composed of short-term instantaneous settlement of crushed stone layer.
- The instantaneous settlement of the crushed stone layer is composed of two parts, one is the compression of the average design elevation when the immersed tube is installed in place, and the other is the short-term compression settlement of the crushed stone layer in the corresponding loading stage.





(4)Fine investigation

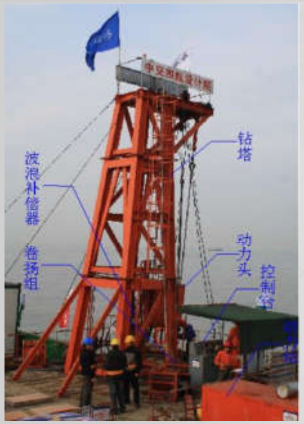
hydraulic lifting drilling platform

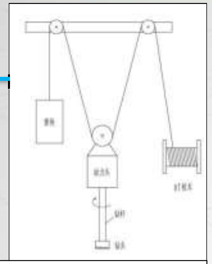
Innovation

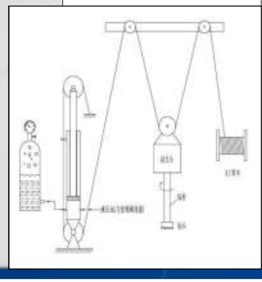
- Easy for transit shipping
- Carry out offshore
- Maximum working water depth of 18.0m

(4)Fine investigation

Separate hydraulic drill with wave compensation





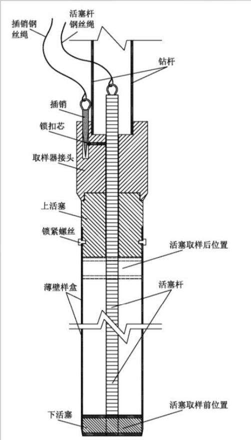


Innovation and application

- Wave compensation
- High drilling quality
- High core recovery rate
- Take the level I undisturbed sample

(4)Fine investigation

New type thin wall sampler with fixed piston



Innovation and application

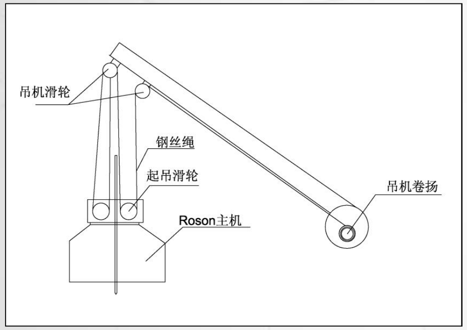
- Wire rope replaced the piston rod to solve the problem of piston rod easily broken
- Take the level I undisturbed sample

(4)Fine investigation

Improved CPTU

Innovation

- Drill rod can be added continuously during test by changing lifting mode

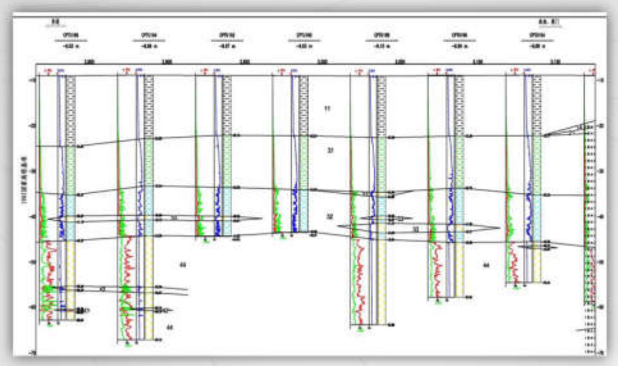


(4)Fine investigation

Improved CPTU

Application

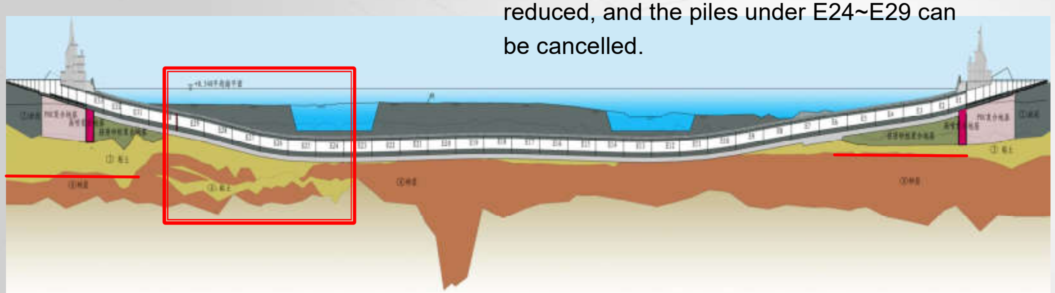
- The penetration depth is large
- Define soil layers accurately
- CPTU formula suitable for the pearl river is established

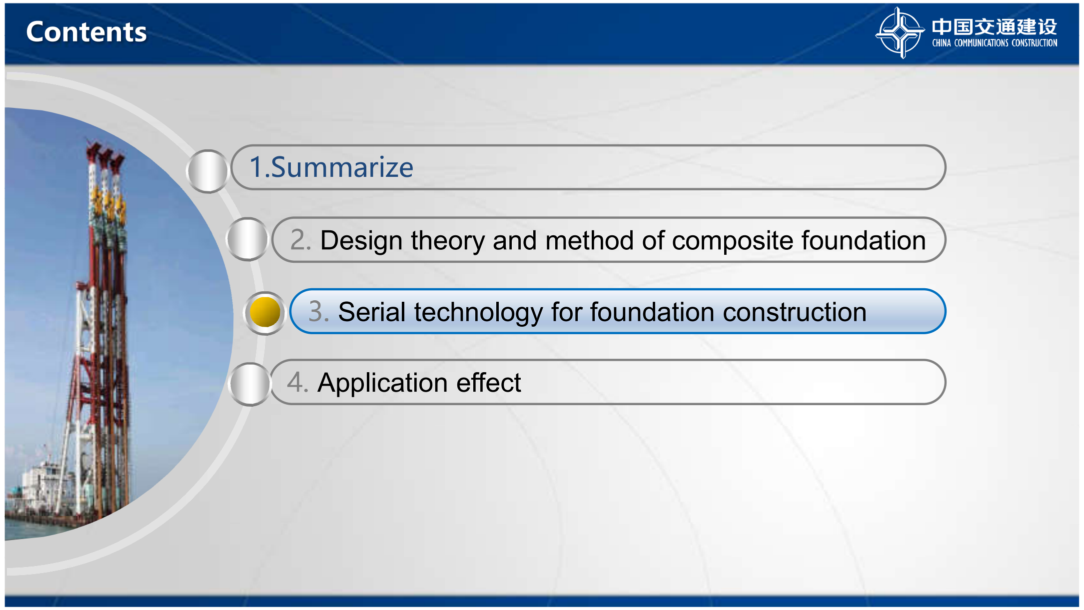


(4)Fine investigation

Outstanding contribution

- The compression modulus of the soil increased by 20%~30%, and the OCR of the underlying main compression layer reached 2.16. The calculation shows that the depth of foundation treatment can be reduced, and the piles under E24~E29 can be cancelled.




