

**IMPORTANCE OF DETAILED CONSIDERATION OF
CONSTRUCTION SEQUENCE IN INSTALLATION OF
RETAINING WALLS WITH EXCAVATION:**

A CASE STUDY

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(2016/2017 BATCH)

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RETAINING WALL WITH EXCAVATION:
A CASE STUDY**

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Thesis/Dissertation submitted in partial fulfillment of the requirements for
the degree of Master of Science / Master of Engineering in Foundation
Engineering and Earth Retaining Systems

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August 2022

DECLARATION

I, the undersigned P.G. Deepika Chathurangani hereby declare that I am the sole author of this thesis. To the best of my knowledge this thesis contains no material previously published by any other person except where due acknowledgement has been made. This thesis contains no material which has been accepted as part of the requirements of any other academic degree or non-degree program, in English or in any other language.

This is a true copy of the thesis, including final revisions.

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ABSTRACT

Earth retaining structures are a key component in most engineering constructions. The present design practice as per the standard procedure is limited to analysis the structure under anticipated permanent design loads. However, in reality, the structure undergoes numerous loading patterns depending on the construction methodology as well as construction sequence that might be vary in type, magnitude, direction and distribution to that of design loads. The recent construction experience of some projects in Colombo indicates that these unaccounted loading patterns in construction stages could lead to failure modes that were not expected during design phase. In this study, a detailed forensic analysis is carried out for one case study by means of numerical modeling of different loadings undergone during various stages of the construction stage and root cause of failure for each case is identified. A gap analysis is carried out to ascertain that why the root causes had not been taken in to consideration during the design phase and recommendations are proposed to the design procedure currently practiced to avoid future such consequences.

Key Words – Construction Sequence

ACKNOWLEDGEMENTS

Prof. S.A.S. Kulathilaka, my study supervisor, deserves special thanks for his patient supervision, passionate support, and helpful criticisms of my research project. The various comments, suggestions, and help in keeping my efforts on track, as well as my extensive knowledge in the field of geotechnical engineering, all contributed significantly to the success of my study.

The course coordinator, Senior Lecturer, Dr. L.I.N. De Silva, deserves special thanks for his support with the Finite Element Analysis using Plaxis 2D software. His invaluable assistance enabled me to focus my studies in the right direction.

Mr.H.L. Prasanga, Project Engineer, thank you very much for your kind assistance in providing essential data which helped my study to get succeed. The encouragement you provided was extremely helpful in completing this study on time.

I would also like to extend my thanks to all the academic staff of Geotechnical Engineering, Prof. U.G.A. Puswewala, Dr. U.P. Nawagamuwa and Dr. (Mrs.) A.S. Ranathunga for helping in various ways to clarify the issues related to my academic works.

P.G. Deepika Chathurangani

01st September 2022

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CHAPTER 1 - INTRODUCTION

1.1 Background

The Metro Colombo Urban Development Project (MCUDP) aims to supplement the Government of Sri Lanka's urban redevelopment effort by lowering the physical and social consequences of floods by improving the Metro Colombo basin's outfall and storage capacity.

In order to accomplish the project objectives, numerous subprojects were initiated which involves detailed geotechnical designs. The set of flood mitigation interventions formulated to be implemented under MCUDP includes measures such as Storm Water Pumping Stations, Storm water tunnels, trans-basin diversions, construction of flood gates, culverts, canal bank protections etc.

The design of earth retaining structures can be identified as a key component of the overall design scope of the project as it plays a crucial role in almost all the sub projects specially in Canal bank protections. Some of these are permanent structures, supporting earth and additional surcharge along the canal banks while some of them are temporary ones as temporary shoring and coffer dams during the construction of deep cut excavations of projects such as pumping stations. Types of these earth retaining structures varies from pure gravity gabion retaining walls, Reinforced earth with gabion facing, cantilever retaining walls by reinforced concrete and sheet piles. The retaining heights of these structures range from 3m at most of the canals in Colombo drainage canals to as high as 10m in Madiwala east diversion canal. There is a significant variation of soil types as it varies from dense sand in Wellawatta canal to Clay and peat soil in areas such as Ambathale. Therefore, it is evident that the range of design parameters over the scope of earth retaining design under MCUDP is quite significant.

In this context, unforeseen circumstances were encountered during the construction of some of these structures resulting in additional costs for alteration of construction methodologies, additional rectification works, design changes and damage repairing works. These resulted in significant consequences in terms of time and cost in

addition to the contractual disputes hindering the construction works. The forensic analysis of these selected projects indicates that there are additional geo-technical considerations that should have been made related to constructional sequence in the design phase. Although the final form of the retaining structure protects the canal bank is provided in the drawings the construction sequence to be adopted in completion of the structure had not been considered in detail. Therefore, it is evident that detailed analysis of the lessons learned from these selected projects as case studies would provide some useful guidance for future designers regarding paying a greater attention to constructional sequence when designing of earth retaining structures in urban areas. The construction sequence may consist of number of stages and a safe design should be provided to each stage. This should supported by detailed method statement.

1.2 Problem Statement

The importance of giving due consideration to construction sequence is highlighted with the case history of buildup damages during the construction of canal bank protection of St. Sebastian North Canal in Sedawatta. The excavation necessary for the support of excavation to the construction of gabion walls was supported by sheetpiles.

The sub project related to this case study is bank protection of the St. Sebastian North canal to ensure the canal conveyance capacity to St Sebastian North pumping station. The bank protection has been proposed to be carried out as a gabion wall and the initial design had been conducted. However, no detailed attention has been paid to the construction stages in placement of the gabion wall. As a result, considerable damages were caused to the nearby buildings resulting many negative social, financial and contractual consequences. The damaged buildings were constructed on shallow foundations and considerable large cracks on the buildings were started to appear during the implementation making those buildings unusable and unsafe to live in (Figure 1.1).



Figure 1.1: The nature of wall and floor crack of a building.

The site observations hinted that these damages occurred due to the ground settlements during the construction stage and may have been occurred due to unfavorable loading condition during the construction stage. Therefore, this study is carried out as a forensic analysis of the above case study.

1.3 General Site Conditions

The site is located at left bank of St. Sebastian North canal at just upstream of the St. Sebastian North storm water pumping station, Colombo 14, in the Western Province of Sri Lanka (Figure 1.2). This site is under the work scope of “Rehabilitation of St. Sebastian North Canal” which is a sub project under “Metro Colombo Urban Development Project (MCUDP)”. This sub project is implemented with the objective of assuring the conveyance capacity and sustainable canal maintenance of St Sebastian North canal by canal bank protection in order to assure the outfall discharge capacity of St Sebastian North pumping station and St Sebastian North gate to flood mitigation of Metro Colombo drainage basin.

The land use of the site location can be identified as densely built urban residential area with almost the entire land is covered with buildings. The distance from the canal bank to the edge of the building line can be approximated as 4m. The

topography of the site is varying +2 to +3 mMSL with a mild slope towards the canal bank at +2 mMSL. The bed level of the canal is -1 mMSL. The normal level of the canal water level varies from -0.2 mMSL to +0.6 mMSL depending on the tide condition and the high flood level at extreme events can reach as high as +2 mMSL.

The buildings within the site area are 2 to 3 story buildings with reinforced concrete slabs with walls of cement blocks built on RR masonry shallow foundations. The age of the existing buildings can be estimated as 15 years.

The socio-economic condition of the area can be identified as a low income urban residential area and hence the population density in the site area is extremely high and thus the risk during construction is increased since damage to any property during construction can cause huge social unrest creating many unfavorable social issues in addition to the actual structural damage.



Figure 1.2: Location Map.

1.4 Scope and Objectives of the Thesis

1. In depth forensic analysis of the selected case study to identify the geo technical root causes that led to the damaged during the placement of the gabion protection wall at the canal bank.

2. A detailed study of the construction sequence adopted and find modifications that should have to be done to prevent / minimize the damage.
- Provide additional recommendations that are to be included in the geotechnical design procedure with respect to construction sequence when designing earth retaining structures in urban areas.

1.5 Thesis Outline

The Thesis is structured with following chapters.

- Chapter 2 – Literature Review

The inadequacy of the detailed analysis on the construction sequence and the subsequent process of remediation were also discussed. Case studies related to excavation induced ground settlements at urban environments in Colombo are discussed in relation to the issues of construction sequence.

- Chapter 3 – Methodology

The study methodology followed is described in detail under this chapter. The methodology was developed based on the analysis methodology of the past studies discussed under the literature review. The methodology has been structured as per the following flow.

- (a) Background of the actual failure discussed in this study.
- (b) Formulation of research hypotheses.
- (c) Methodology of the analysis and presentation of the results.
- (d) Discussion of the results of the analysis and guidelines for development of conclusions and outcomes of the study.

- Chapter 4 – Simulation of the Construction Sequence through finite element analysis

This Chapter can be considered as the essence of the study where the simulation of the finite element analysis is discussed in detail. Critical stages in the construction sequence all the model parameters with respect to soil details, drainage status, model analysis and boundary conditions are outlined. Detailed description of the

construction sequences with construction stages and the ground settlements in each construction stage and the impact on the surrounding structures is discussed.

All the results are presented in terms of the vertical settlements and summarized in graphs for all stages of each sequence.

- Chapter 5 – Conclusion and Recommendations.

The case study's final conclusions and future recommendations when designing earth retaining structures in urban areas are presented in this chapter.

CHAPTER 2 - LITERATURE REVIEW

This chapter presents some case studies in the past regarding the damages caused to nearby buildings due to excavation and dewatering and their connection with the construction sequence followed at the site.

The geotechnical root causes of these failures varies from inadequate temporary shoring, consolidation settlements due to dewatering and soil heaving during the excavation. However, it was revealed that almost all these failures had caused due to inadequate analysis on the stages of the construction sequence and the damages of almost all the cases would have been avoided had there been enough analysis on the construction sequence. The damages were able to be mitigated by remediation measures where there had been limited analysis on the construction sequence and heavy costs and irreversible structural damages were observed.

2.1 Overview

Because soil is exceedingly variable in nature and subject to a range of loads, pore pressures, time-dependent processes, and other variables, it is the most challenging engineering material encountered in civil engineering constructions (Tonks et al, 2017). Even so, breakthroughs in research, laboratory and in-situ testing, and analytical techniques have increased understanding of the soil behavior.

Other challenging soils such as; expansive and collapsible soils, make geotechnical design and construction more complex. Those problematic soils can support load in an undisturbed, in-situ condition, but they react differently when exposed to water, cyclic stresses, temperature variations, and other factors. Most cohesive soils, are extremely sensitive to changes in moisture content, resulting in progressive settlements or swelling. Coarse granular soils, on the other hand, usually settle quickly when subjected to a load.

Ground settlements, tilt, and even cracking of neighboring structures can occur as a result of certain construction operations including excavation and dewatering. Dewatering generated ground settlements by changing the soil conditions from submerged to saturated or dry owing to stress re-distribution in the soils (Zumrawi

and Hassan 2016). If safety measures are not taken throughout the design process, surrounding structures may eventually fail. (Samaaneh and Al-Gadhib 2013)

If extremely loose layers are present at shallow depths and subjected to a quick rise in overburden pressure, the implications get adversed due to change in soil stresses. Therefore, appropriate analytical technique should be used to estimate projected ground settlements well before the dewatering process (Tan et al., 2014). This allows the theoretically produced conclusions to be cross-checked with field monitoring data at each step of construction (Charles 2008).

2.2 Importance of considering construction sequence in urban environments

When new construction are done there is a legal obligation to ensure that it will not have any adverse effect on the existing structures. In an urban environment with very closely spaced developments this would be a very challenging task.

As such consideration of construction sequence is of utmost importance. Steps in the construction sequence should be clearly identified and temporary / permanent items should be designed to take the loads and control the deformations.

Ground excavation projects for infrastructure development are being planned in response to the expansion in urban development. When designing a braced excavation, it's important to think about not just the lateral supporting structure's stability, but also the possible serviceability issues that excessive wall deflections and ground movements might be caused in surrounding structures (Wengang et al.2017).

Stress changes in the soil around the excavation may occur during ground excavation, resulting in unavoidable ground losses. These changes in soil layers and ground losses have an impact on the surrounding ground in the form of ground movements, which eventually impose direct stresses on neighboring buildings. The amount and distribution of ground movements in an excavation are heavily influenced by soil properties, excavation design, excavation depth, breadth, length, wall and support system types, and, most critically, construction procedures (Chungsik and Dongyeob,2008).

In metropolitan locations, excavations are frequently carried out extremely near to existing structures. As a result, the safety of nearby structures may become a major concern. To minimize damage to nearby structures and existing buildings, a precise estimate of ground surface settling surrounding an excavation is critical. Diaphragm walls, sheet pile walls, and composite supporting systems are examples of systems that can be used to support excavations (Ou et al.1993). Furthermore, case study records of both successful and unsuccessful projects may be utilized as a supplement to the observational technique, which is becoming more common in geotechnical design (Ciria 1999).

As an outcome, various case study records have been chosen and addressed below in order to highlight the relevance of construction sequence for geotechnical designs.

2.3 Case Studies

2.3.1 Case 1 - Structural Damage to Adjacent building during removal of shoring for gabion construction in Ambathale , Sri Lanka.

The report of “Damage Assessment Report for No. 52, Wellampitiya Road, Ambathale” carried out by National Building Research Organization (Annexure I) was referred for the information of this case study and details were further reviewed during the site visit to actually damaged site.

2.3.1.1 Case details.

This case study is related to construction of gabion canal bank protection for Madiwala East diversion canal at Ambathale. Structural damage to the adjacent building occurred during the removal of temporary shoring system. The location of the site is at Ambathale near the confluence of MED canal with Kelani river (Figure 2.1).



Figure 2.1: Location of the site.

The building named as “shop” in Figure 2.2 is a newly built two story building on shallow foundations. However, it was confirmed that this building was constructed with proper tie beams in between footings such that it could withstand significant differential settlements.

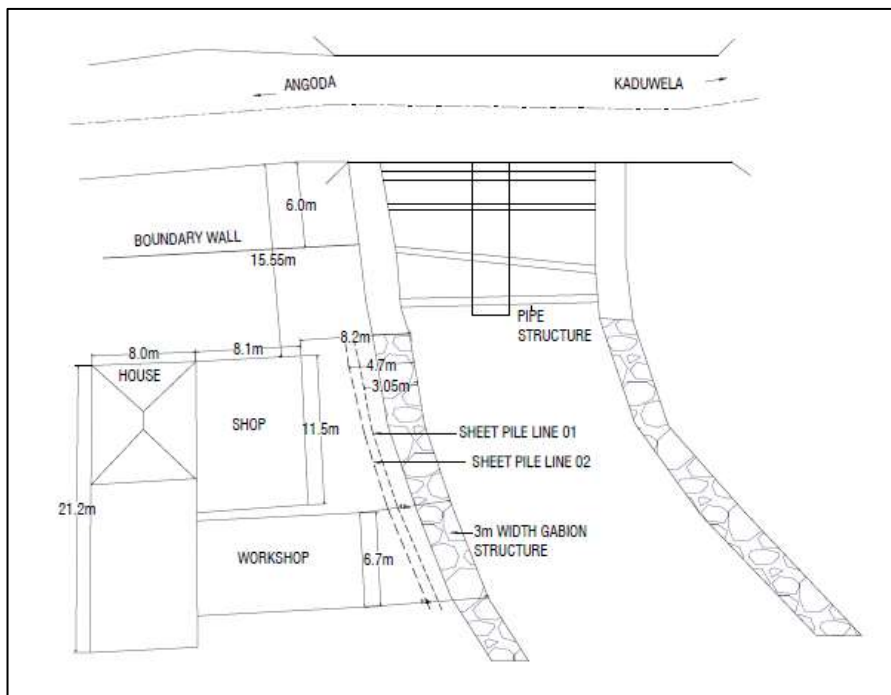


Figure 2.2: Plan view of project area and distances to the surrounding buildings.

The adjoining building named as “House” in figure 2.2 is a comparatively old single-storey residential building on shallow foundations and it was confirmed that the foundation of this house is on a rubble foundation such that no tolerance for differential settlements was provided.

Canal bank protection design of the project area.

The design section relevant to the project area is shown in Figure 2.3.

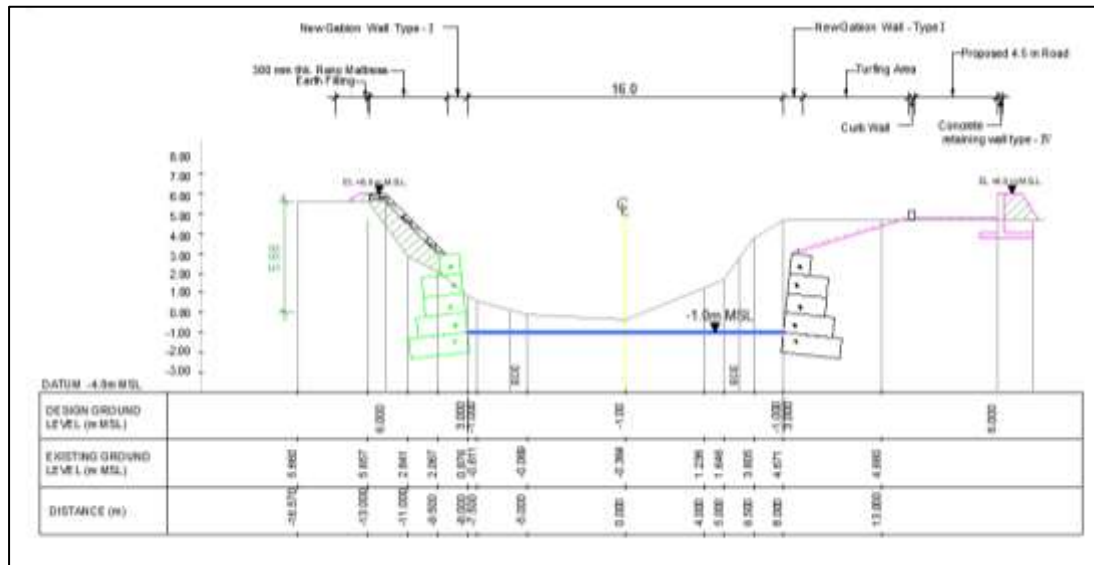


Figure 2.3: Cross sectional view of the canal bank protection design used at the site.

According to the original design (Figure 2.3), the level difference between the gabion top level and the present ground level at the left bank is nearly 3m. Because the level difference is significantly greater, it was planned to use two lines of 9m length sheet piles, with the top level of sheet pile line 01 at +3.00m MSL and the top level of sheet pile line 02 at +5.65m MSL, to retain the ground without causing differential settlements in neighboring structures (Figure 2.4). Sheet pile line 01 was a 5m x 5m square shoring area which has been braced using 100*100*6mm H iron (Figure 2.5). Sheet pile line 02 was installed with 9m length sheet piles and shoring plates. In the design section, this level difference was proposed to be finished with a stable slope protected by a gabion reno-mattress.

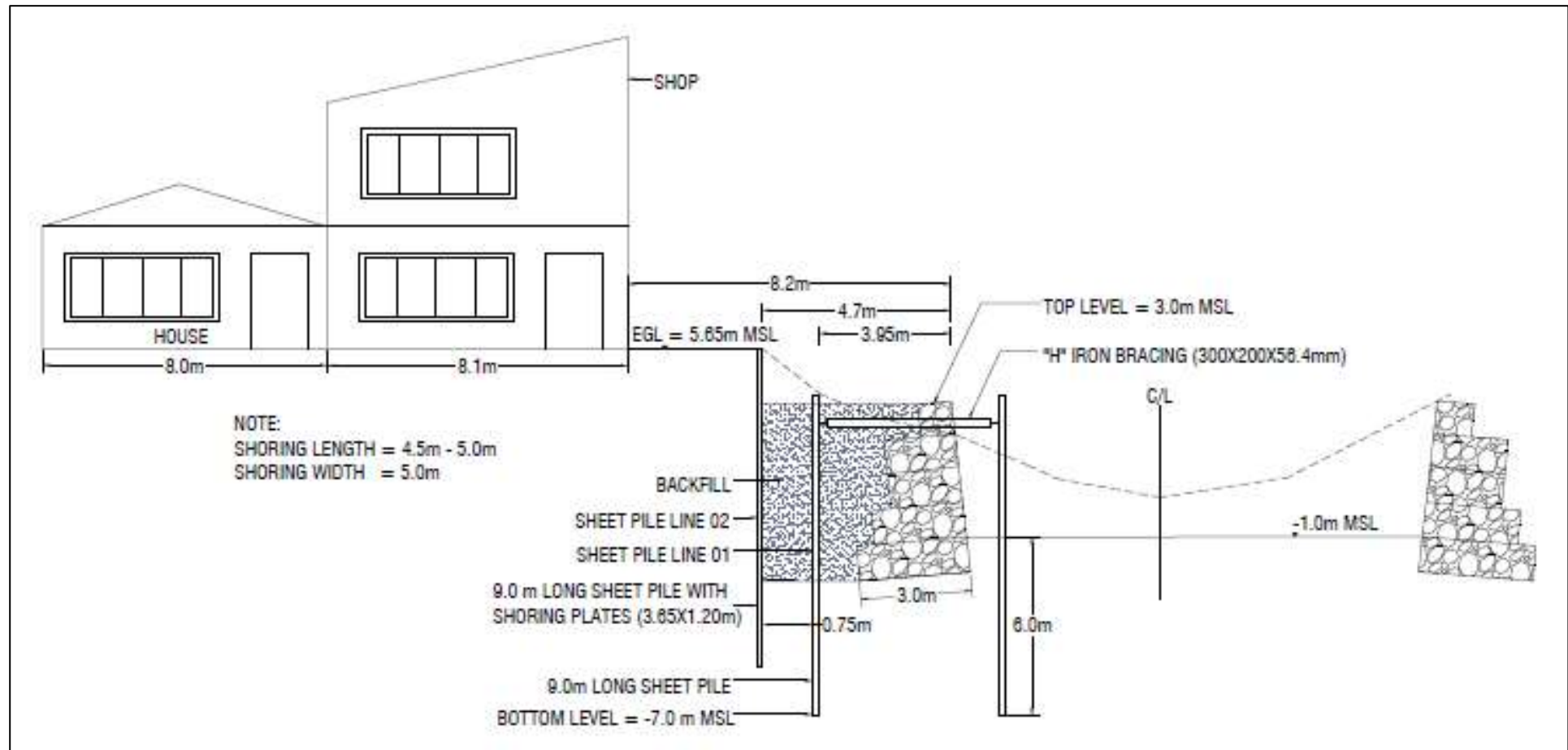


Figure 2.4: Temporary shoring system used at the site and surrounding building details

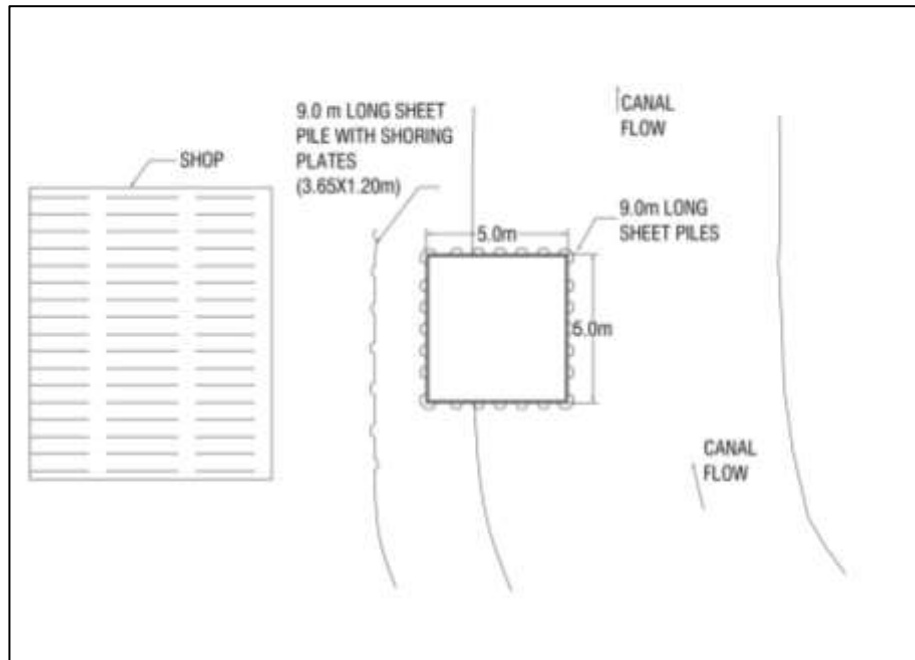


Figure 2.5: Plan of the temporary shoring arrangement followed at site.

Soil condition within the project area.

The soil condition at the project area can be tabulated as Table 2.1 according to the bore hole investigation report (Annexure II).

Table 2.1: Soil condition of the project area.

Depth (m MSL)	Soil Type	Average SPT N value
(+5.7) - (+1.7)	Clayey Silt	6
(+1.7) - (-1.3)	Clay	5
(-1.3) - (-2.3)	Peat with timber pieces	11
(-2.3) - (-10.3)	Clayey Sand	4

As per the borehole investigation report, the first 4m of top soil (from +5.7 mMSL to +1.7 mMSL) is classified as loose to medium dense clayey silt with SPT around 6 and underlaid by a 2m thickness clay layer , 1m thickness peat with timber pieces layer and 8m thick clayey sand layer with SPT N values around 5, 11 and 3 respectively.

2.3.1.2 Nature of Damage

A design for the temporary shoring system had been conducted prior to the implementation and required sheet pile lengths and depth of embedment had been designed and however, any analysis with respect to construction sequence had not been conducted.

The shoring arrangement had been set up as per mentioned above and the gabion walls had been constructed. The backfill of the land side of the gabion had only completed upto +2m MSL. As per the construction methodology, the next activity is removing the interlocked sheet pile shoring (Sheet Pile Line 01) and prepare the canal embankment to the design slope to lay the gabion reno-matress. However, once the sheet piles of the interlocked segment were ejected severe ground cracks were appeared since outer sheet pile line (Sheet Pile Line 02) was deformed towards the canal side (Figure 2.6).



Figure 2.6: Initiation of Ground Cracks.

In a short while after the initiation of ground cracks, severe cracks started to appear in the residential building (i.e. the building named as “House”). The nature of these cracks was so severe that the safety of the people inside the house were at risk. The construction works had been stopped immediately and monitored whether the cracks were propagating.

The locations of the cracks appeared in the “House” are shown in Figure 2.7.

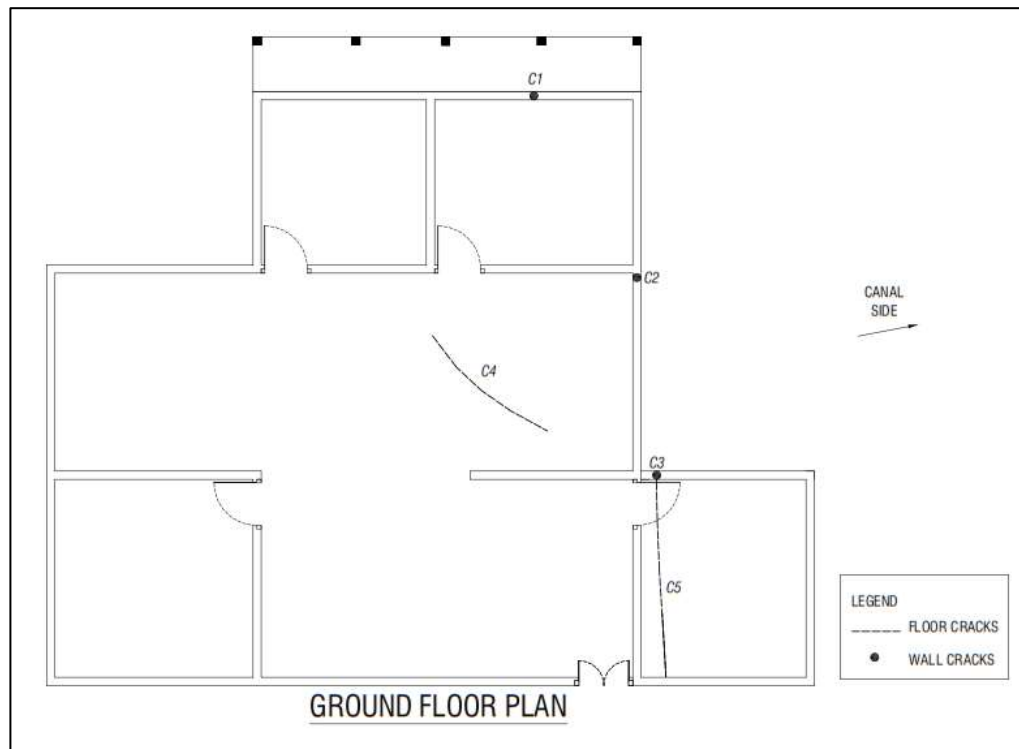


Figure 2.7: Crack locations at "House".



(a) C1 crack outside view

(b) C1 crack inside view

Figure 2.8: C1 crack.



(a) C2 wall crack

(b) C4 floor crack

Figure 2.9: C2 and C4 cracks.



Figure 2.10: C3 wall crack.



Figure 2.11: C5 floor crack.

Many minor and major cracks were visible in all over the house and contractor immediately started to identify all the cracks inside the building and started to monitor them. Once the crack propagation was observed to be stable, forensic analysis was conducted in collaboration with NBRO and it was decided that these settlements was occurred due to removal of interlocked sheet piles, backfilling is done only for +2 mMSL and the only resistive force provided by intermediate sheet piles with plates were not enough to hold the lateral earth pressure.

As per the initial design, the top of gabion finished level was at +3 mMSL and existing ground was at +6 mMSL and hence there was a 3m vertical difference which was originally proposed to be stabilized by a gabion mattress. Due to this unexpected collapse, the design and methodology were modified and it was decided to modify the gabion section such that the top level of gabion was at +4 mMSL (with reduced lateral pressure as the level difference was now 2m) and construct the sloped gabion mattress starting from +4 mMSL and that is the completed canal bank protection as seen now (Figure 2.12).



Figure 2.12: Completed canal bank protection as seen now.

Due to this sudden collapse, the construction works had to be stopped for a considerable duration creating many contractual issues and delays of completion date. The compensations had to be paid for the owner of the building for the property damage. A considerable idling of resources was also caused due to this incident.

2.3.2.3 Relevance of the case on this study.

It was mentioned above that a design for the temporary sheet piles had been already conducted prior to the analysis and however any analysis on construction sequence had not been conducted. Had any analysis on construction sequence been conducted prior to the implementation, the excessive ground settlements during sheet pile removal could have been captured and if so, the construction methodology would have been modified saving compensation money, additional cost for design modifications, cost for resource idling contractual issues and delays due to this incident.

Therefore, this case can be treated as an example to highlight the importance and need of this study and the objective of this study is to use the lessons learned from this kind of cases and develop recommendations for future designs in order to avoid this kind of damages in future. The value on the analysis of construction sequence is further emphasized by this case study.

2.3.2 Case Study 2 – Structural Damage to Adjacent building during construction of North Lock Pumping Station in Sedawatta , Sri Lanka.

The report of “Building Condition Technical Evaluation Report of Dwelling House at No. 14/19, Sedawatta Road, Wellampitiya” carried out by Central Engineering Consultancy Bureau Organization was referred for the information of this case study and details were further reviewed during the site visit to actually damaged site.

2.3.2.1 Case details.

This case study is related to construction of pumping station for North canal at Sedawatta. The structural damage occurred to the adjacent dwelling house during the excavation and dewatering process in the project area is broadly explained. The house was located on the right bank of St. Sebastian North Canal adjacent to the construction site of North Lock Gates and Pumping Station Project (Figure 2.13).

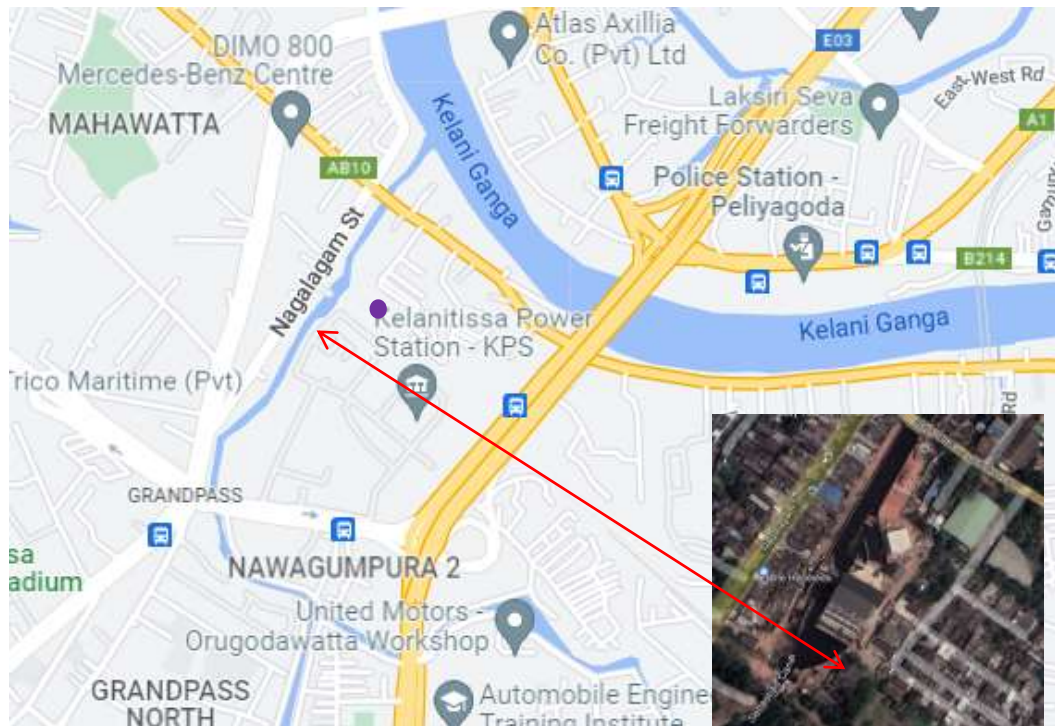


Figure 2.13: Location of damaged dwelling house.