Contents
中国交通建设
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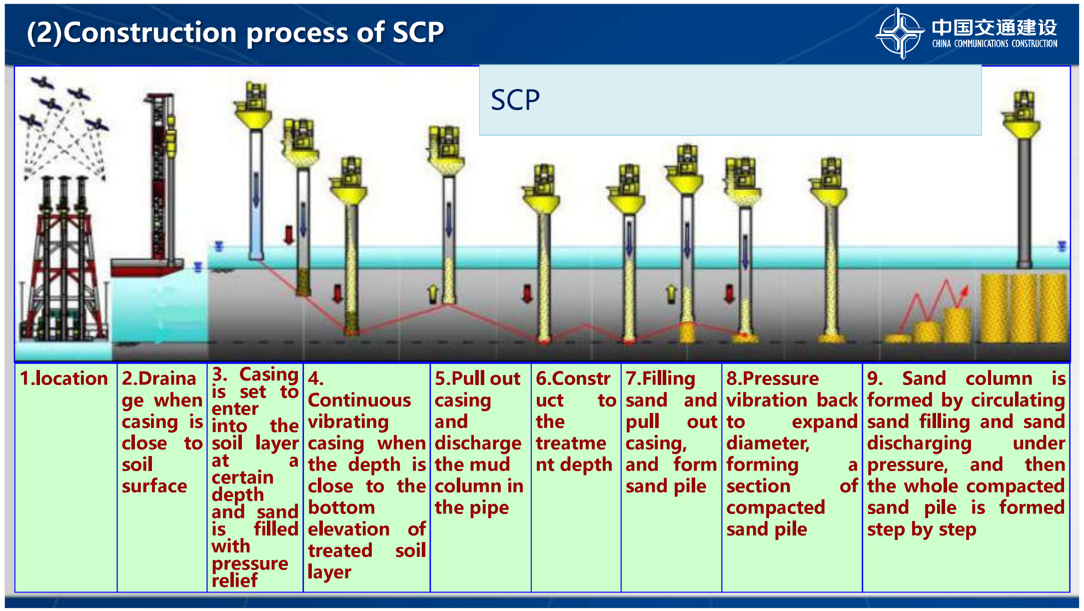
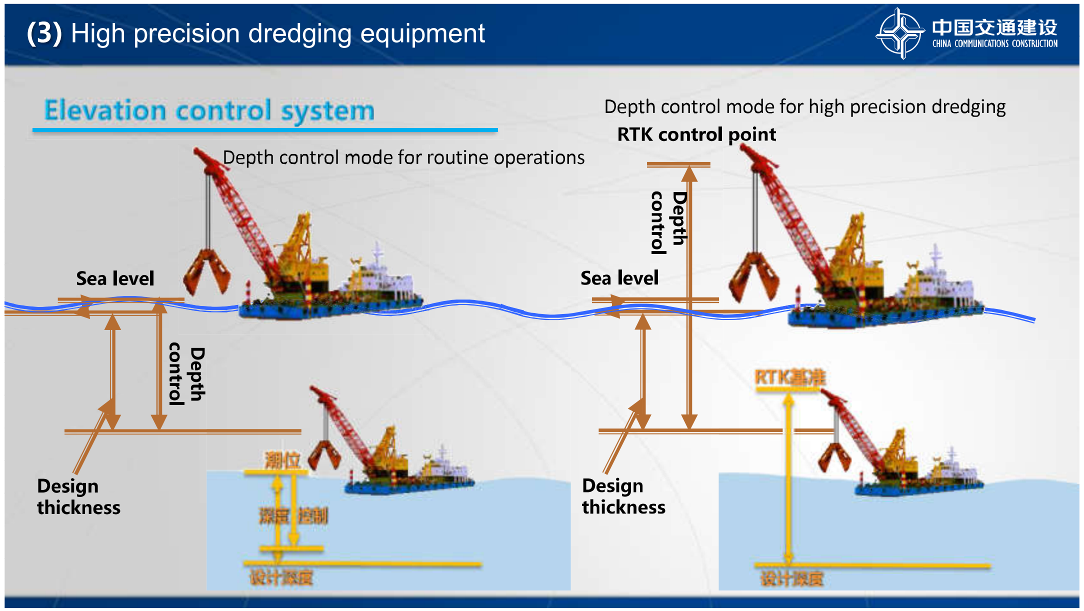
2. Design theory and method of composite foundation
3. Serial technology for foundation construction
4. Application effect


(1) Construction technology
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Construction process—composite foundation（SCP/Jet grouting/PHC）
PHC
Jet grouting
SCP

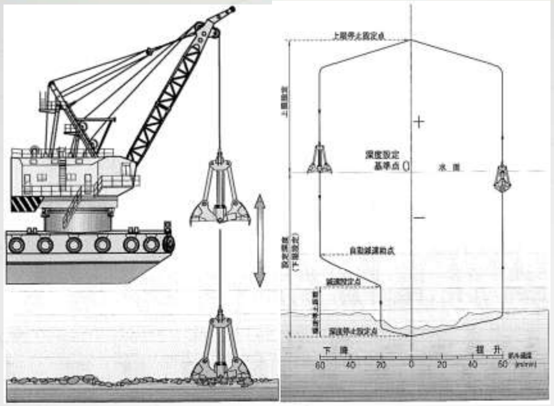

(2)Construction process of SCP
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
SCP
1.location
2.Drainage when casing is close to soil surface
3. Casing is set to enter into the soil layer at a certain depth and sand is filled with pressure relief
4. Continuous vibrating casing when the depth is close to the bottom elevation of treated soil layer
5.Pull out casing and discharge the mud column in the pipe
6.Construct the treatment depth
7.Filling sand and pull out casing, and form sand pile
8.Pressure vibration back to expand diameter, forming a section of compacted sand pile
9. Sand column is formed by circulating sand filling and sand discharging under pressure, and then the whole compacted sand pile is formed step by step


(3) High precision dredging equipment
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Elevation control system
Depth control mode for high precision dredging
RTK control point
Depth control mode for routine operations
Depth control
Sea level
Depth control
Sea level
Design thickness
Design thickness

(3) High precision dredging equipment

Elevation control system

- The application of RTK can avoid the influence of waves
- Height positioning accuracy can reach the centimeter level



Monitoring system

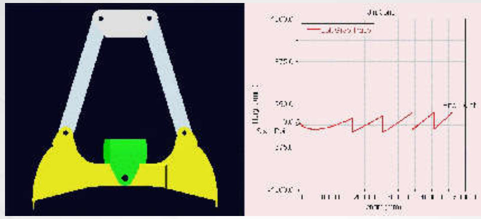
- The direct digital control system (DDC) is developed to realize the accurate movement control of the grab

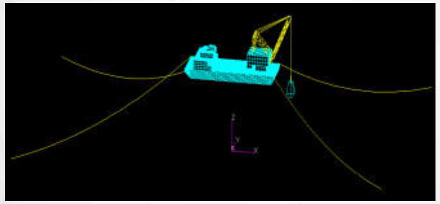
(3) High precision dredging equipment

Grab dredger equipment renovation

The dredging height difference was reduced from 30cm to 5cm

- Deep system optimization
- Excavation technical renovation
- Anchoring system optimization

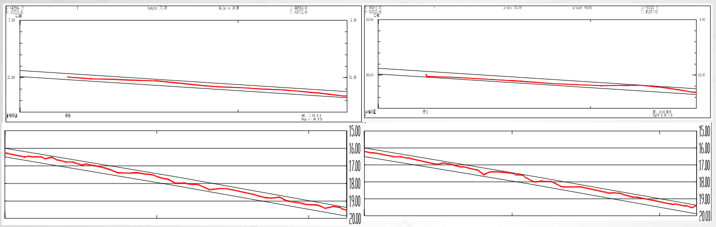




(3) High precision dredging equipment

Result

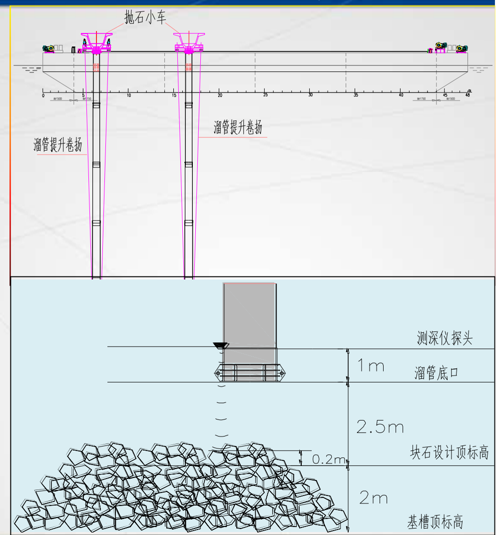
Maximum over dredging depth of grab dredger should not be more than 0.5m.