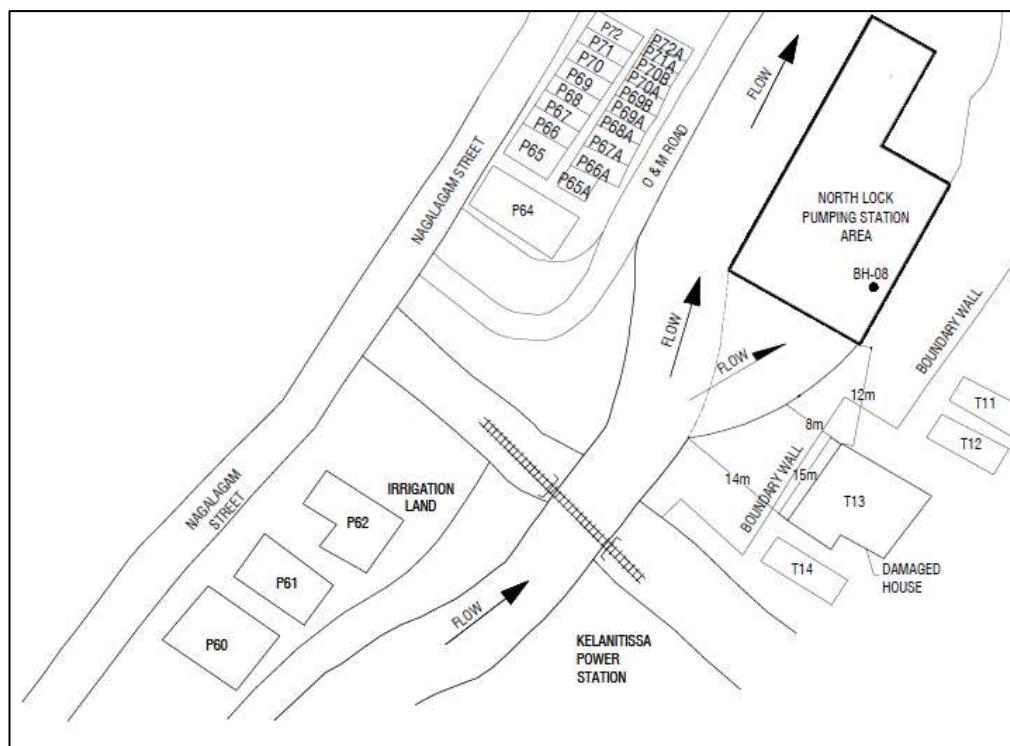


Figure 2.14: Plan view of project area and distances to the damaged house.

The dwelling house shown in Figure 2.14 as T13 is around 6 years old, and the foundation was built using random rubble masonry walls. Bricks with cement plaster, a thickness of 150mm were used to build the walls.

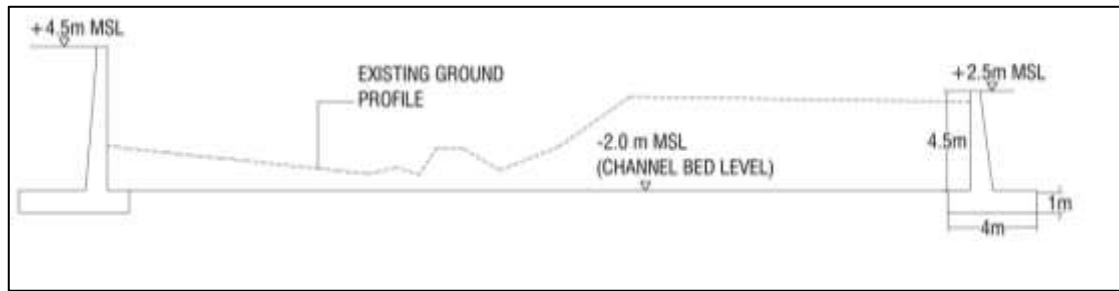
Design Section of the Inlet Channel.

Figure 2.15: Cross section showing the design of retaining walls in inlet channel area.

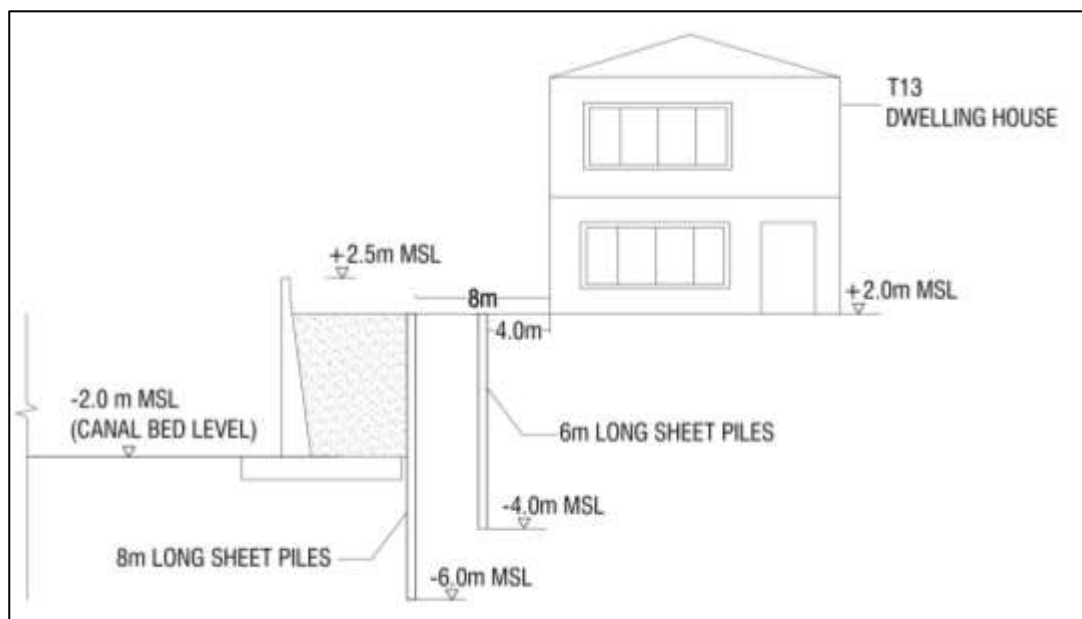


Figure 2.16: Section Showing Sheet Pile Arrangement Used at the site to construct the inlet channel area.

House, T13 is located in a close proximity, approximately 8m from the 8m long sheet pile line of inlet channel area and approximately 4m from the 6m long sheet pile line which is driven closer to the boundary line of the site. (Refer Figure 2.16 and 2.17). Further, the house is located approximately 12m southeast direction of the construction area of the pumping station.

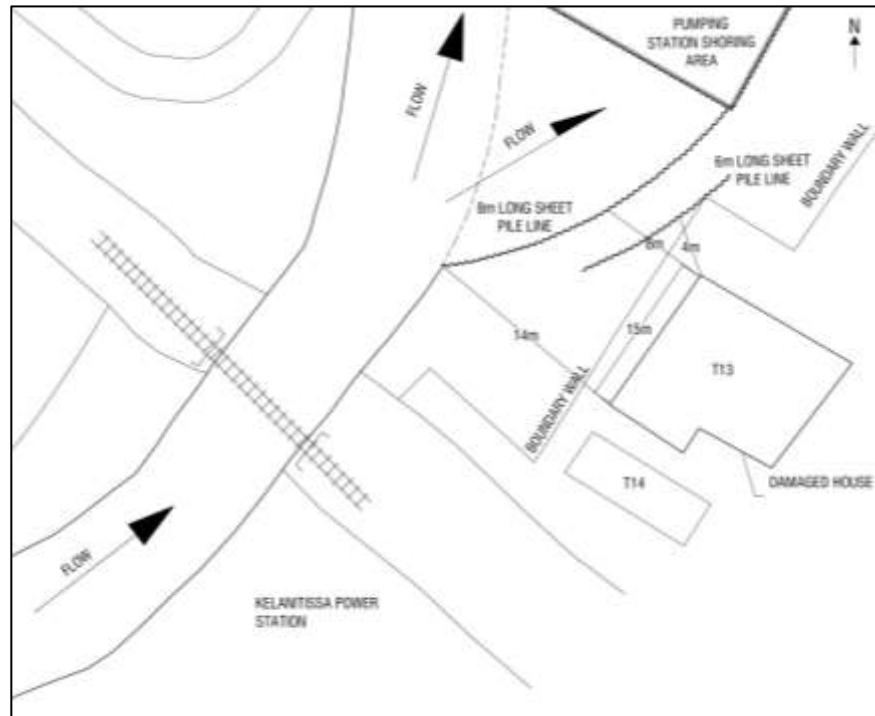


Figure 2.17: Plan Showing Sheet pile Arrangement Used at the site to construct the inlet channel area.

Ground condition underneath the foundation of House T13.

The types of soil present underneath the damaged house were obtained using the Geotechnical Investigation Report prepared for the St. Sebastian North Lock Gate and Pumping Station Project. Data obtained for the Borehole 08 (Refer figure 2.14 and Annexure III) has been selected to determine the soil condition underneath the T13, since it represents the closest ground condition of the damaged dwelling house.

Table 2.2: Soil condition of the project area.

Depth (m MSL)	Soil Type	Average SPT N value
(+1.0) - (+0.0)	Clayey Sand	0
(+0.0) - (-0.5)	Very Loose Sand	1
(-0.5) - (-2.5)	Very Loose Clayey Sand	3
(-2.5) - (-4.0)	Very Loose Sand	1
(-4.0) - (-5.0)	Loose Sand	6
(-5.0) - (-7.0)	Very Loose Sand	3
(-7.0) - (-8.5)	Organic Clay	7
(-8.5) - (-10.0)	Lateritic Clay	12

As per the above log sheet, very loose fine to coarse sand with a SPT value of less than 6, is available up to a depth of 8m from the ground level(+1.0 to -7.0m MSL). Beyond that, layers of organic and lateritic clay with a SPT value less than 12, are available up to a depth of 11m from the ground level (-7.0 to -10.0m MSL).

2.3.2.2 Nature of the damage.

There was another excavation pit (shown in figure 2.14) for the pumping station substructure carried out next to this proposed retaining wall construction site. It is to be mentioned that both these were two activities in the same site. The excavation at that pit had already been started together with dewatering. The sheet piling for the shoring for this retaining wall excavation had started while the constant dewatering was ongoing in the adjacent excavation pit for the pump house sub structure.

At this point of time, appearance of ground cracks was visible (Figure 2.18) and subsequently separation of the boundary walls (Figure 2.19) had been visible and at the same time, cracks were started to appear within multiple areas of the damaged house making the structure almost unserviceable.



Figure 2.18: Ground Cracks at front side of T13 House.



Figure 2.19: Separation of boundary wall of T13 House.

Thereafter, the construction works had temporarily been suspended and crack widths, crack propagations were monitored carefully. The preliminary inspection yielded that some of these cracks were actually widenings of the existed cracks which were identified during pre-construction survey and some were new severe cracks appeared due to unstable ground condition.

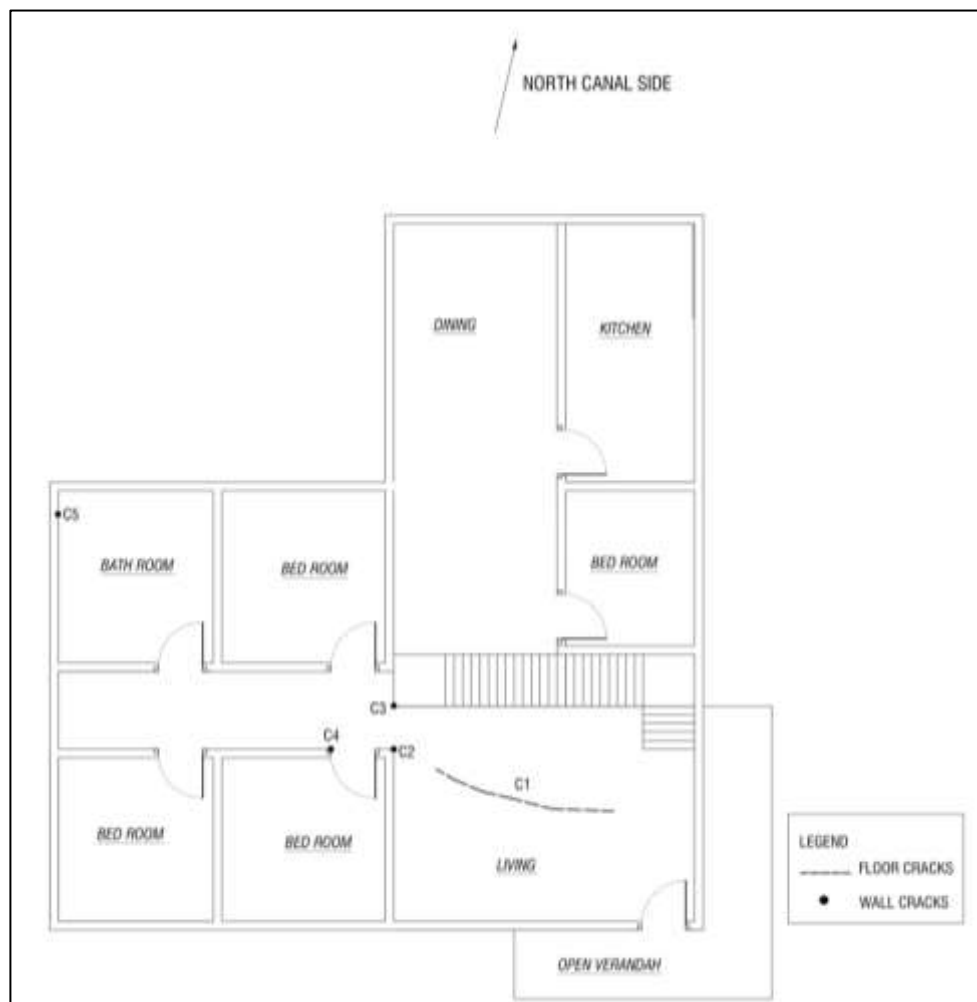


Figure 2.20: Ground Floor Crack Locations of T13 House.

The location and nature of these cracks are shown in figures 2.20 to 2.25. The monitoring observations during 19th may 2018 to 12th October 2018 has yielded that rear side of the building had undergone a maximum settlement of 101mm with a differential settlement of 17mm.



Figure 2.21: C1 crack on 19.12.2018.

This floor crack denoted as C1 was appeared along the living of the house and crack width was 4mm.



Figure 2.22: C2 crack on 11.09.2018 and 19.12.2018.

The width of the crack C2 was recorded as 7.36mm initially and it was further widened to 19.29mm during the subsequent 03 months.



Figure 2.23: C3 crack on 11.09.2018 and 19.12.2018.

The vertical crack C3 was initially measured as 8.62mm during the initial assessment and it was further widened to 11.46mm within the course of next 03 months.



Figure 2.24: C4 crack on 11.09.2018 and 19.12.2018.

The crack C4 was a widening of an existed crack. Crack width was 2mm prior to the construction and it had widen to 13mm during dewatering and further widened to 17.15mm after 03 months of the incident.



Figure 2.25: C5 crack on 11.09.2018 and 19.12.2018.

The width of crack C5 was 9.46mm immediately after the incident and further widened to 25.44mm over the course of next 03 months.

After the settlement had ceased, Central Engineering Consultancy Bureau (CECB) had been requested for a conditional report (Annexure IV) and following are the key remarks.

Based on the readings of the monitoring stations the structure had tilted towards the rear side of the building affecting the verticality of the structure. This tilt could cause additional stresses on columns and footings at rear side of the building. Since there were no cracks at reinforced concrete elements, it had been assumed that the whole two storied building had tilted as one unit.

The subsurface condition up to 8m depth consists of loose sand layers with low SPT values and it was likely that this failure due to ground settlement had caused by combined actions of increase of effective stress due to lowering of groundwater table under the footings in loose sand strata due to dewatering of the adjacent excavation pit and vibrations due to sheet pile driving and other site activities such as vibratory compaction has also caused the settlement.

It had been concluded that structural integrity of the walls had been lost due to wide cracks and roof connections are at a vulnerable state as they were dislodged due to tilting. Though the allowable settlement of a building depends on rigidity and type of construction, the general rule of a total settlement of 25mm and a differential settlement of 20mm between columns could be considered under service loads and yet a maximum settlement of 101mm had been reported beyond acceptable limits and hence it had been concluded that the structure was unsafe without restoration works.

It had also been concluded that retrofitting works such as underpinning with micro piling and jacking up were specialized engineering works and cost of such works would exceed the cost of demolish and rebuilding and therefore, the proposed option was to demolish and rebuild the structure.

Accordingly, a compensation of a cost exceeding 8 million LKR had to be spent to the owner and additionally many time delays and resource idling were also occurred creating contractual issues.

2.3.2.3 Relevance to this study.

It had been revealed that there was no analysis on construction sequence had been conducted prior to construction on this. Had such an analysis was conducted, the occurrence of excessive settlements during the intermediate stages would have been captured and accordingly remediation measures such as using longer sheet piles (to assure adequate penetration in to clay layer beyond loose sand strata), ground water recharging would have been proposed and the damage could have been avoided.

Therefore, this case study can also be treated as a classic case to illustrate the importance of the analysis on construction sequence in excavations in urban areas and high price that would have to be paid due to lack of analysis on construction sequence. This case study is therefore highlighting the importance and the value of this study and highlight the types of failures that would be prevented in future by accomplishment of the objectives of this study.

2.3.3 Case Study 3 – Structural Damage to Adjacent buildings during construction of Sheet Pile Canal bank protection near Galle Road bridge of Wellawatta Canal, Sri Lanka.

The report of “Damage assessment for the Structural distress occurred at Jinananda Children’s Home” carried out by Central Engineering Consultancy Bureau Organization (Annexure V) was referred for the information of this case study and details were further reviewed during the site visit to actually damaged site.

2.3.3.1 Case details.

Wellawatta Canal is a major drainage canal of the Colombo Canal system with sea outlet in Wellawatta area and the average canal width can be estimated as 24m . The width of this canal gets substantially contracted under the Galle Road bridge where the canal width is around 14m. Since these sudden canal contractions associated with higher head losses, it had been decided to provide flow guidance walls to streamline the flow to the contracted bridge section so that the head loss is minimal. These flow guidance walls were decided to be done with sheet piles and once the constructions were in progress, adjacent building was damaged severely (Refer Figure 2.26).

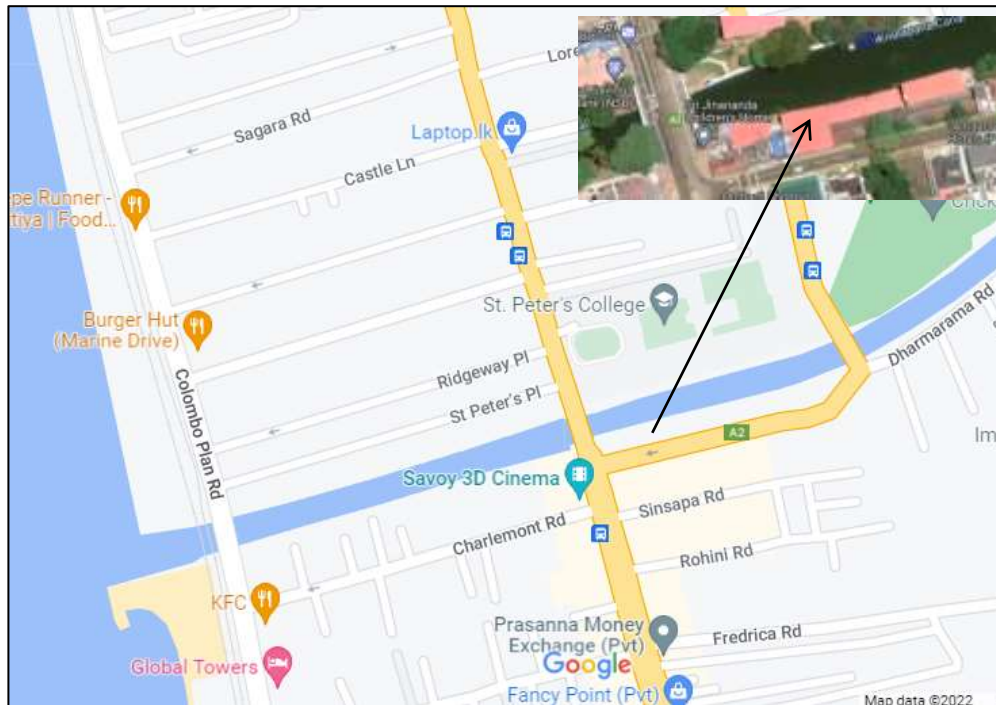


Figure 2.26: Location of the Damaged Building.

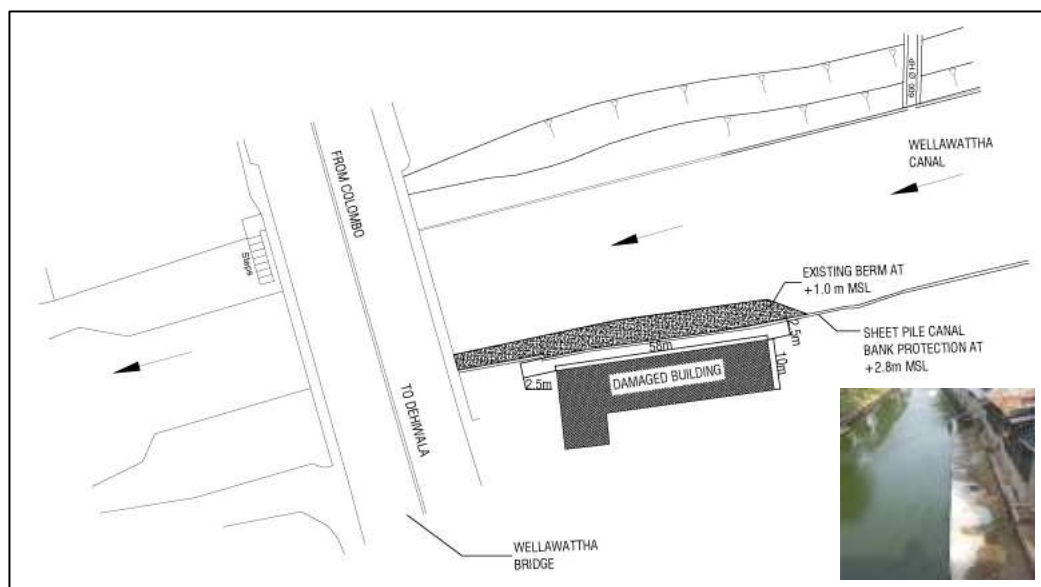


Figure 2.27: Plan view of the project area and distances to damaged building.

The building named as “Damaged Building” located on Dhammarama road, Wellawatta. This is a three storied building with a ground floor at existing road level of +8.8m MSL facing Dhammarama road with two basement levels of which the

lower basement is at the canal bank level of +2.8 mMSL and upper basement is at the level of +5.8m MSL. The building was using as an accommodation as well as educational and religious building for the children.

The dimensions of the building are such that it is 10m in width, 58m in length and 10m in height from lower basement level (Figure 2.27). The building has been constructed on a reinforced concrete frame structure with masonry walls. Timber roof with Calicut tiles were used as the roof. The actual as built drawings of the structure were unavailable and it had identified that some columns in the lower basement were built on shallow foundations constructed on quarry dust filled concrete cylinders, bored in to the hard soil strata. Beam spans ranges from 3.5m to 6m with varied beam depths while slab thicknesses were 150 – 200mm.

Canal bank protection design of the project area.

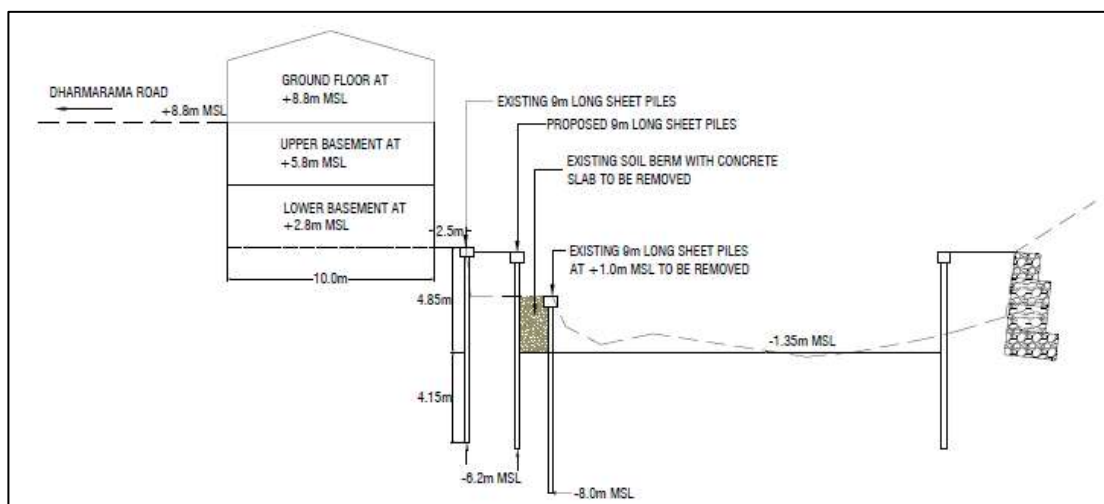


Figure 2.28: Cross sectional view of the canal bank protection design used at the site.

As shown in Figure 2.28 and Figure 2.27 the proposed canal bed level was at -1.35 mMSL and the top of the bank level of the canal left bank was +2.8 mMSL. There was a soil berm located at left bank at a level of approximately +1 mMSL. There was two rows of existing sheet piles such that one row at the existing canal bank level and one at berm level. The new sheet pile line for flow guidance was proposed to be constructed in between these two sheet pile lines across the berm. The row of sheet piles along the berm was proposed to be removed once the new sheet piles are constructed. . A segment of 3m in between the end of the exiting berm and end of the proposed sheet piles can be identified in figure 2.29. This segment can be considered

as an extremely critical length as this segment loose the passive pressure once the berm is removed and yet it is not protected by the new proposed sheet pile and thus there is a significant loss of resistive forces for this segment.

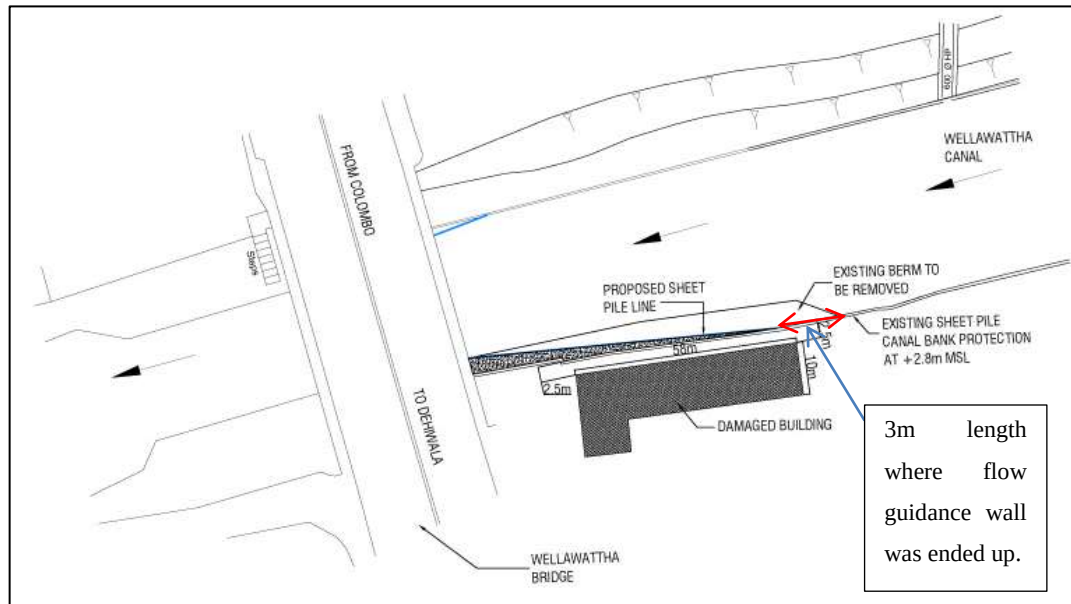


Figure 2.29: Plan view of the design.

Soil condition within the project area.

The Bore hole investigation had been conducted on the berm to get the soil profile within the project area. (Annexure VI)

Table 2.3: Summary of the bore hole information done on the berm

Layer No.	Depth/ m MSL	Layer Description	Avg. SPT N value
1	(+1.0) – (-0.5)	Fill Material	-
2	(-0.5) – (-2.5)	Loose Clayey Sand	2-9
3	(-2.5) – (-4.9)	Very Dense Clayey Sand	>50
4	(-4.9) – (-9.0)	Residual Soils	7-34

It was observed that a loose sand layer of 2.0m depth is overlaid by a dense sand layer up to 6.0m depth.

2.3.3.2 Nature of Damage

It was recorded that a significant modification to this building had been carried out from the design stage to the construction stage. Part of the building close to the canal had not been included during the design stage and it has been modified significantly during the construction stage.



a) Building during design stage



b) Building during construction stage

Figure 2.30: Damaged building condition during design stage and construction stage

This additional surcharge due to structure modification has not been taken in to consideration during the design and no analysis on construction sequence has been conducted

During the implementation, the new proposed set of sheet piles had been set up and tried to cut the berm sheet piles as per the construction methodology. However, once the berm sheet piles were cut off heavy ground settlements were appeared on the lower basements and numerous cracks were visible at multiple points of the structure (Figure 2.31). The lower basement (+2.8m MSL level) had been severely affected where the existing ground columns were tilted towards the canal due to unstable foundation condition (Figure 2.32).



Figure 2.31: Location of canal bank instability initiated.

It had been identified that 03 columns in the lower basement were severely damaged. The excessive ground settlements had occurred due to insufficient lateral support from the canal bank level existing sheet piles. Poor construction quality of the lower basement had also worsened the damage.



Figure 2.32: Photographs of cracks in the lower basement.

The cracks were appeared on structural elements of beams and slabs as well. Some of these cracks were occurred as a direct result of the ground settlements and some were believed to be enlargement of the existed hairline cracks.

The cracks appeared in the upper basement (at +5.8 m MSL level) were mainly on the walls due to differential settlements of the lower basement columns (Figure 2.33).



Figure 2.33: Photographs of cracks in the upper basement.

The major cause of the damage was found as the loss of lateral pressure from the berm and lost the stability. All construction works were temporality suspended after the collapse and widening or propagation of the cracks were monitored. The central engineering consultancy bureau were requested to assess the damage and submit a remediation plan .

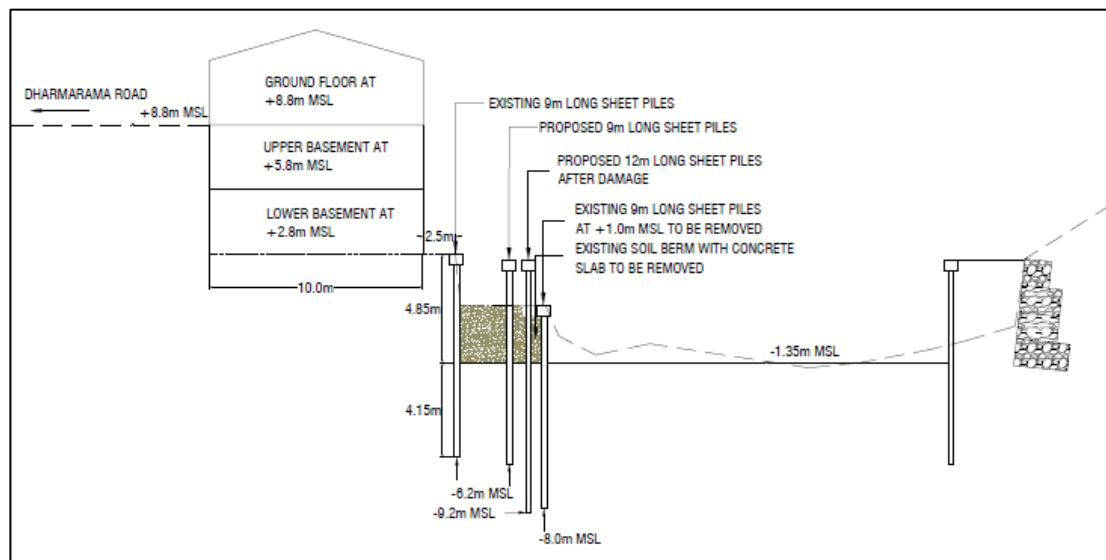


Figure 2.34: Cross sectional view of the canal bank protection design used at the site after damage.

Accordingly, another row of sheet piles with 12m in length were driven in between new sheet pile line and the berm to recapture the lost passive pressure of the berm (Figure 2.34). Also, this 12m length sheet piles extended sufficient length beyond the berm such that the area protected by the berm are protected by this new sheet pile line.

In addition to the cost of remediation sheet piles, an additional cost around 10M LKR had to be spent on building repair works in addition to the contractual problems and time delays.



Figure 2.35: Project site as seen today after the remediation sheet piles

2.3.3.3 Relevance of the case on this study.

However, it is evident that if any analysis on construction sequence is conducted on construction methodology prior to the actual implementation, this added surcharge would have considered and thereby this structural failure could have been forecasted. Furthermore, if the original design has underestimated any unfavorable loading, the analysis on construction sequence would still have captured unfavorable settlements and thereby, there would have been an opportunity to modify the original design avoiding the structural damages. Thus, the analysis on construction sequence adds an additional reinforcement to the original design. This case study can therefore be treated as a case where the damage could have been avoided had an analysis on construction sequence been conducted prior to the implementation. Therefore, this case study further establishes the value of this study on construction sequence for the future engineering projects involved with deep excavations in urban areas.

CHAPTER 3 - BACKGROUND OF FAILURE AND RESEARCH METHODOLOGY

3.1 Background of failure

The houses which have been damaged during the construction works of canal bank protection works in left bank of St. Sebastian North Canal is shown in Table 3.1.

Table 3.1: Details of damaged houses.

Reference No. in Layout Map.	House No.	Address.
P62	174	Nagalagam St., Colombo 14.
P61	172	Nagalagam St., Colombo 14.