(4)Key technology of dumped rock and tamping

Dumped rock system

- The trunk suitable for water depth of 16~52m is developed
- The bottom of the trunk is equipped with a depth sounder for real-time monitoring



(4)Key technology of dumped rock and tamping

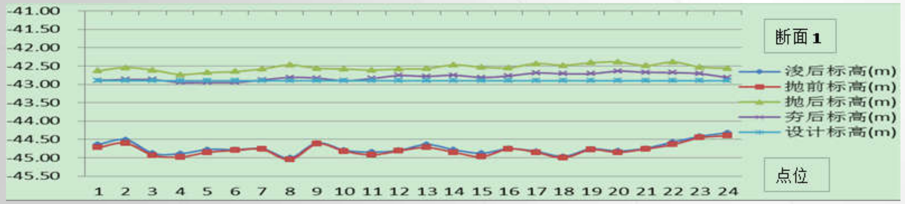
Tamping system

- The hydraulic vibratory hammer was creatively used to tamp the rock foundation bed
- The length of rammer plate is 5m,and the width is 4m, the tamping area of a rammer can reach 20m^2, and continuous tamping 30 seconds can achieve the ramming effect of 12 times of common ramming

(4)Key technology of dumped rock and tamping

Result

- The elevation deviation of the tamping rock stone is 30cm



(5)Key technology of crushed stone bed

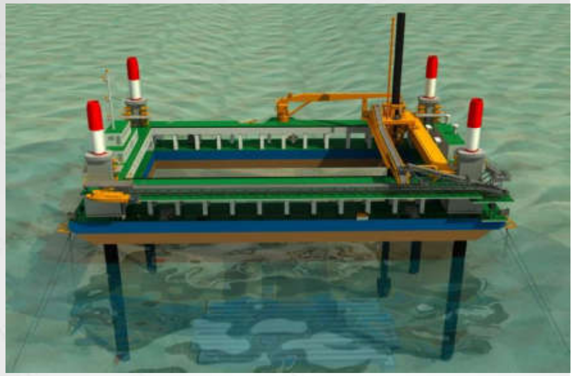
Key technology of crushed stone bed

- Lifting system: 4 pile legs and 32 lifting mechanisms realize simultaneous lifting of the hull
- Walking system: walking big car and small car to form a ridge and furrow type bed leveling

(5)Key technology of crushed stone bed

Key technology of crushed stone bed

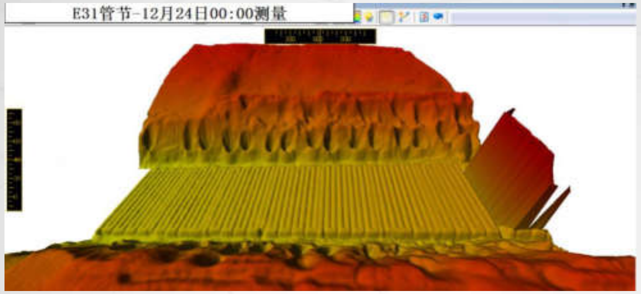
- Feeding system: composed of 3 belt conveyors to realize the supply of crushed stone
- Measurement and control system: it is composed of 5 sets of GPS and 1 set of hydraulic cylinder system to realize accurate positioning and elevation control of the ship



(5)Key technology of crushed stone bed

Key technology of crushed stone bed

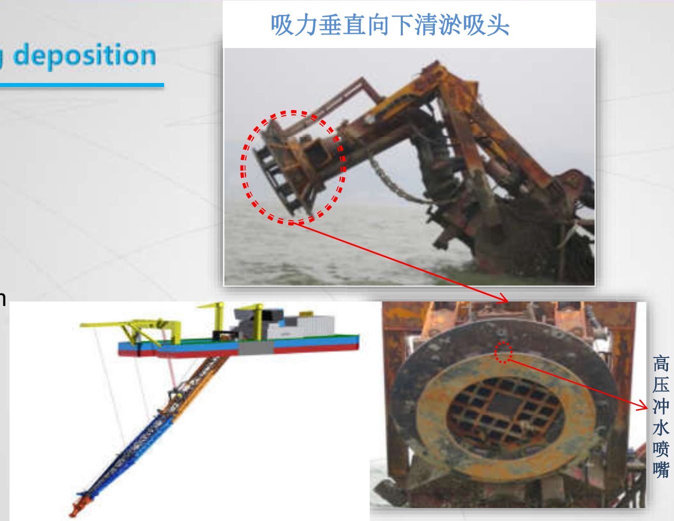
Verifying the result by sonar and multi-beam, it was finally confirmed that the foundation bed leveling construction of ± 3cm was achieved



(6)Key technology of crushed stone bed

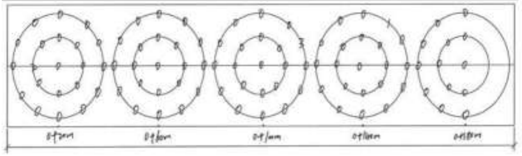
Key techniques for removing deposition

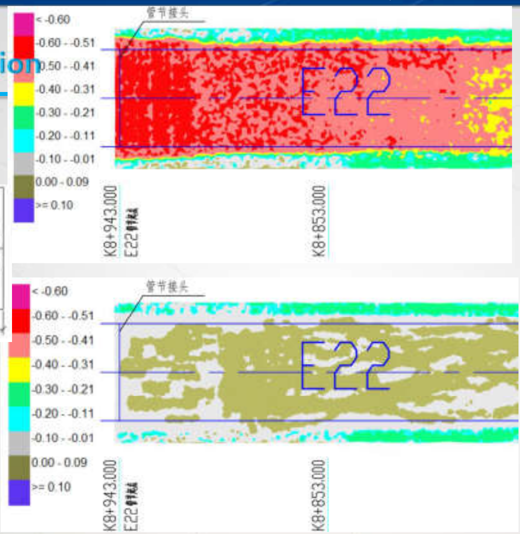
- A new multifunctional suction device was developed
- The bridge of 80m long rigid truss structure is developed
- A high precision monitor system for the positioning of suction head is developed
- The construction technology of removing deposition for fixed location is developed



(6)Key technology of crushed stone bed

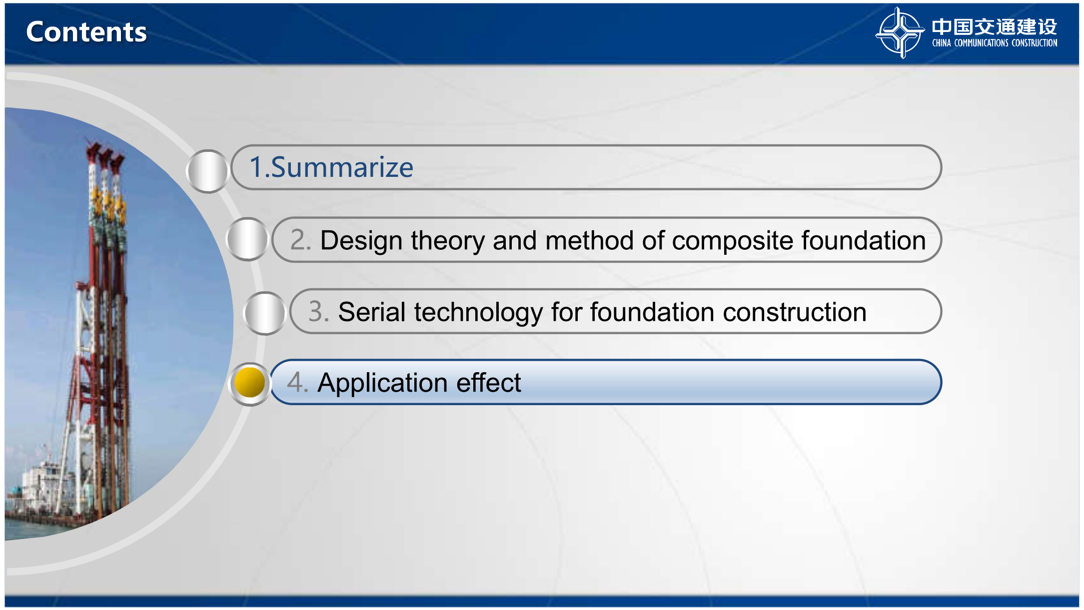
Key techniques for removing deposition





(7)Serial technology for foundation construction

Construction proceed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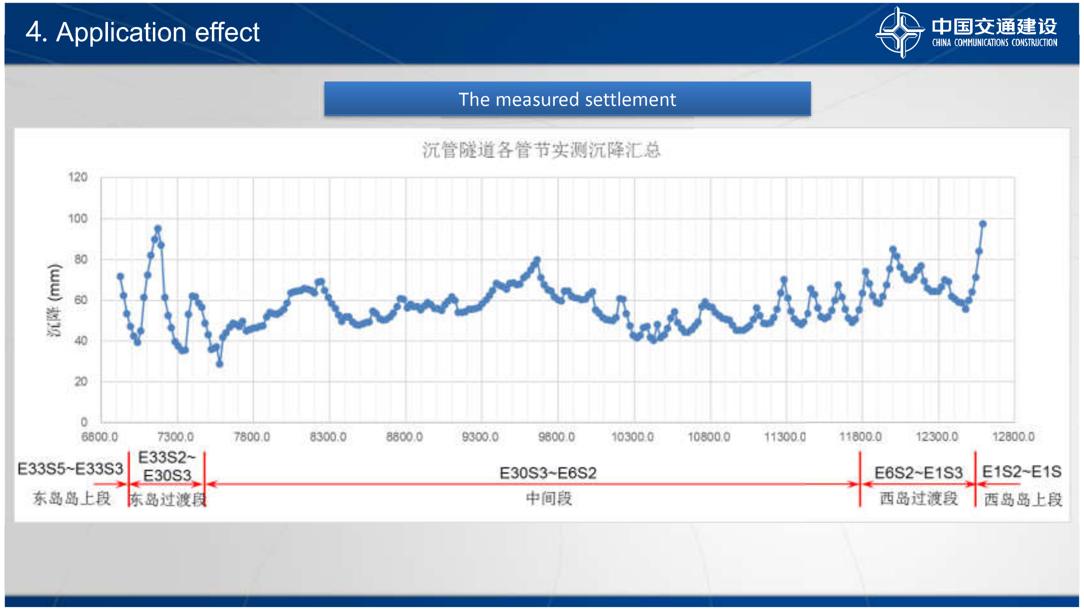
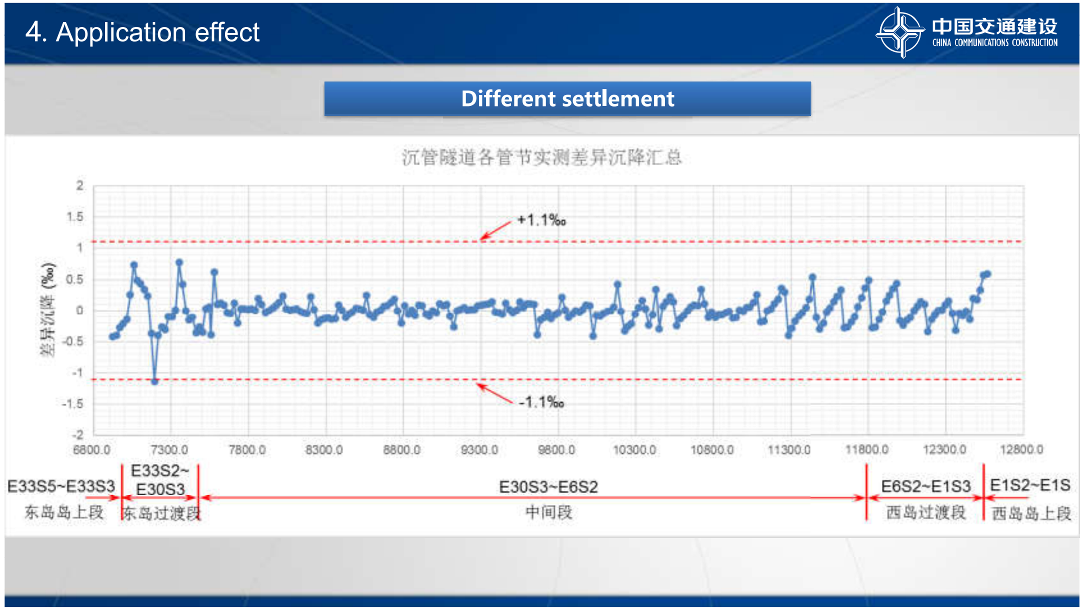
4. Application effect
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
The new composite bed structure was applied for the first time
The excavation deviation is 50cm
The leveling deviation of deep-water tamping is 30cm
Crushed stone leveling control 3.0cm
Deep water dredging at fixed location


4. Application effect
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
The measured settlement
沉管隧道各管节实测沉降汇总
沉降 (mm)
120
100
80
60
40
20
0
6800.0
7300.0
7800.0
8300.0
8800.0
9300.0
9800.0
10300.0
10800.0
11300.0
11800.0
12300.0
12800.0
E33S5~E33S3
东岛岛上段
E33S2~E30S3
东岛过渡段
E30S3~E6S2
中间段
E6S2~E1S3
西岛过渡段
E1S2~E1S
西岛岛上段


4. Application effect
中国交通建设
CHINA COMMUNICATIONS CONSTRUCTION
Different settlement
沉管隧道各管节实测差异沉降汇总
差异沉降 (‰)
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
+1.1‰
-1.1‰
6800.0
7300.0
7800.0
8300.0
8800.0
9300.0
9800.0
10300.0
10800.0
11300.0
11800.0
12300.0
12800.0
E33S5~E33S3
东岛岛上段
E33S2~E30S3
东岛过渡段
E30S3~E6S2
中间段
E6S2~E1S3
西岛过渡段
E1S2~E1S
西岛岛上段

4. Application effect

No leakage

Intellectual property

item	Patent for invention	Patent for utility models	National construction method	paper	criterion
quantities	9	23	1	41	1

谢　谢

THANKS

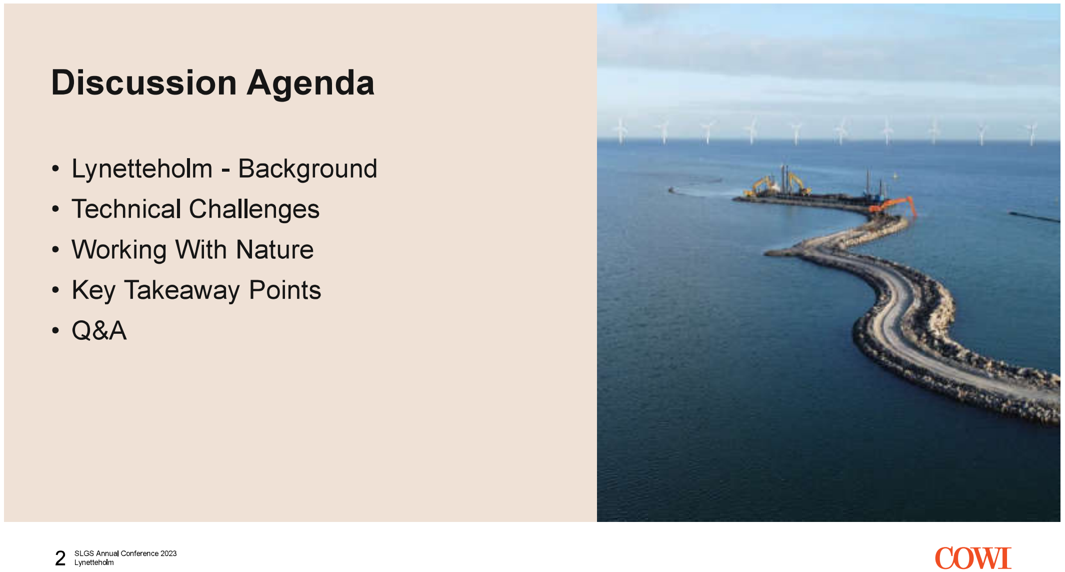
“COASTAL RESILIENCE THROUGH URBAN EXPANSION: LYNETTEHOLM CASE STUDY”

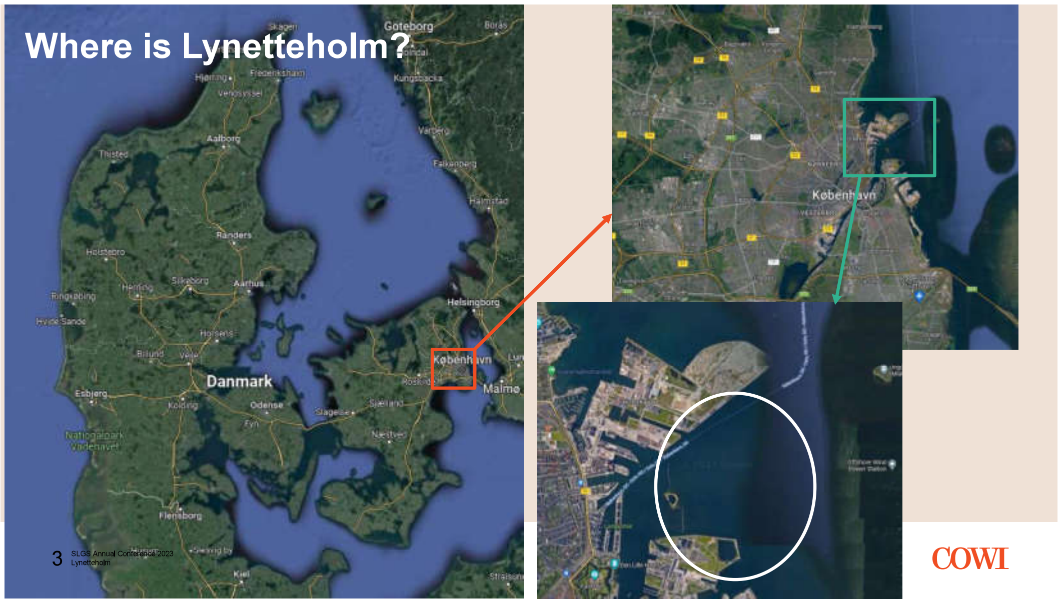
by
Ir. Sridhar Krishnan,
Technical Director, Marine and Coastal Engineering,
COWI Singapore