Above mentioned houses and the project area are indicated in the Figure 3.1.

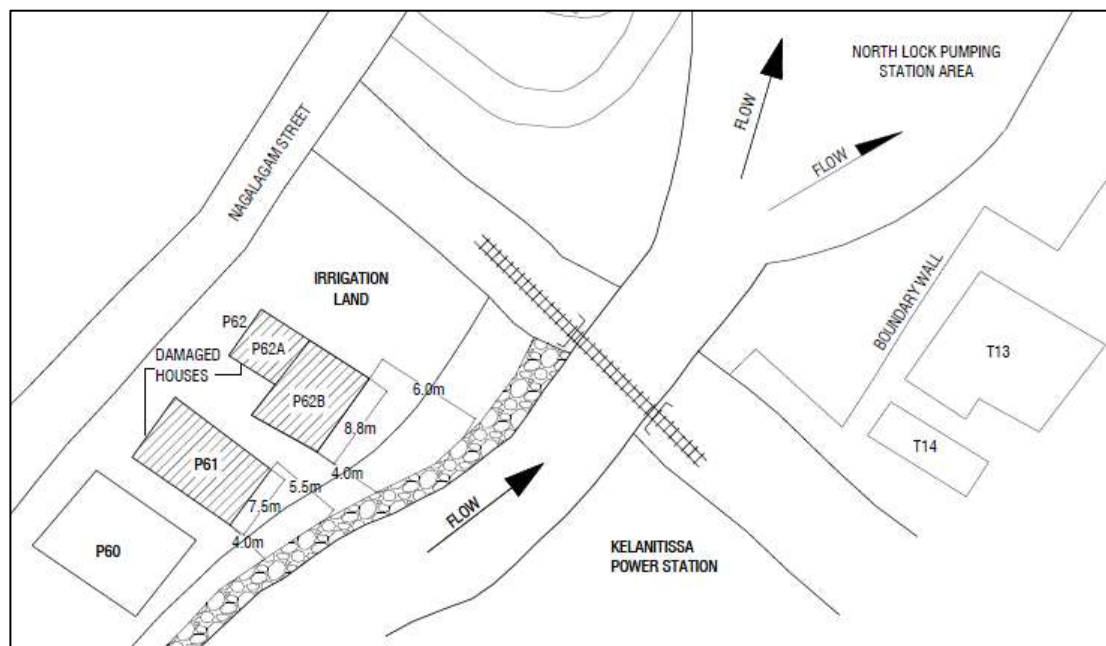


Figure 3.1: Layout Map showing the damaged houses and distances to the canal bank.

3.1.1 Canal bank protection design of the project area.

According to the design, it was planned to retain the left side of the canal bank using 4.5m height gabion wall structure where 1m x 2m depth gabion box is kept below the -1.0m MSL design bed level (Figure 3.2).

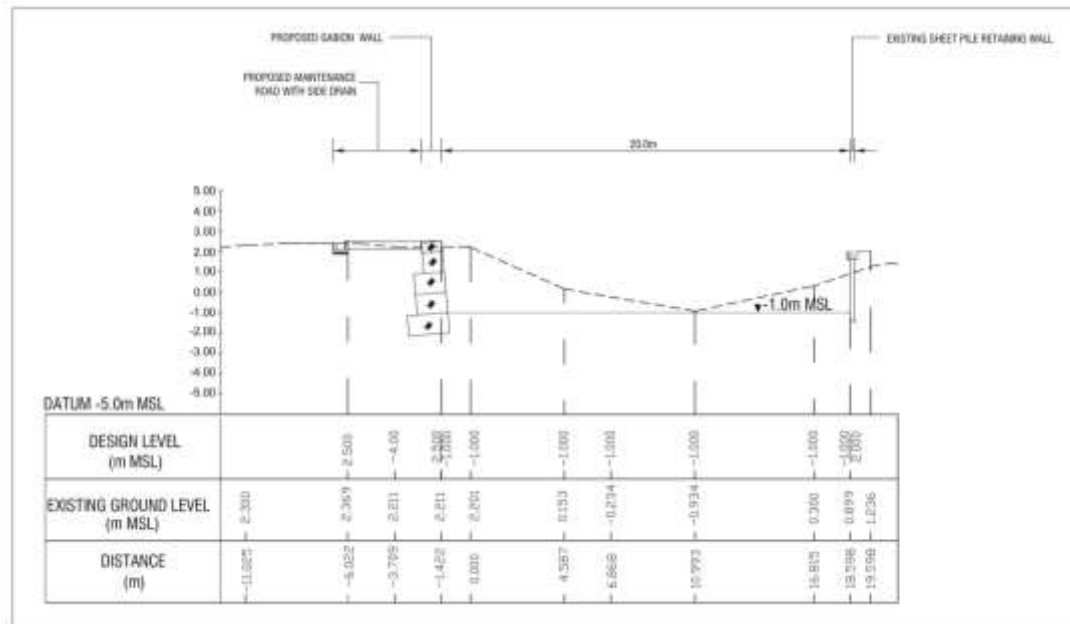


Figure 3.2: Design Section of Canal Bank Protection.

3.1.2 Temporary shoring arrangement used at the site.

For the application of design section to the actual site, it was decided to have a temporary shoring arrangement using 6m length sheet piles with H iron bracings at top level to retain the bank during gabion wall construction works. Sheet piles were driven upto -3.5m MSL and depth of excavation was 4.5m. The system adopted in the site is presented in Figure 3.3 and Figure 3.4.

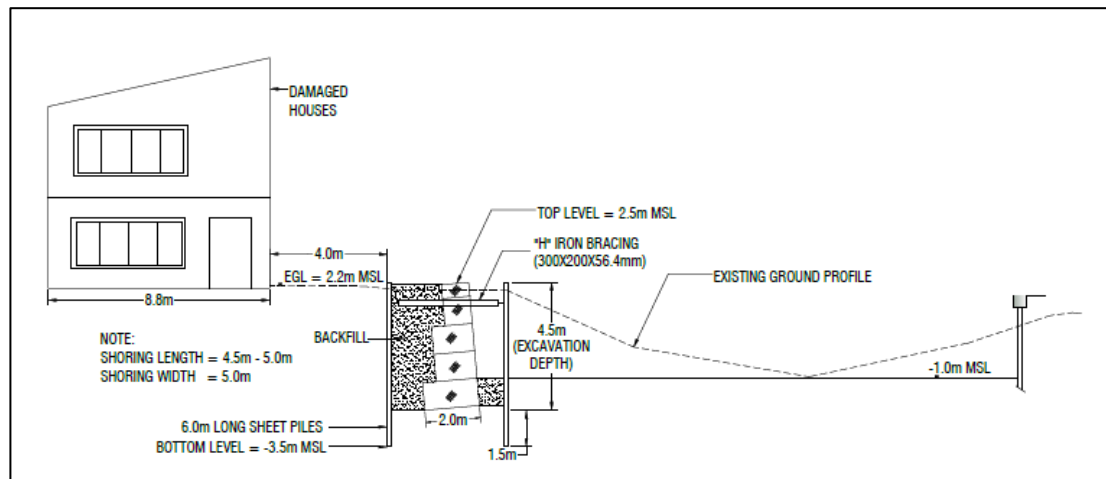


Figure 3.3: Section showing the Temporary Shoring Arrangement Used at the site.

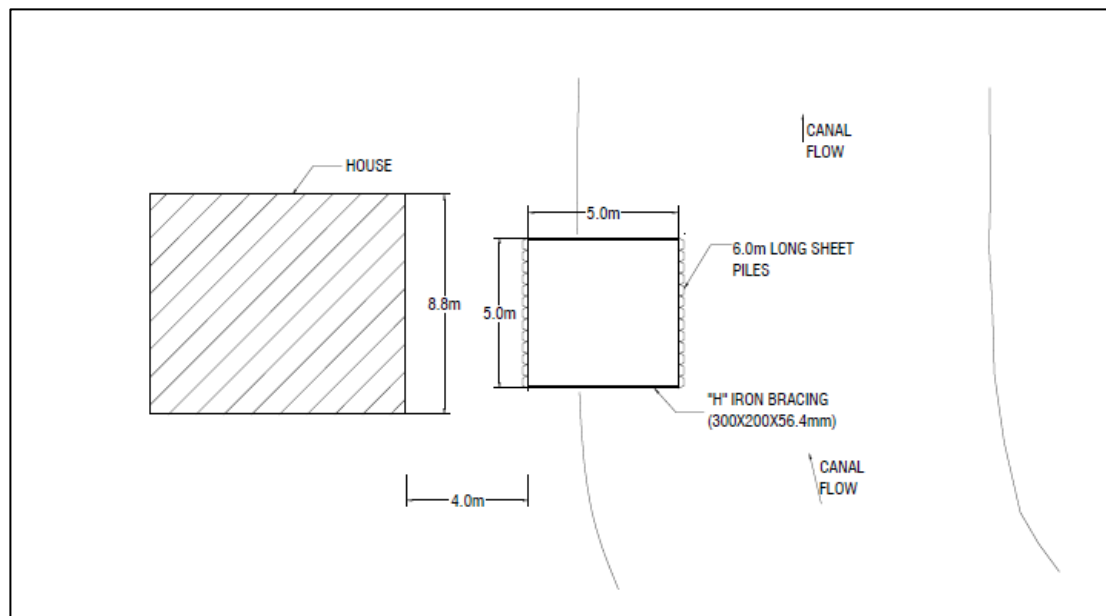


Figure 3.4: Plan view showing the Temporary Shoring Arrangement Used at the site.

3.1.3. Construction Methodology used at the site.

The construction methodology followed at the site can be described as follows.

Stage 01 - Excavation and removing of 0.5m soil layer from existing ground and setting out was done for temporary shoring area of 5m length stretch.

Stage 02 - Installation of light weight shoring with 6m length sheet piles and prop at top level.

Stage 03 - Excavation, dewatering and removing excavated material up to -2 m MSL, design bottom level of gabion wall.

Stage 04 - Dewatering and placing of gabion boxes with geotextile cloth of 1st row which finishes at -1m MSL.

Stage 05 - Dewatering and placing of gabion boxes with geotextile cloth of 2nd row which finishes at 0m MSL.

Stage 06 - Dewatering and placing of gabion boxes with geotextile cloth of 3rd row which finishes at +1m MSL.

Stage 07 - Backfilling and removing the temporary shoring.

Stage 08 - Continue the shoring for next 5m length.

As per the construction methodology, the temporary sheet piles have to be pulled off once the gabion walls are constructed up to +2.5 m MSL. But the temporary sheet piles have been pulled off at +1m MSL level of gabion wall construction resulting heavy movement of embankment causing structural damages to nearby buildings. It was observed that the rear wall of the House Ref. No. P62 has tilted towards the area of bank of the canal (Figure 3.5).



Line of Soil
Movement towards
the Canal.

Figure 3.5: Ground movement at the back side of the House Ref No. P61.

3.1.4 Condition details of House Ref. No. P62.

The house referred as P62 in above figure 3.1, was built 15 years ago using Random Rubble Masonary shallow foundation and 150mm thick walls with cement blocks. The roof has been constructed as a concrete reinforced slab.

The front part of the P62 house which face to Nagalagam Street has been demarcated as P62A in Figure 3.6 and that part was used as a steel formwork storage yard. The back part of the P62 house which face to St. Sebastian North Canal has been demarcated as P62B in Figure 3.6 and that part was used as the house structure of the owner.



(a) P62A – Front view



(b) P62B – Back view

Figure 3.6: Front view and back view of P62 house.

3.1.3.1 Observations

During the construction works of the gabion canal bank protection were in progress, the back part of P62 house had detachments mainly at two locations as shown in figure 3.7. Detachment 1 was observed in the floor and Detachment 2 was observed in the wall.

It was to be mentioned that there were no records about those detachments in the structure during the pre condition survey. So that, it was clear that the back part of P62B house has been moved towards the St. Sebastian North Canal.

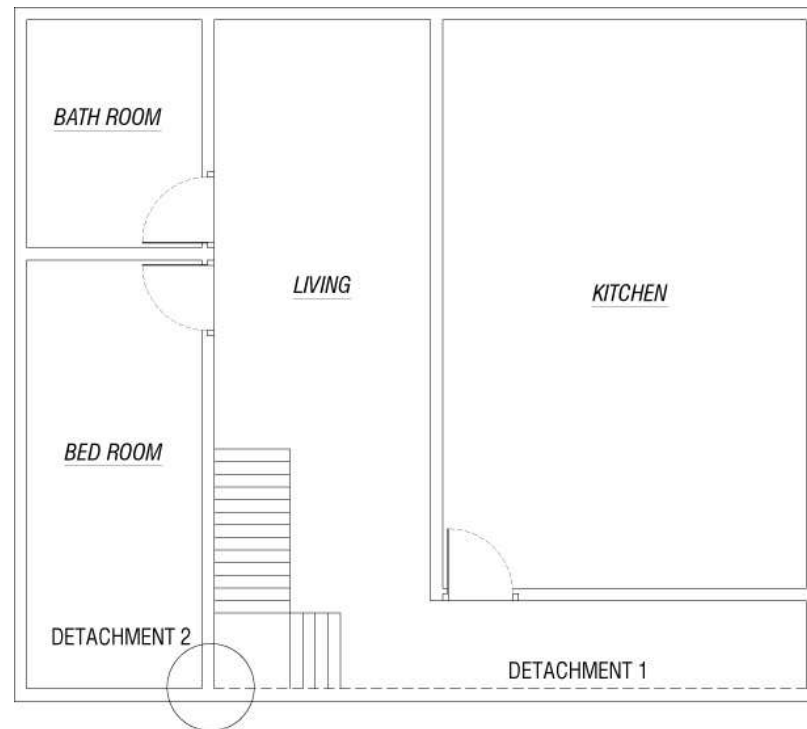


Figure 3.7: Ground Floor Plan of the damaged P62B house and detached locations.



Figure 3.8: Detachment -1 occurred in the floor.

As shown in Figure 3.8, the detachment -1 was observed in closer to P62A and P62B dividing wall. The extent of detachment was 53.54mm which was occurred during the construction of gabion walls were in progress.



Figure 3.9: Detachment – 2 observed in the wall.

As shown in figure 3.9, Detachment -2 was observed in the internal wall which is closer to the staircase. The extent of detachment was 223 mm which was occurred during the construction of gabion walls were in progress.

3.1.4 Condition details of House Ref. No. P61.

The house referred as P61 in Figure 3.10, was built 7 years ago using Random Rubble Masonary shallow foundation and 225mm thick walls with burnt clay bricks. The roof has been constructed as a concrete reinforced slab.



Figure 3.10: Front view of P61 house from Nagalagam Street.

3.1.4.1. Observations

Mainly two cracked locations were observed in back side of P61 house as shown in Figure 3.11. C1 was a floor crack appeared in the kitchen and C2 was a wall detachment appeared in the space in between bedroom and bathroom.

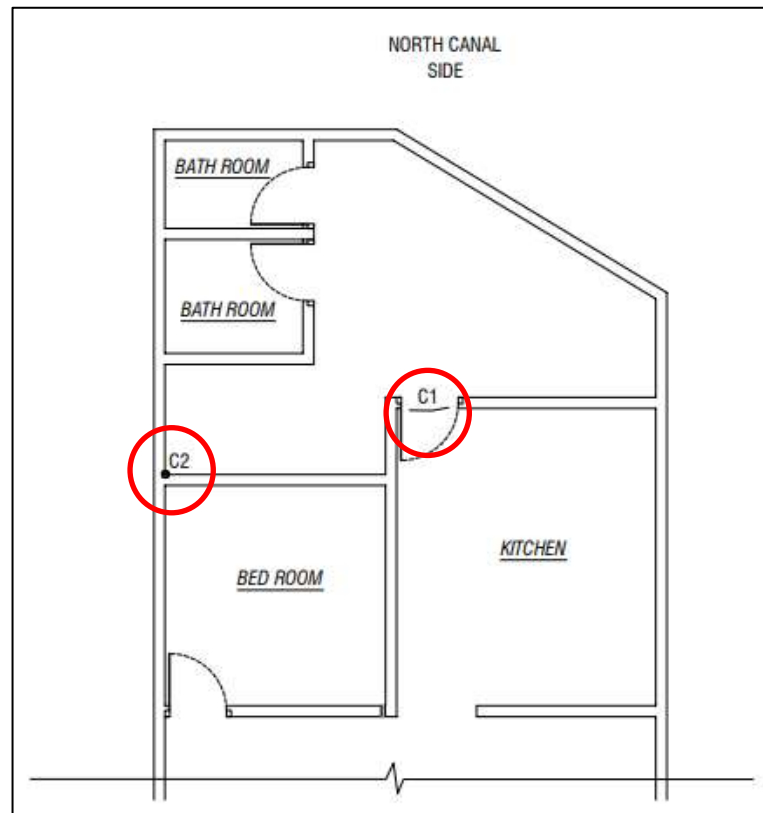


Figure 3.11: Back view plan of P61 house from Canal side.



Figure 3.12: C1 floor crack.

No records were available regarding this crack in Pre-Condition Survey Report.

Measured Crack Width on 23.01.2019 was 16.46mm. This crack has been appeared at the rear side of the House Ref. No. P61. The crack which is depicted in above photograph has widened horizontally and displaced vertically. Through the observations, it can be determined that this part of the House Ref. No. P61 which is cracked along the line as depicted in the photograph is moving towards the canal gradually.