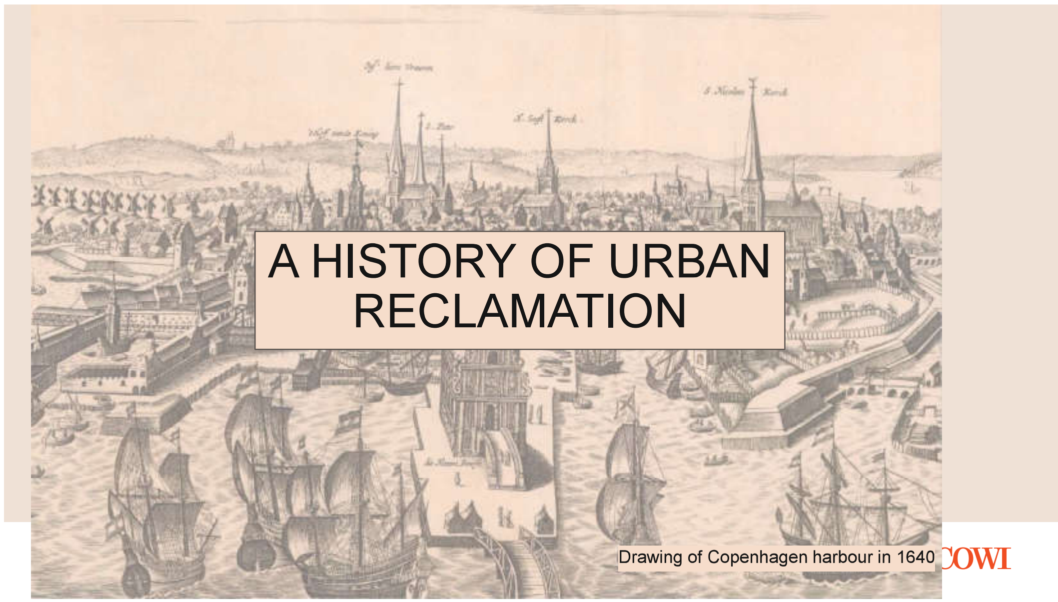
Ir. Sridhar Krishnan is a Registered Professional Civil Engineer and Certified Project Management Professional with over 30 years of experience in designing, leading, and delivering civil, structural, geotechnical, and maritime engineering services for a diverse portfolio of projects in the transport, energy, mining, water resources, real estate and infrastructure sectors. These projects, spanning across the Asia-Pacific, Middle East, Africa, the Americas, and Europe, include ports and terminals, coastal and maritime structures, power plants, dams, highways, railways, bridges, airports, and residential, commercial, retail, industrial and mixed-use developments. Through these projects, Ir. Krishnan has worked closely with a wide range of clients, from project proponents to turnkey contractors, consulting engineers, and government agencies. He has published papers in peer-reviewed journals, made keynote presentations at international conferences, and delivered invited lectures to learned societies.

Ir. Krishnan has served as a member of the technical divisions of professional organizations and participated in the drafting of technical standards, guidelines, and specifications for industry use. He has also served on panels of engineering experts for professional organizations and project boards.


Coastal Resilience Through Urban Expansion: Lynetteholm Case Study
"Loon-net-eh-hom"
1
SLGS Annual Conference 2023
Lynetteholm
COWI


Discussion Agenda
• Lynetteholm - Background
• Technical Challenges
• Working With Nature
• Key Takeaway Points
• Q&A
2
SLGS Annual Conference 2023
Lynetteholm
COWI


Where is Lynetteholm?
Danmark
København
Malmö
3
SLGS Annual Conference 2023
Lynetteholm
COWI


A HISTORY OF URBAN RECLAMATION
Drawing of Copenhagen harbour in 1640
COWI

Development of Copenhagen through reclamation

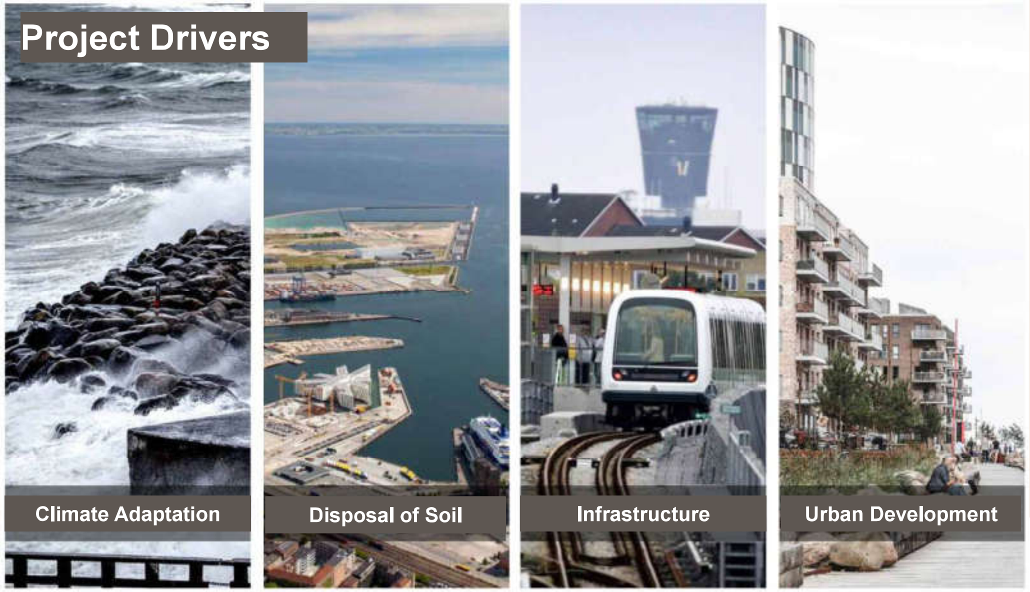
Industrial and Commercial Port (1900)

Urban Expansion (late 1900s)

Climate Adaptation (present)

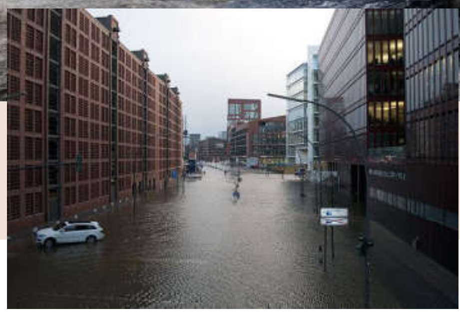
COWI

Project Drivers

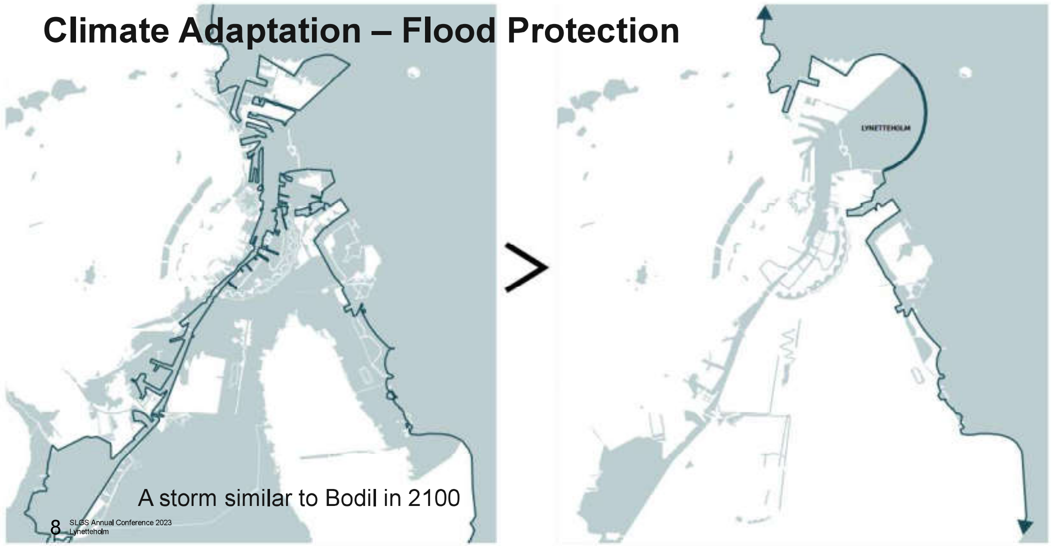


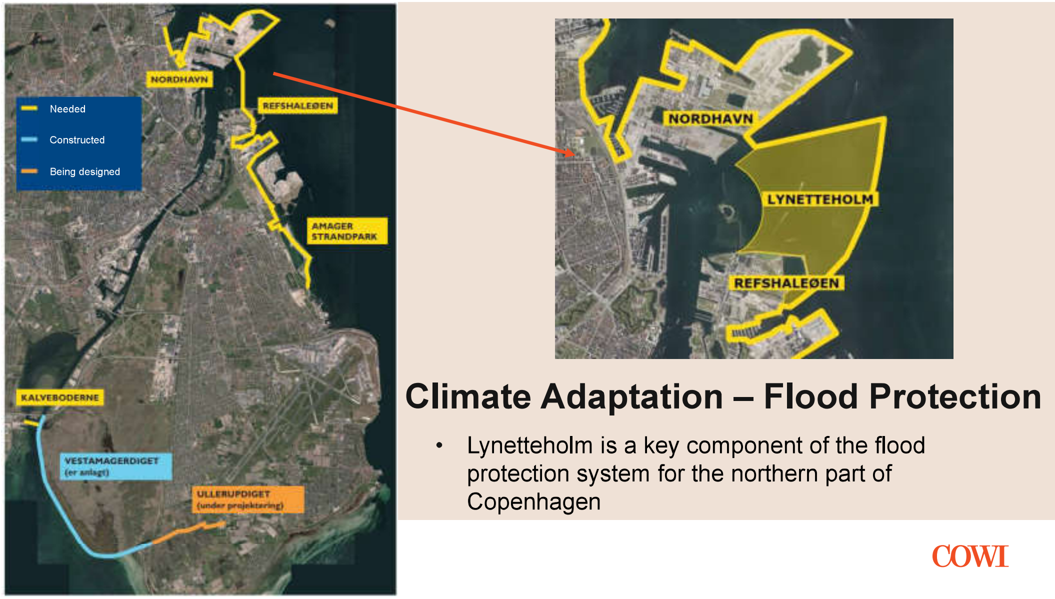
The storm "Bodil"

- Hit Denmark 5th of December 2013
- Mean wind speed of 36.6 m/s (~200 YRP)
- Gust wind speed of 44.2 m/s
- Storm surge in inner Danish waters (~200 YRP in Copenhagen)
  - Maximum of 2.06 m MSL
  - Copenhagen 1.72 m MSL (highest ever recorded)
- Flood damage of 140 mio. USD

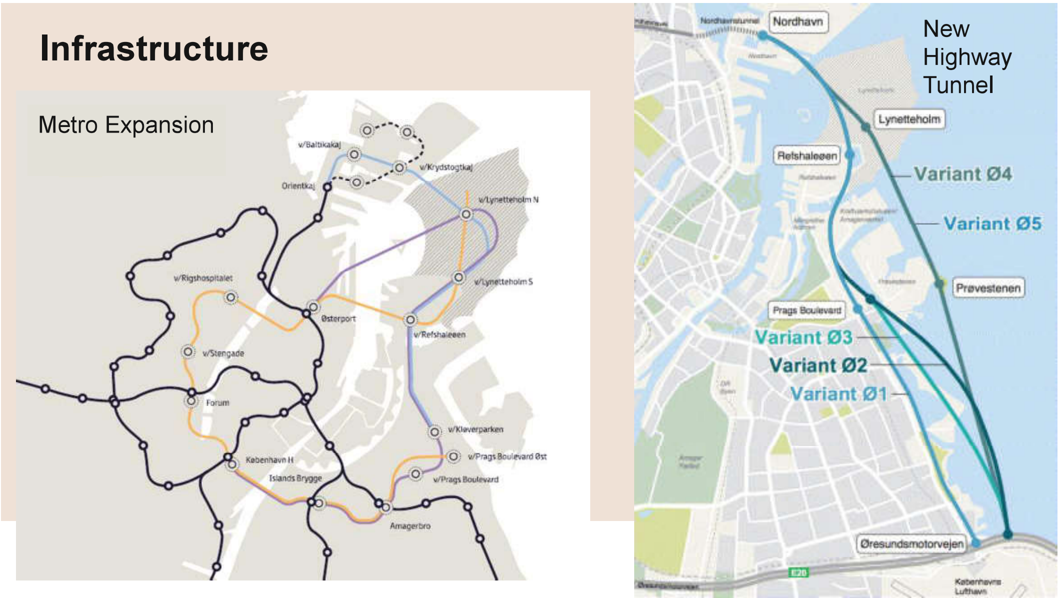


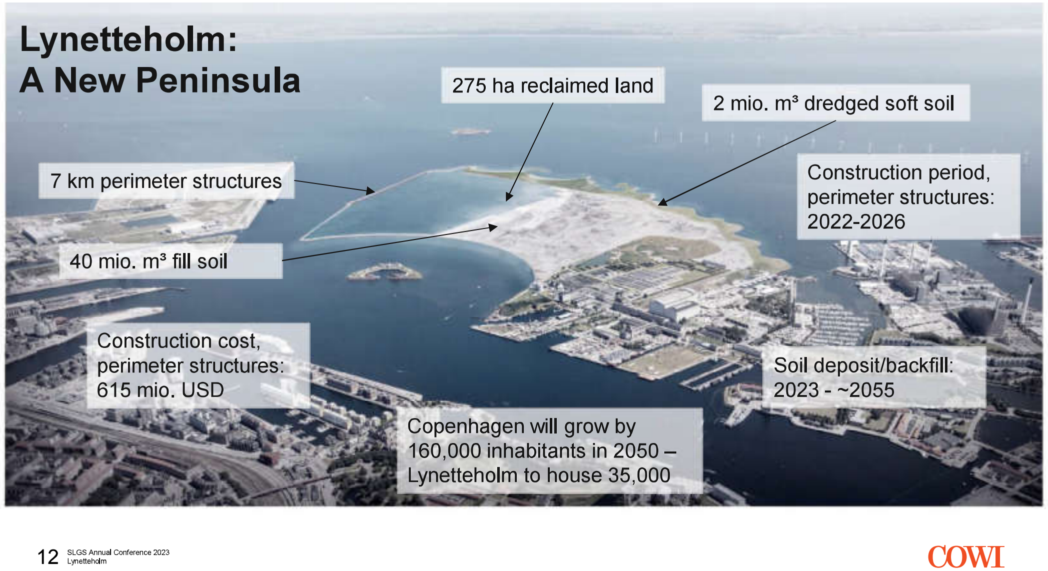
Climate Adaptation – Flood Protection




NORDHAVN
REFSHALEØEN
Needed
Constructed
Being designed
AMAGER STRANDPARK
KALVEBODERNE
VESTAMAGERDIGET (er anlagt)
ULLERUPDIGET (under projektering)
NORDHAVN
LYNETTEHOLM
REFSHALEØEN
Climate Adaptation – Flood Protection
• Lynetteholm is a key component of the flood protection system for the northern part of Copenhagen
COWI


Disposal of Soil
• 2,600,000 tons of excess soil generated annually
• ~350 full trucks daily
• Existing area in Nordhavn full
• Municipality obliged to provide reception facilities
• Additional 40 mio m³ capacity for soil disposal over ~30 years provided by Lynetteholm
• 'Bund now – fill later' design philosophy
10


Infrastructure
Metro Expansion
New Highway Tunnel
Nordhavn
Lynetteholm
Refshaleøen
Variant Ø4
Variant Ø5
Prøvestenen
Prags Boulevard
Variant Ø3
Variant Ø2
Variant Ø1
Øresundsmotorvejen


Lynetteholm:
A New Peninsula
275 ha reclaimed land
2 mio. m³ dredged soft soil
7 km perimeter structures
Construction period, perimeter structures: 2022-2026
40 mio. m³ fill soil
Construction cost, perimeter structures: 615 mio. USD
Soil deposit/backfill: 2023 - ~2055
Copenhagen will grow by 160,000 inhabitants in 2050 – Lynetteholm to house 35,000
12
COWI