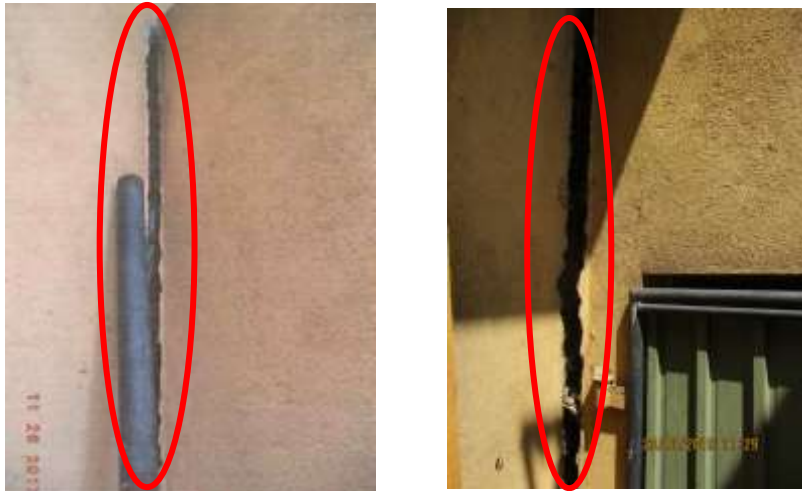


Figure 3.13: Comparison of the C2 crack recorded in the Pre-Condition Survey on 26.11.2017 and the Condition Survey on 23.01.2019 of the House Ref No. P61

The initial crack width has not been recorded during the Pre-Condition Survey on 26.11.2017 and only the photographic record was done. But, according to the photographic records of both condition surveys, it can be concluded that the crack width has widened gradually. The recorded Crack width on 23.01.2019 was 58.77mm.

3.2 Methodology

The following hypotheses were set up before the analysis and the research methodology and analysis has been lined up to highlight the fulfilment or nonfulfilment of these hypotheses and thereby approach the conclusions and the study outcomes.

- (a) The structural damages discussed in the section 3.1 has caused due to the excavation.
- (b) The above structural damages could have been prevented had there been a proper analysis and corrective measures implemented.
- (c) The analysis on construction sequence followed in this study could be used as a specimen for the constructions involved with deep excavations in urban areas and therefore recommended the initiation of actual construction only after the construction sequence analysis report.
- (d) The main drawback is in the excavation support system.

The actual method adopted at the site will be analysed and since the temporary support system actually used had not been effective in keeping the displacements under control, three alternative systems were analysed.

System 02 - Temporary shoring system with 6m long sheet piles with 3 props at 1.5m intervals.

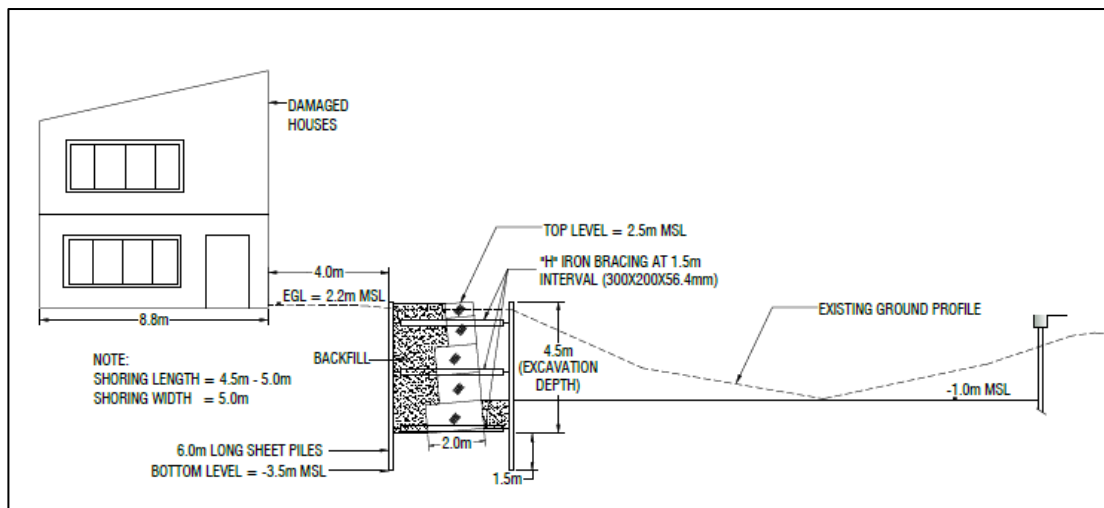


Figure 3.14: Schematic diagram showing the System 02 arrangement.

- (i) System 03 - Temporary shoring system with 9m long sheet piles with only 01 prop at the existing ground level.

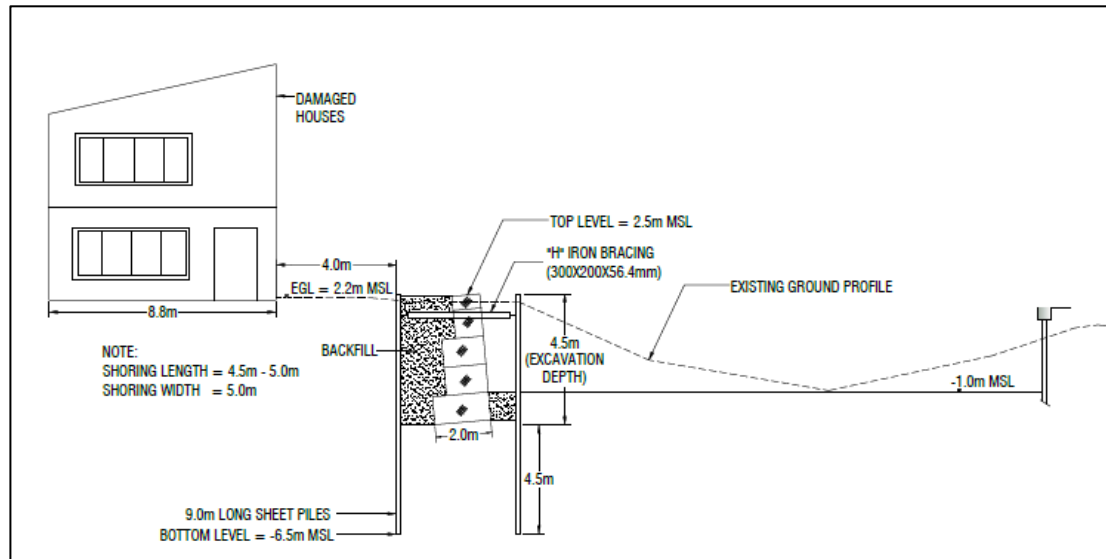


Figure 3.15: Schematic diagram showing the System 03 arrangement.

- (ii) System 04 - Temporary shoring system with 9m long sheet piles with 3 props at 1.5m intervals.

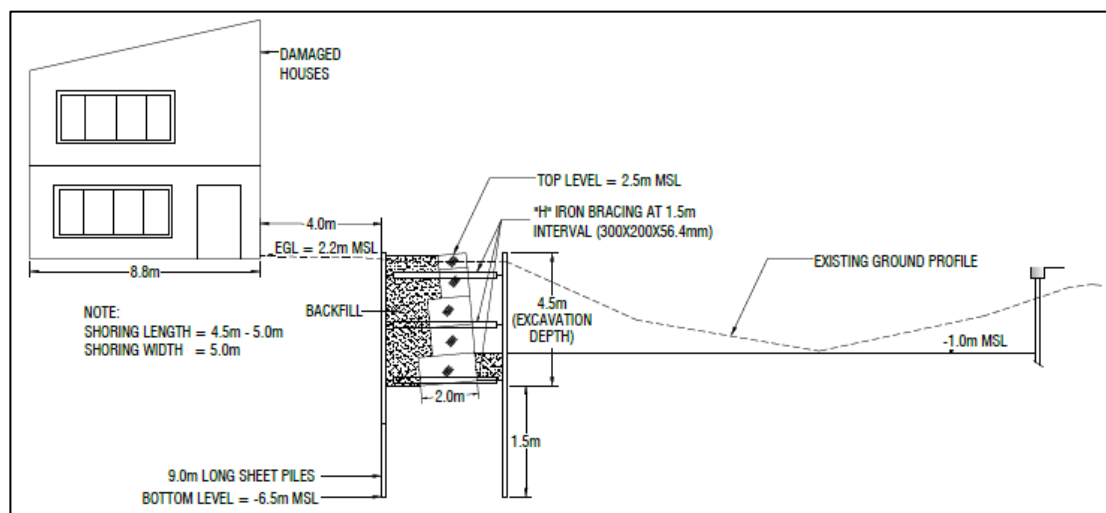


Figure 3.16: Schematic diagram showing the System 04 arrangement.

Each of these construction sequences consist of number of construction stages and the details of these construction stages are discussed in detail in chapter 04.

Once all the stages with respect to each construction sequence are identified, following analysis steps were carried out.

1. Identification of all the Construction stages of 4 construction sequences to be analyzed.

In this step, all the dimensions, levels, water levels and construction activities are identified.

2. Simulation of the construction process by Finite Element Analysis using PLAXIS 2D commercial package.

Finite Element Analysis was widely used for simulation of construction processes and comparatively accurate method for an analysis of this nature. Therefore, PLAXIS 2D was selected as the software for the Finite Element Analysis and

3. Exploring and analyzing the results with the help of graphs and diagrams.

The vertical settlement, horizontal movement and wall deformation were observed for each stage of each sequence. Subsequently, the maximum deformations of each stage were plotted along the construction stages to identify the maximum stage deformation and critical stage which might violate the allowable settlements.

The sub soil properties were obtained from site investigation reports and prepared a table of soil properties prior to the analysis as shown in Table 5.1 prior to finite element analysis.

It is a common practice that design engineers put more emphasis on the final design to be more conservative and without adequate concerns on the analysis of the system for different stages of the construction sequence. Hence a comprehensive analysis was carried out for various loading conditions with respect to the stages of the construction sequence.

Due to the nonlinear, anisotropic nature of the sub soil and the analysis is time depended due to the varied conditions with respect to different construction stages, a 2D finite element method using PLAXIS was used for the analysis. PLAXIS 2D is a

widely used software for 2D simulation for deformation and stability analysis for multiple construction stages considering steady groundwater flow for saturated or partially saturated conditions.

The PLAXIS 2D simulation was carried out for multiple stages along the construction sequence for three different shoring systems. The details of the stages of the construction sequence and the details of the tested shoring systems are stated in sections 6.1 and 6.2 respectively. The settlement plots are generated for each of the stage for each of the shoring system and the system whose settlements are within allowable limits are selected and the suitable shoring system and it is the system which should have been implemented.

CHAPTER 4 MODELLING ALTERNATE CONSTRUCTION PROCESSES.

The construction sequence involved in the placement of the gabion walls by the embankment of the canal was simulated using the PLAXIS 2D finite element package.

As outlined in the methodology, 4 systems were modelled including actual construction process adopted.

1. System 01: Actual process adopted
2. System 02: Temporary shoring system with 6m long sheet piles with 3 props at 1.5m intervals.
3. System 03 - Temporary shoring system with 9m long sheet piles with only 01 prop at the existing ground level.
4. System 04 - Temporary shoring system with 9m long sheet piles with 03 props at 1.5m intervals.

4.1 Soil Condition along the canal bank protection area

The Mohr-Coulomb (MC) model is a basic constitutive model for soil. This model assumes the stress – strain relation to be linear elastic and its failure criterion is Mohr-Coulomb's failure criterion. Hence, soil was modeled using Mohr-Coulomb (MC) model for this study.

The MC model involves five input parameters, I.e. the soil friction angle, cohesion, Young's modulus, Poisson's ratio and dilatancy angle.

In all cases drained material was selected, so the soil friction angle and the cohesion are involved.

For sands with $\phi' \leq 30^\circ$:

$$\psi' = 0^\circ$$

For sands with $\phi' > 30^\circ$:

$$\psi' = \phi' - 30^\circ$$

*Table 4.1: Typical values for Soil friction angle and cohesion***Table 3.10** ϕ' for some soils and rocks as suggested by AS 4678-2002

	Material	$\phi' (^{\circ})$
Soils	Soft and firm clay of medium to high plasticity, silty clays, loose variable clayey fills, loose sandy silts (use $c' = 0-5$ kPa)	17-25
	Stiff sandy clays, gravelly clays, compacted clayey sands and sandy silts, compacted clay fill (use $c' = 0-10$ kPa)	26-32
	Gravelly sands, compacted sands, controlled crushed sandstone and gravel fills, dense well graded sands (use $c' = 0-5$ kPa)	32-37
	Weak weathered rock, controlled fills of roadbase, gravelly and recycled concrete (use $c' = 0-25$ kPa)	36-43
Rocks	Chalk	35
	Weathered granite	33
	Fresh basalt	37
	Weak sandstone	42
	Weak siltstone	35
	Weak mudstone	28

*Table 4.2: Typical values for Young's modulus and Poisson's ratio***Table 3.16** Typical values of E_u for clays

Clay	E_u (MPa)
Very soft clay	0.5-5
Soft clay	5-20
Medium clay	20-50
Stiff clay, silty clay	50-100
Sandy clay	25-200
Clay shale	100-200

After U.S. Army (1994) and Bowles (1986)

Table 3.17 Typical values of Poisson's ratio

Material	Poisson's ratio
Saturated clays (undrained)	0.5
Saturated clays (drained)	0.2-0.4
Dense sand	0.3-0.4
Loose sand	0.1-0.3
Loess	0.1-0.3
Ice	0.36
Aluminum	0.35
Steel	0.29
Concrete	0.15

Bowles (1986), Kulhawy and Mayne (1990), and Lambe and Whitman (1979)

Table 4.3: Typical values for bulk and saturated unit weights

Table 2.2 Typical values for bulk and saturated unit weights					
		Bulk unit weight (kN/m ³)		Saturated unit weight (kN/m ³)	
		Loose	Dense	Loose	Dense
Granular soils	Gravel	16.0	18.0	20.0	21.0
	Well graded sand and gravel	19.0	21.0	21.5	23.0
	Coarse or medium sand	16.5	18.5	20.0	21.5
	Well graded sand	18.0	21.0	20.5	22.5
	Fine or silty sand	17.0	19.0	20.0	21.5
	Rock fill	15.0	17.5	19.5	21.0
	Brick hardcore	13.0	17.5	16.5	19.0
	Slag fill	12.0	15.0	18.0	20.0
	Ash fill	6.5	10.0	13.0	15.0
Cohesive soils	Peat (high variability)	12.0		12.0	
	Organic clay	15.0		15.0	
	Soft clay	17.0		17.0	
	Firm clay	18.0		18.0	
	Stiff clay	19.0		19.0	
	Hard clay	20.0		20.0	
	Stiff or hard glacial clay	21.0		21.0	

After AS 4678-2002

Table 4.4 lists the input parameters of soil layers for the MC model used in this study.

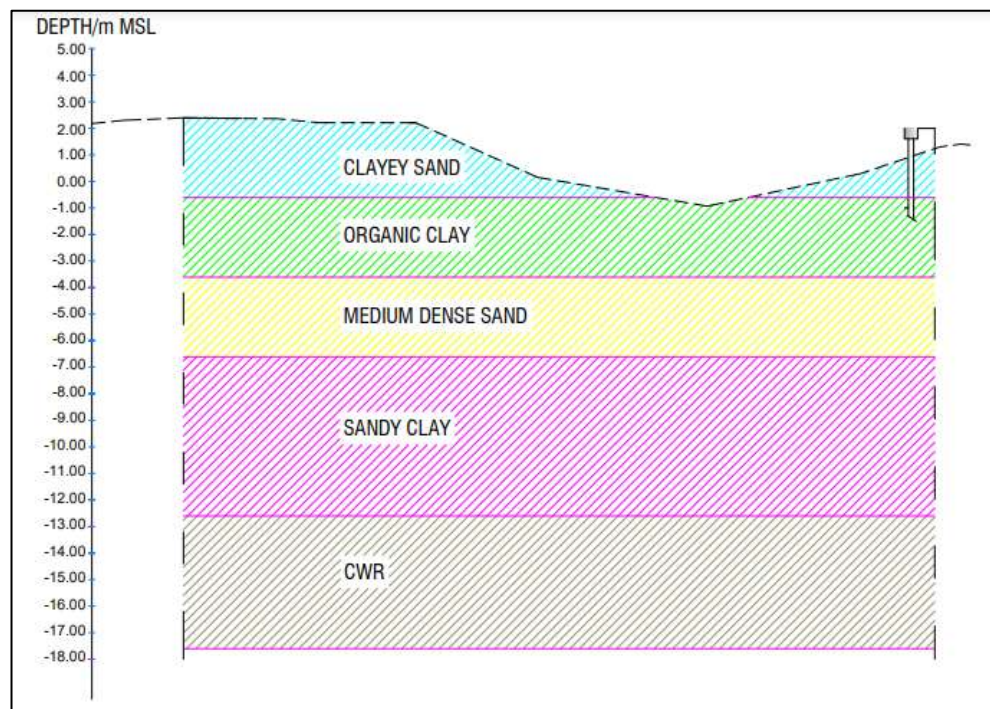
*Figure 4.1: Soil Profile of the site.*

Table 4.4: Input parameters for the soil layers for the MC model.