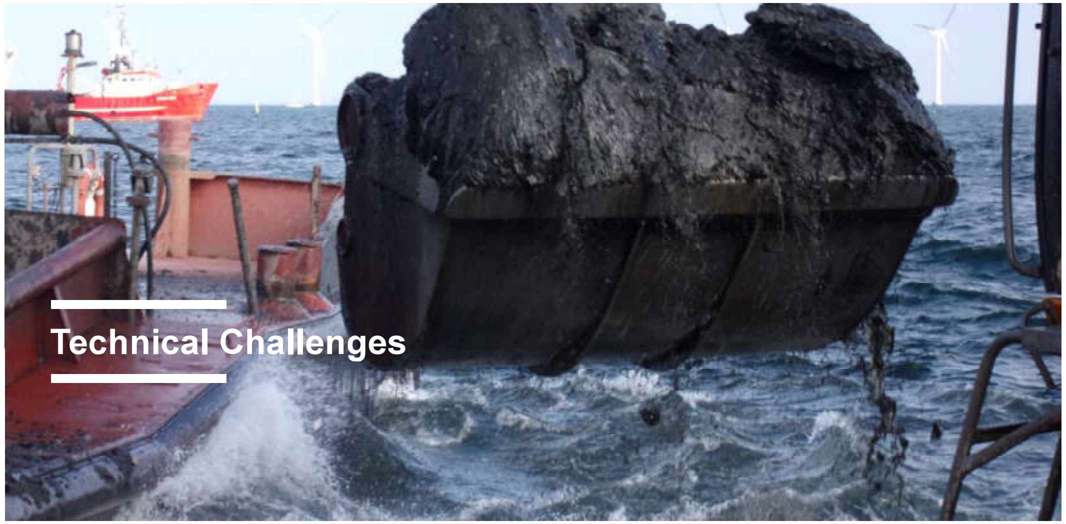
Construction Phases

Phase 1:

- **E1:** South-eastern perimeter to be constructed in contract E1 2021-2023

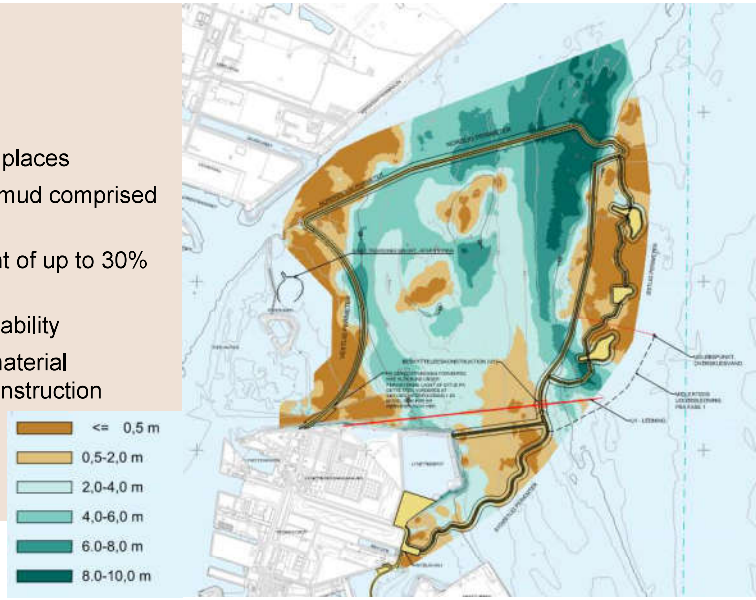
Phase 2:

- **E2:** Northern perimeter to be constructed in contract E2 2024-2026
- **E3:** Eastern perimeter to be constructed in contract E3 2023-2026
- **E4:** Western perimeter to be constructed in contract E4 2023-2025



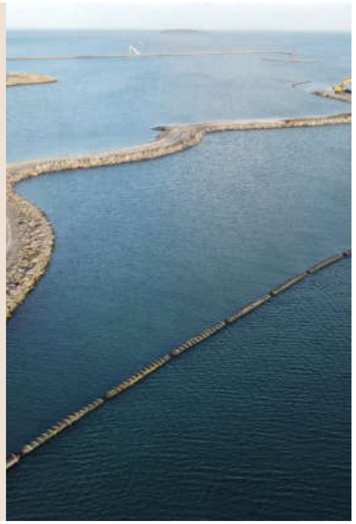
Soft Soil

- Up to 10 m soft soil in places
- Mainly extremely soft mud comprised of organic material
- Potential for settlement of up to 30% of layer thickness
- Potential for slope instability
- 2 mio m³ of soft bed material excavated for bund construction



Soft Soil - Solutions

- Due to risk of settlement and slope stability issues, soil improvement required for perimeter structures
- Alternatives for soil improvement considered:
  - Dredge all soft soil to competent layer
  - “Reinforce” soft soil with compacted crushed rock
  - Partially dredge and construct with overburden to account for settlements
- Preferred outcome was to dredge all soft material to reduce the risk and ease construction process



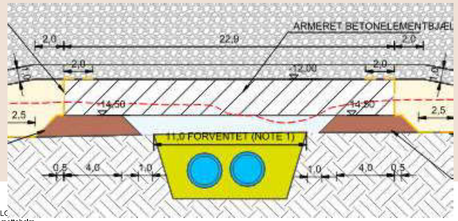
Soft Soil

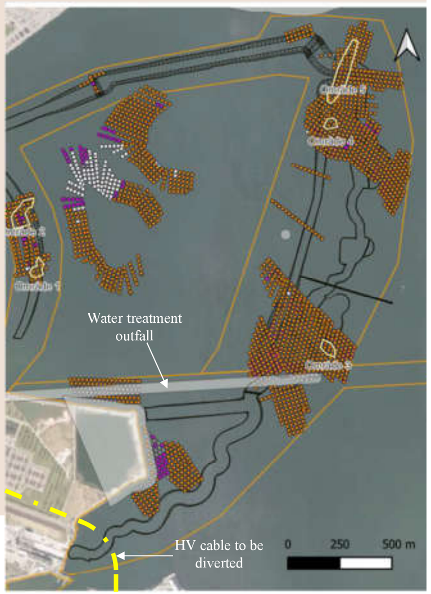
Reasonable firm at impact

Almost liquified when disturbed

Utilities and Existing Infrastructure

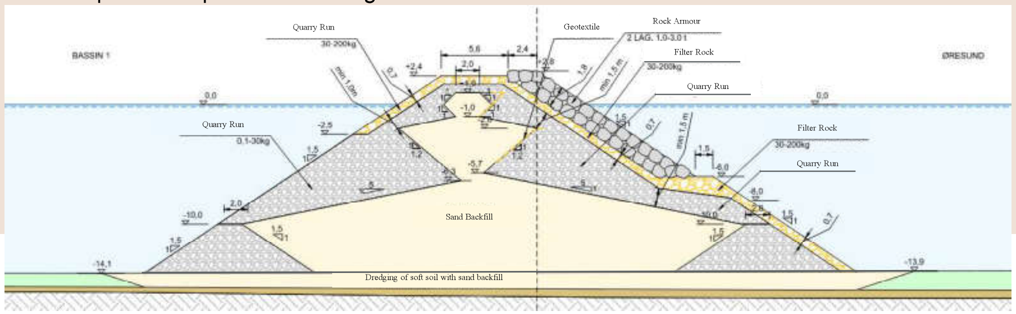
- Archeological pre-investigations – 2144 locations
- Main outfall from water treatment plant crossing the perimeter
  - Robust protection structure required for outfall – in the form of an underwater 'highway bridge'
- Diversion of existing high voltage power cables





Typical Bund/Breakwater Cross Section

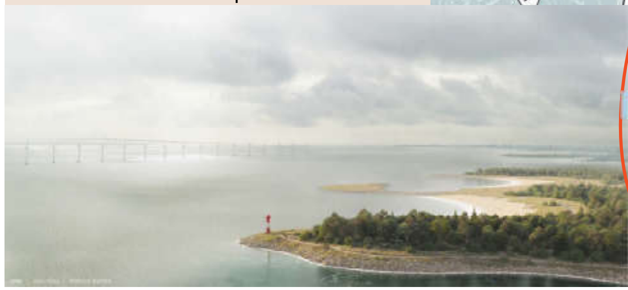
- Minimum 2 m sand filter below +1.0 to prevent suspended material (contaminated) from filtering through the structure
- Installation of sand under water, at 1:5 slope
- Installation of rock layers under water, at 1:1.5 slope
- To reduce the dredging volume/footprint, a dredge slope of 1:3 was adopted
- Complex work processes → higher cost