Depth/ (m)		Soil Description	Avg N	Y_{wet} (kN/m ³)	Y_{sat} (kN/m ³)	C' (kPa)	ϕ'	E (kPa)	ν'
From	To								
0	3	Clayey Sand	2	15	16	3	22	3000	0.35
3	6	Organic Clay	2	15	15	3	20	2000	0.35
6	9	Medium Dense Sand	10	18	18	0	28	16000	0.3
9	15	Sandy Clay	23	19	19	5	29	32000	0.3
15	20	CWR	40	21	21	8	35	64000	0.25

The Classification of Soil with respect to different soil layers were obtained from soil investigation report and the drained soil parameters for friction angle, cohesion, Young's modulus and Poisson's ratio and saturated and bulk unit weights of each layer were obtained by referring the typical values presented by Prof. Braja M. Das in the book "Correlations of Soil and Rock Properties in Geotechnical Engineering: as suggested by AS 4678-2002 and shown in Table 4.1, Table 4.2 and Table 4.3.

The complete set of estimated soil parameters for each layer to be used for Finite Element Model are shown in Table 4.4.

Plate elements were used to represent the sheet pile shoring system, and node-to-node anchor elements were used to simulate the steel props. Sheet piles and steel props were both simulated using the linear elastic model. Section modulus and Poisson's ratio are the two input parameters for this model. For both sheet piles and steel props, the Poisson's ratio was assumed to be 0.2. The Section modulus of sheet piles were obtained using the technical specifications for FSP IV sheet piles (Annexture VII) and Section modulus of steel props were obtained from BS 4-Part 1-1993, Structural Steel Sections (Annexture VIII).

The friction between the earth and the sheet pile wall was represented by the interface elements. According to Khoiri and Ou,2013, the strength reduction factor of interface elements, R_{inter} , might be set to 0.67 to represent ground disturbance between wall and soil.

Table 4.2 and Table 4.3 present the input parameters of the sheetpile wall and steel props used in the study.

Table 4.5: Input parameters of sheetpile wall

Parameter	Value	Unit
Width, w	400	mm
Height, h	170	mm
Thickness, t	15.5	mm
Sectional Area	96.99	cm ² /pile
Weight	76.1	Kg/m
Moment of Inertia, I	38600	cm ⁴ /m
Section Modulus, E	2270	cm ³ /m
Axial stiffness, EA	4.85x10 ⁶	kN/m
Flexural stiffness, EI	7.72x10 ⁴	kNm ² /m
Equivalent Thickness,d	0.437	m
Weight,w	1.864	kN/m/m
Poisson's ratio,v	0.2	

Table 4.6 : Input parameters of steel props

Parameter	Value	Unit
Section designation	305x305x137	
Mass per unit length	136.9	kg/m
Sectional Area, A	176	cm ²
Sectional Modulus, E	205000	cm ³ /m
Axial stiffness, EA	2x10 ⁶	kN
L _{spacing}	5	m

4.2 Boundary Conditions

When selecting the model sizes the boundary conditions such as the prescribed deformation and forces become important considerations (PLAXIS, 2002). Too small size of geometry may lead to wrong results, as they would be highly influenced by the boundary conditions. In order to avoid that, horizontal and vertical dimension of models were selected as 40m and 25m respectively. Plaxis allows entering model boundary conditions based on known loads. Of all the options available, only the "standard fixities" option was used. Assigning fixity enables us to keep a zero displacement in both vertical and horizontal axis ($U_x = U_y = 0$).

4.3 Water table.

The normal water level of the study area is +0.6 mMSL. The water table of the analysis was adjusted in the model to suit with the ground excavation.

4.4 Sequence of Analysis

In order to investigate the effect of the construction sequence, 4 systems were analyzed in PLAXIS 2D. Calculations for the multi phases numerical analysis were performed using the staged construction procedure. The calculations were executed in several phases including the sheet pile installation, prop installation at each interval, excavation, dewatering, gabion wall construction and backfilling ending with the state where all finite element model components including surcharge

4.4.2 System 02: 6m long sheet piles with props at 1.5m interval.

System 02 is an alternative construction process which have been proposed since the actually adopted construction process illustrated in system 01 was caused structural damages for the neighboring houses.

System 02 is an improvement of System 01 where 6m long sheet piles were used and number of props have been increased upto 3 installed at 1.5m vertical distances.

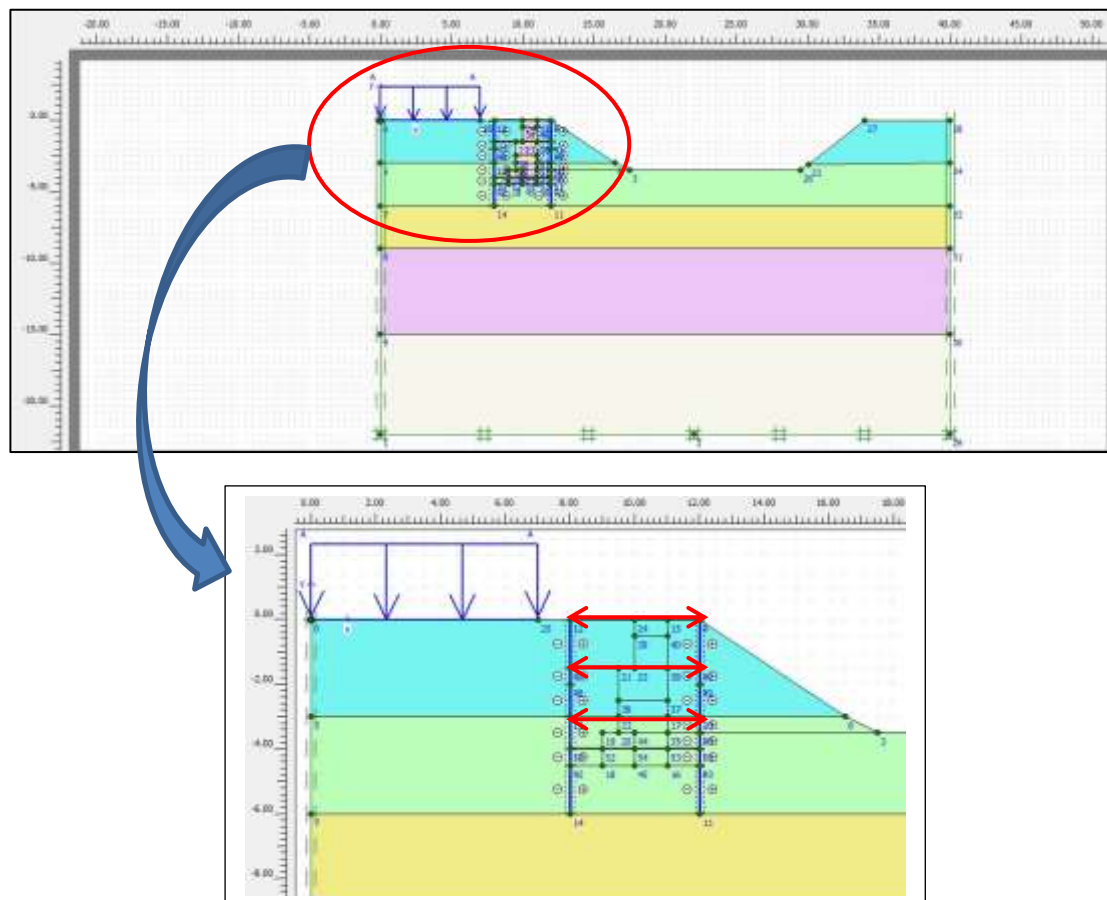


Figure 4.4: Proposed system 02.

4.4.3 System 03: 9m long sheet piles proped at existing ground level.

System 03 was proposed by enhancing the length of the sheet piles from 6m to 9m and it was proped at existing ground level using “H” iron bracings. This is another type of alternate construction process instead of System 01 where sheet pile length have been increased considering the excavation depth and embedment depth.

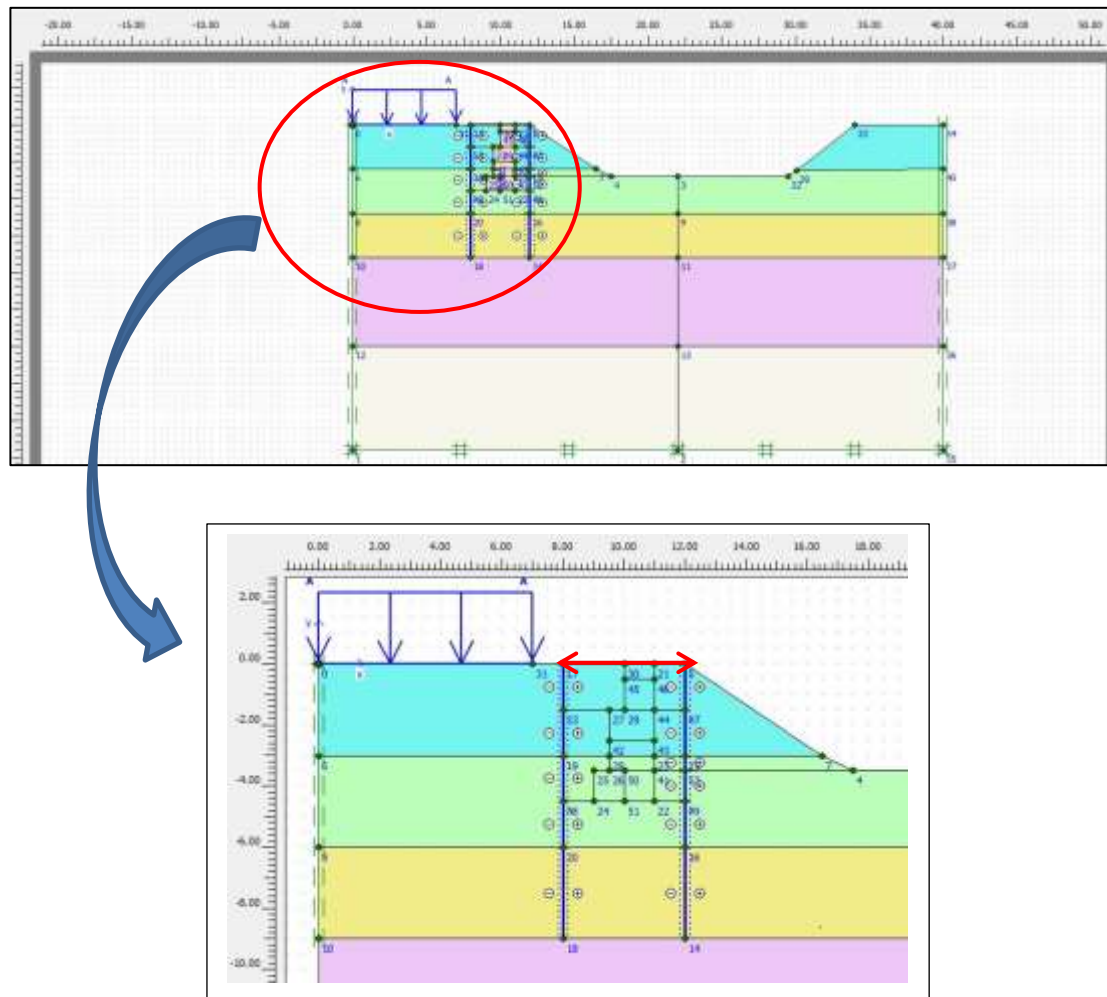


Figure 4.5: Proposed system 03.

4.4.4 System 04: 9m long sheet piles with 3 props at 1.5m intervals.

System 04 was another type of alternative construction process which is having 9m long sheet piles and propped at 3 levels of 1.5m vertical distance. This alternative construction process mainly proposed due to the closed proximity to the dwelling houses. By considering the soil profile of the site, excavation depth and distances to the nearby buildings, System 04 was proposed to minimize the damages which can be occurred

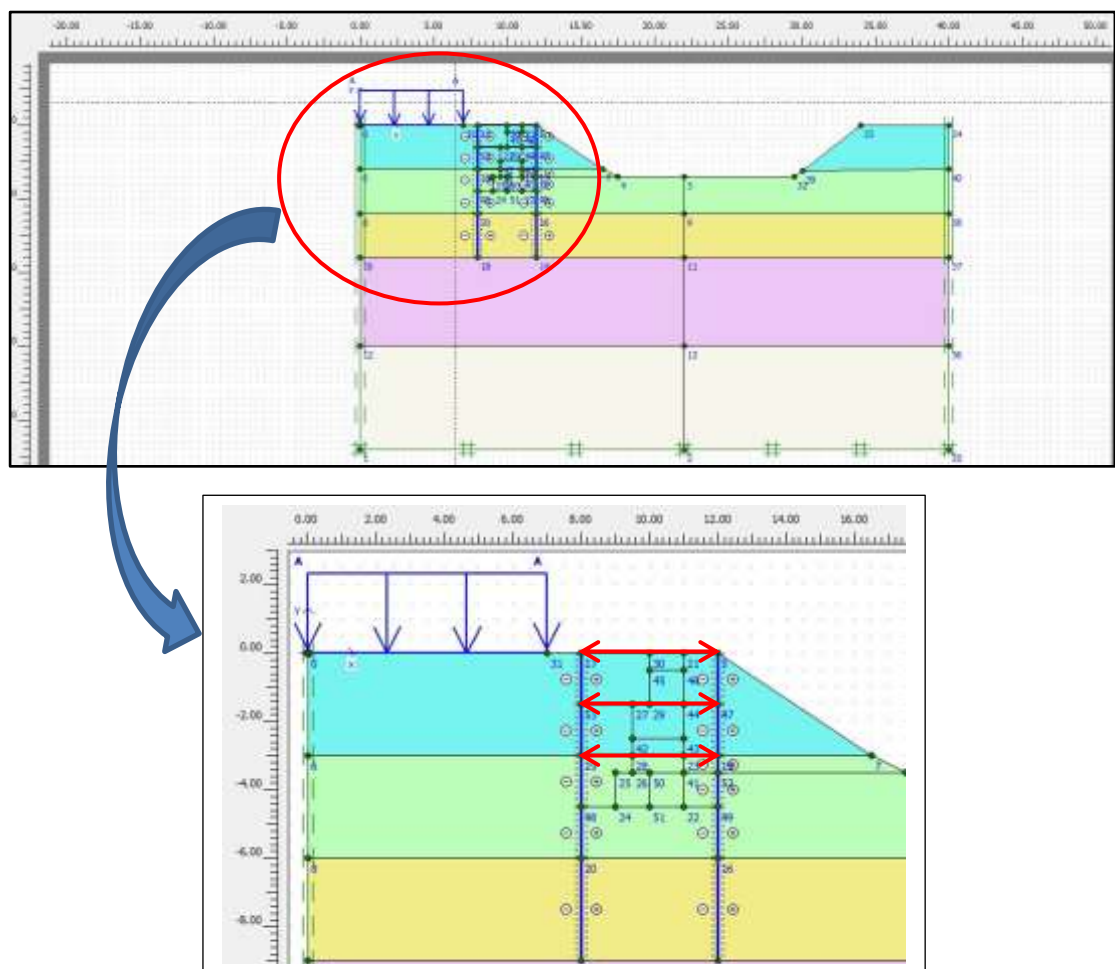


Figure 4.6: Proposed system 04.

4.5 Results.

4.5.1 System 01

Table 4.4 shows the construction sequence used in the PLAXIS model for system-1.

The Plaxis model for system – 1 includes 10 stages of construction. It included major construction steps in an excavation and dewatering process such as sheet pile installation, steel prop installation, excavation, dewatering, gabion construction, and backfilling. Steel props were installed at 0.0m depth.

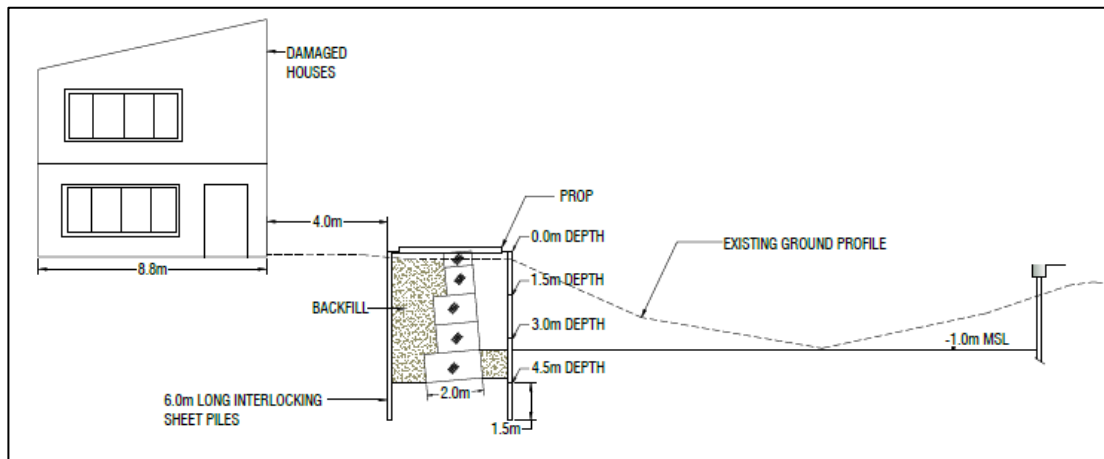


Figure 4.7: Actually adopted Shoring system used for the Plaxis Model .