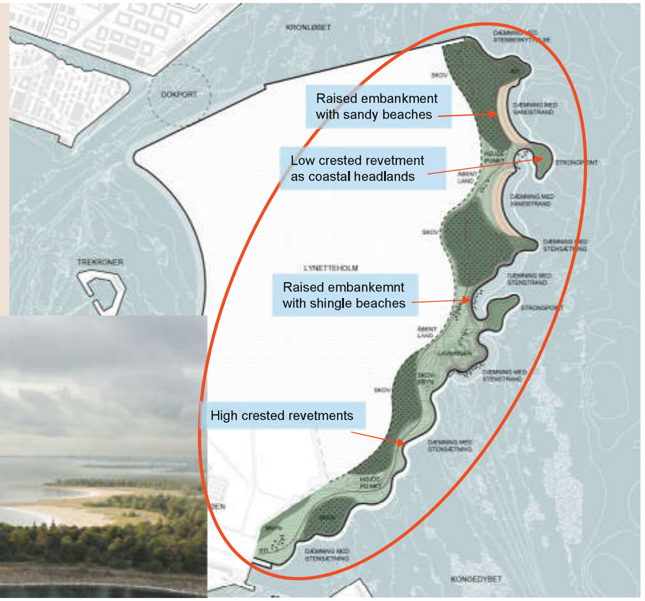


Breakwater cross section

The Landscape

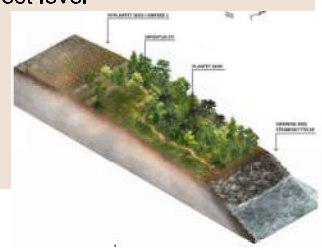
- 60 ha allocated to a coastal landscape
- > 60 mio. USD to develop in the future
- Aim to mirror the natural coastal landscape





Types of Edge Structures

- The Danish coastline consists of mainly beaches - sand or gravel
- Coastal protection is traditionally done using rock protection
- Different types of coastal structures were adopted for Lynetteholm to for the flood protection system
- Structures adaptive to align with localised conditions / requirements, to increase in crest level

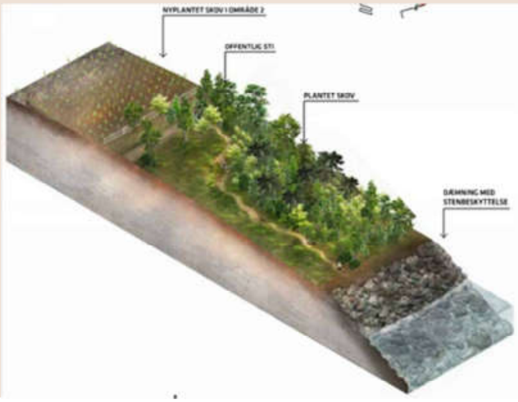


Structures – Vertical Wall

- Required level +4.3m MSL in 2070
- Advantages:
  - Multi purpose as quay and protection
  - Minimised footprint at harbour entrance
- Disadvantages:
  - Not natural
  - High maintenance cost
  - Hard to increase crest level
  - Limited lifetime

Structures – Rock Revetment

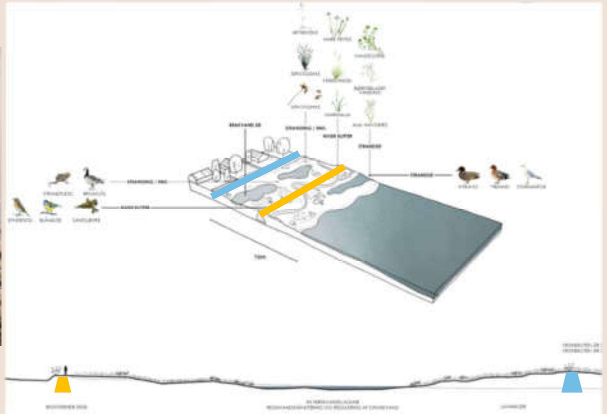
- Required level +3.5m in 2070
- Advantages:
  - Potential to increase crest level in the future
  - Allows for the dissipation of wave energy
  - More natural form of defence compared to a vertical wall, and offers potential for habitat creation
  - Low maintenance costs
- Disadvantages:
  - Limited access to the water
  - No access to foreshore for recreational uses

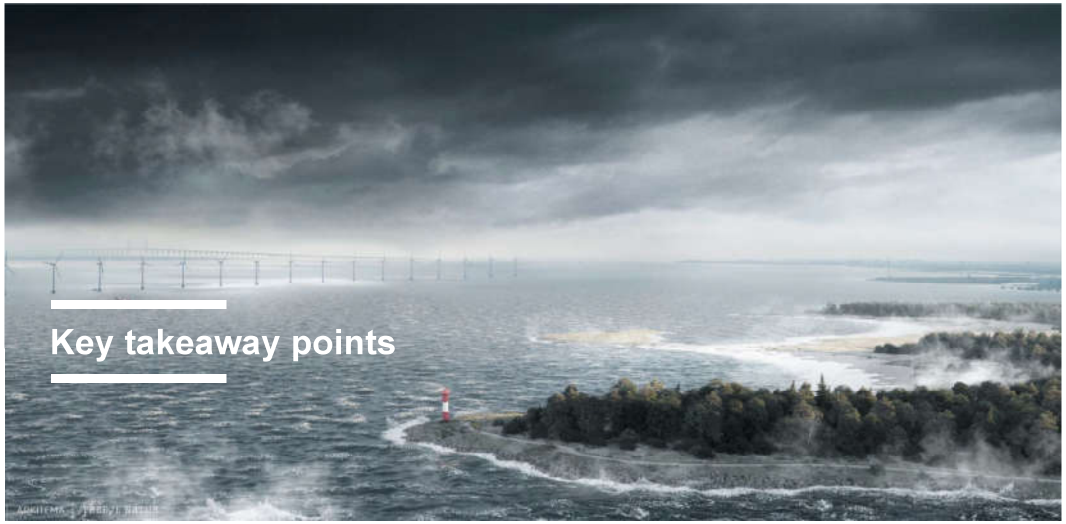


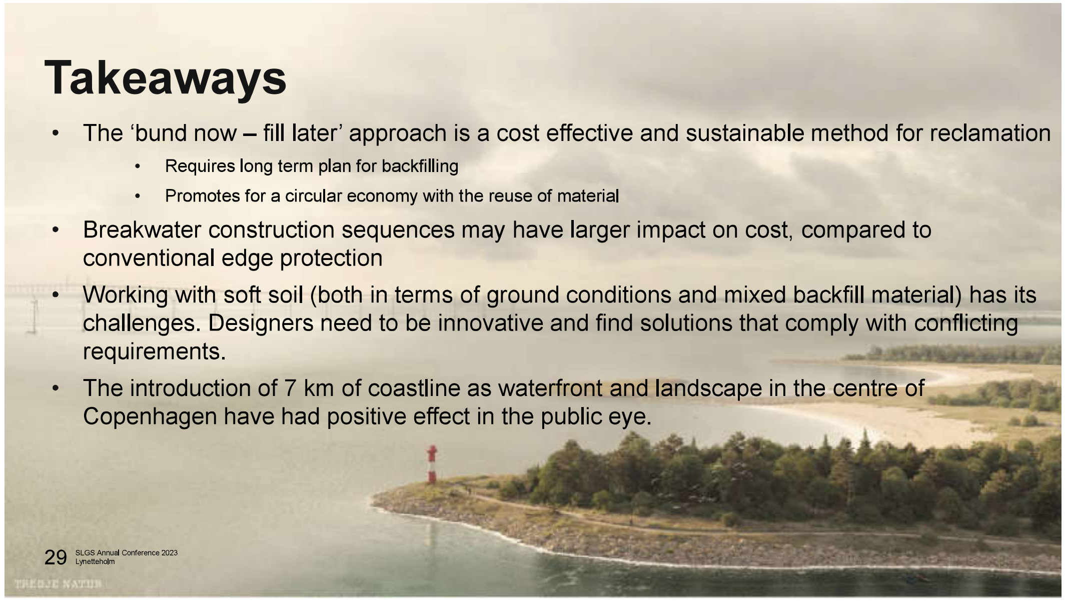
Structures – sand and gravel beaches

- Required level +2.6m in 2070
- Advantages:
  - Multi purpose as both coastal protection and beach
  - Creates a natural coastline, with flood defences less visual
  - Easy access to the to shoreline for recreation
  - Promotes habitat creation and positive impact on local wildlife
- Disadvantages:
  - Relatively high maintenance cost
  - Requires a wide footprint on the foreshore.

Structures – Beaches & Wetlands






Takeaways
• The 'bund now – fill later' approach is a cost effective and sustainable method for reclamation
• Requires long term plan for backfilling
• Promotes for a circular economy with the reuse of material
• Breakwater construction sequences may have larger impact on cost, compared to conventional edge protection
• Working with soft soil (both in terms of ground conditions and mixed backfill material) has its challenges. Designers need to be innovative and find solutions that comply with conflicting requirements.
• The introduction of 7 km of coastline as waterfront and landscape in the centre of Copenhagen have had positive effect in the public eye.
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