The stages of the plaxis model which represents the actually adopted construction sequence can be described as below.

At the first stage, 6.0m long sheet piles were installed for 5m length. Thereafter, 300x200x56.4mm “H” iron had been propped at 0.0m depth level. The excavation process were started by initially excavation of 1.5m depth from existing ground level (0.0m depth level). After that, another 1.5m depth had been excavated as the second stage of excavation and further 1.5m had been excavated as the third stage of excavation which reaches the bottom level of the gabion i.e 4.5m depth level while dewatering.

Then the construction works were started meanwhile the dewatering process was ongoing. As the first stage of construction, 2mx1m gabion box was laid and backfilled at the canal side and land side around the gabion. Then two number of 1.5mx1m gabion boxes were laid while the dewatering process was ongoing. Backfilling of the land side was done paralally up to 1.5m depth level. After that

prop and the sheet piles were removed and continued to next 5m without completion of gabion construction works.

Table 4.7: Construction sequence used for system - 1

Stage Identification	Description
Stage 01	Installation of Sheet Piles
Stage 02	Install Prop at 0.0m depth level
Stage 03	Excavation of 1.5m depth – upto 1.5m depth level
Stage 04	Excavation of 1.5m depth – upto 3.0m depth level
Stage 05	Excavation of 1.5m depth – upto 4.5m depth level
Stage 06	Laying 1 st row of gabion (2mx1m)
Stage 07	Backfilling around the gabion
Stage 08	Laying 2 nd and 3 rd rows of gabion (1.5mx1m)
Stage 09	Backfilling around the gabion
Stage 10	Remove prop and 6m long sheet piles

The ground settlement profile for each stage of construction from the edge of sheet pile (at R) towards 7m distance in land side (at P) is obtained from the Plaxis model and plotted as shown in Figure 4.8. The vertical line at 3m (at Q) indicates the boundary of the damaged house.

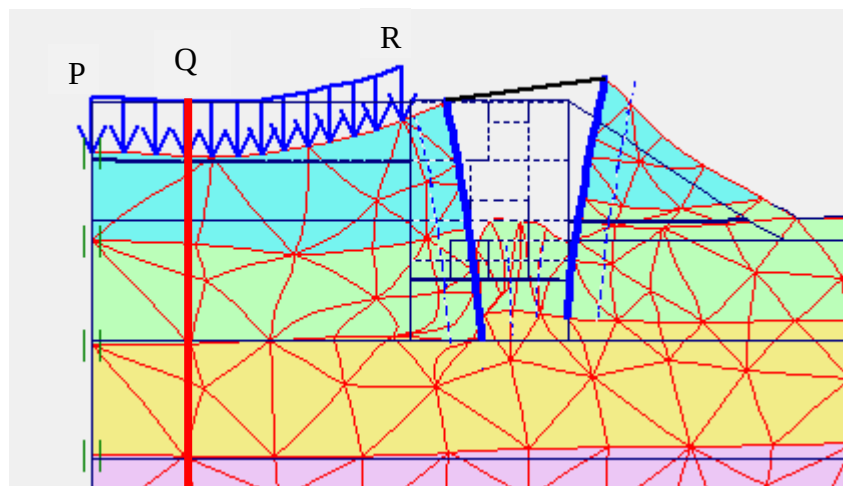


Figure 4.8: Deformed mesh of stage 05 of System 01.

For the simplicity of understanding the each P, Q and R locations, the deformed mesh of stage 05 was taken from the Plaxis model and shown in Figure 4.8.

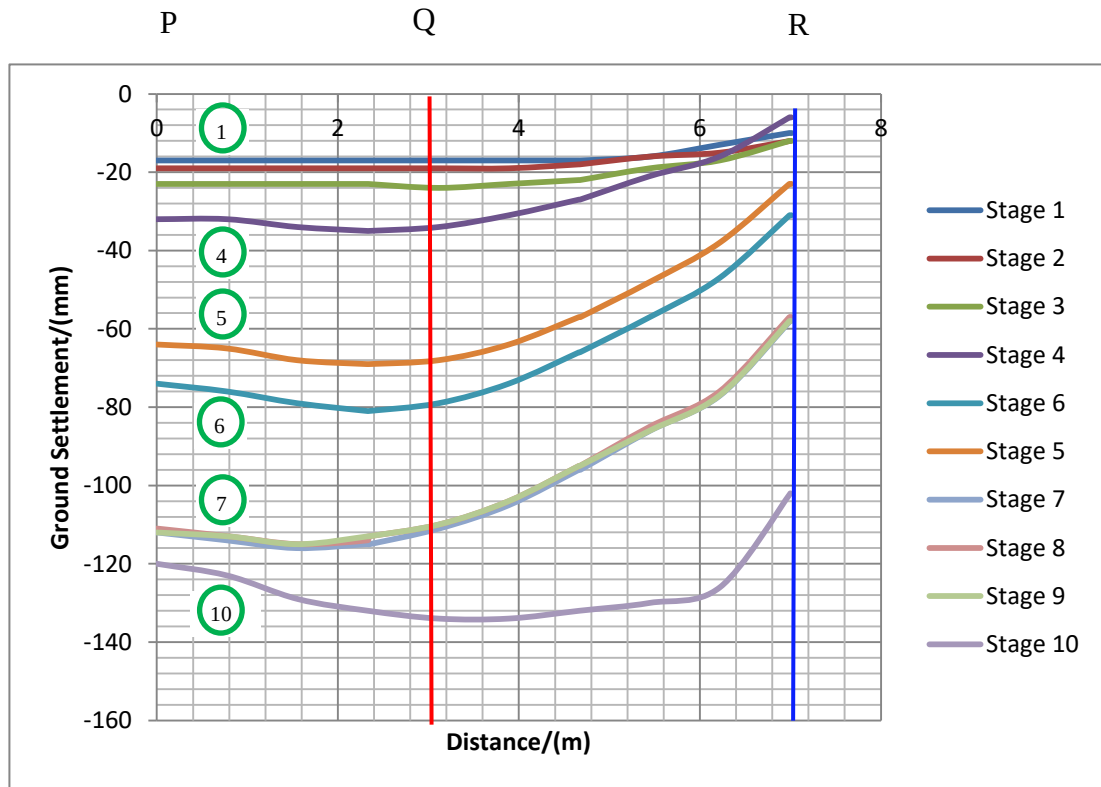


Figure 4.9: Ground settlement profile of System 01.

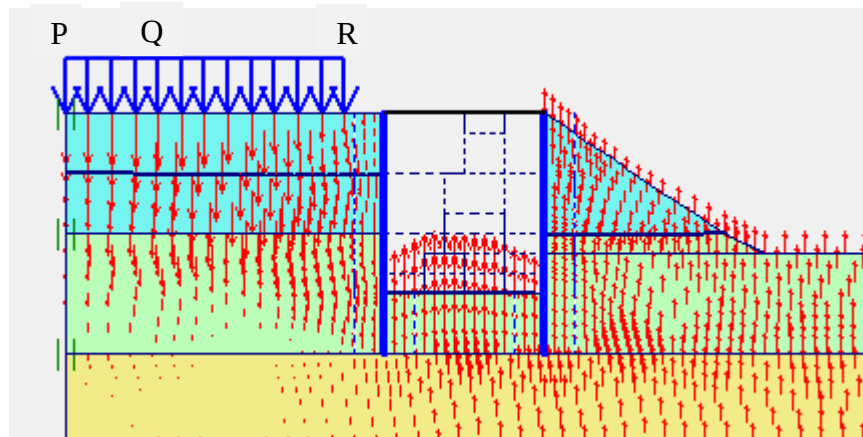


Figure 4.10: Ground Settlement Profile of stage 05 of System 01.

Accordingly, at the first stage, consolidation settlement due to the surcharge, 20mm ground settlement can be observed closure to the damaged structures (at Q) as shown in Figure 4.9

Ground settlements was gradually increased to 30mm up to 4th stage where excavation has been done to 3.0m depth level. Further excavation of 4.5m depth level in 5th stage, the ground settlements have been increased to 70mm. During the

construction works of gabion canal bank protection, the ground settlements have been increased to 110mm since the dewatering is being continued. Finally, before the completion of gabion structure, the sheet piles and the “H” iron prop have been removed increasing the ground settlements to 130mm which is beyond the allowable ground settlement limit.

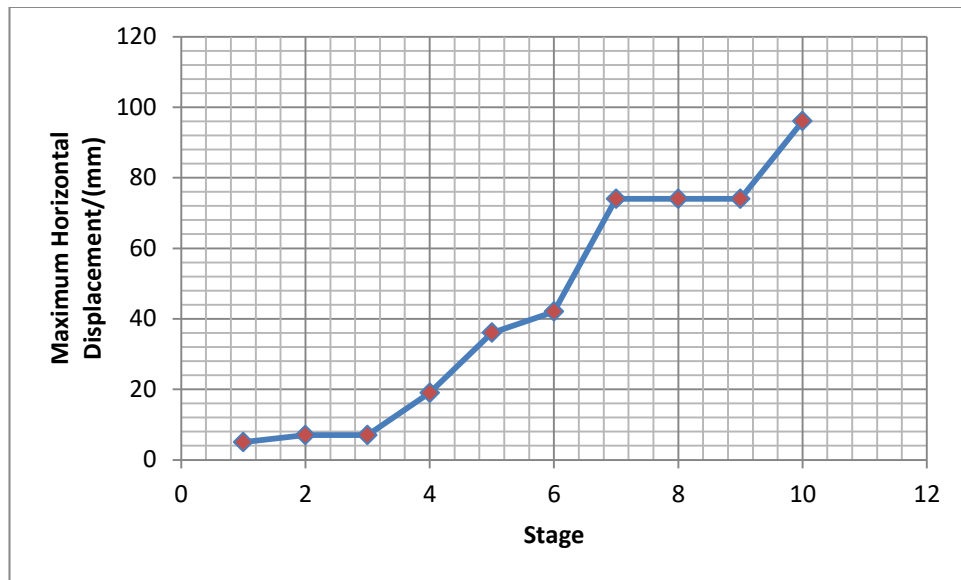


Figure 4.11: Maximum Horizontal displacement at damaged building location for each stage for System 01.

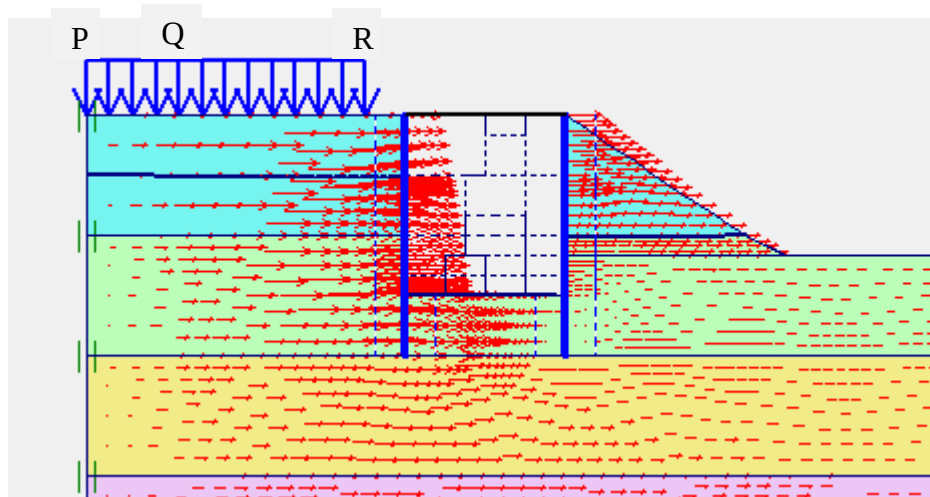


Figure 4.12: Horizontal displacement Profile of stage 05 of System 01.

The horizontal displacement along the construction sequence shown in Figure 4.11 is also in consistent with the vertical settlement variation. The gradient of the horizontal displacement is mild up to stage 03 and thereafter an exponential increase in

displacement is observed in between 3rd and 7th stage which corresponds to excavation. The displacement after 7th stage reach 70mm exceeding the tolerable limit. The displacement increment is minimum during backfilling and shows a significant increase once again at the end during the sheet pile removal with a total of 90mm which is way beyond the allowable limit. The settlement pattern of the horizontal displacement is closely related to the vertical settlements along the sequence.

4.5.2 System 02

The Plaxis model for system – 02 includes 14 stages of construction. Steel props were installed at 0.0m depth, 1.5m depth and 3.0m depths for the 6m long sheet piles.

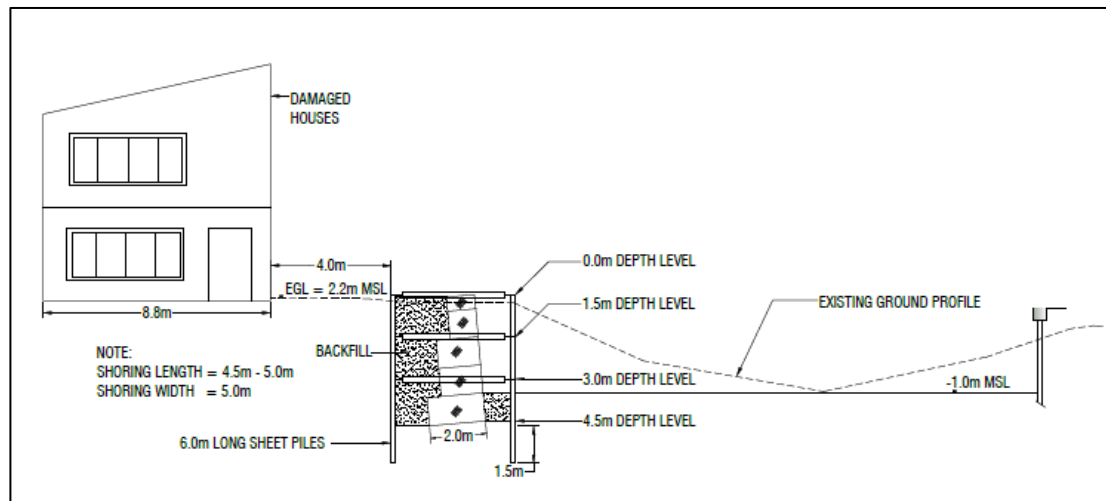


Figure 4.13: Schematic Diagram of System 02 used for the Plaxis Model .

The stages of the plaxis model which represents the construction sequence can be described as below.

At the first stage the 6m long sheet piles were driven upto 6m depth and then “H” iron prop has been installed at 0.0m depth level as the second stage. Excavation of 1.5m depth upto 1.5m depth level was introduced and second prop has been installed as 3rd stage. Another 1.5m depth has been excavated and dewatered as the 4th stage up to 3.0m depth level and 3rd prop has been installed at same 3.0m depth level.

Then another 1.5m depth has been excavated and dewatered up to 4.5m depth level which reaches the design bottom level of gabion structure. This was done at the 5th stage. Gabion construction was started thereafter. At the 6th stage, 2mx1m gabion

box was laid and backfilled the both sides at 7th stage. Then the prop at 3.0m depth level was removed at the 8th stage while the dewatering was in progress.

After that, 1.5mx1m gabion box was laid on top of 2mx1m gabion box and another 1.5mx1.0m gabion box was laid up to 1.5m depth level as the 9th stage. Again backfilling was done at land side upto 1.5m depth level at the 10th stage. Then prop at 1.5m depth level was removed at 11th stage. Thereafter, 1.0mx1.0m gabion box was laid on top of 1.5mx1.0m gabion box and 0.5mx1.0m box was laid on top of it at 12th stage and continued backfilling up to 0.0m depth level at 13th stage. After, the prop at 0.0m depth level was removed and finally the sheet piles were removed at 14th stage.

Table 4.8: Construction sequence used for system - 2

Stage Identification	Description
Stage 01	Installation of Sheet Piles
Stage 02	Install Prop at 0.0m depth level
Stage 03	Excavation of 1.5m depth – upto1.5m depth level Install Prop at 1.5m depth level
Stage 04	Excavation of 1.5m depth – upto3.0m depth level
Stage 05	Install Prop at 3.0m depth level Excavation of 1.5m depth – upto4.5m depth level
Stage 06	Laying 1 st row of gabion (2mx1m)
Stage 07	Backfilling around the gabion
Stage 08	Remove Prop at 3.0m depth level
Stage 09	Laying 2 nd and 3 rd rows of gabion (1.5mx1m)
Stage 10	Backfilling at land side up to 1.5m depth level
Stage 11	Remove Prop at 1.5m depth level
Stage 12	Laying 3 rd and 4 th rows of gabion
Stage 13	Backfilling at land side up to 0.0m depth level
Stage 14	Remove Prop at 0.0m depth level and remove sheet piles

The ground settlement profile for each stage of construction from the edge of sheet pile (at R) towards 7m distance in land side (at P) is obtained from the Plaxis model and plotted as shown in Figure 4.15. The vertical line at 3m indicates the boundary of the damaged house (at Q).

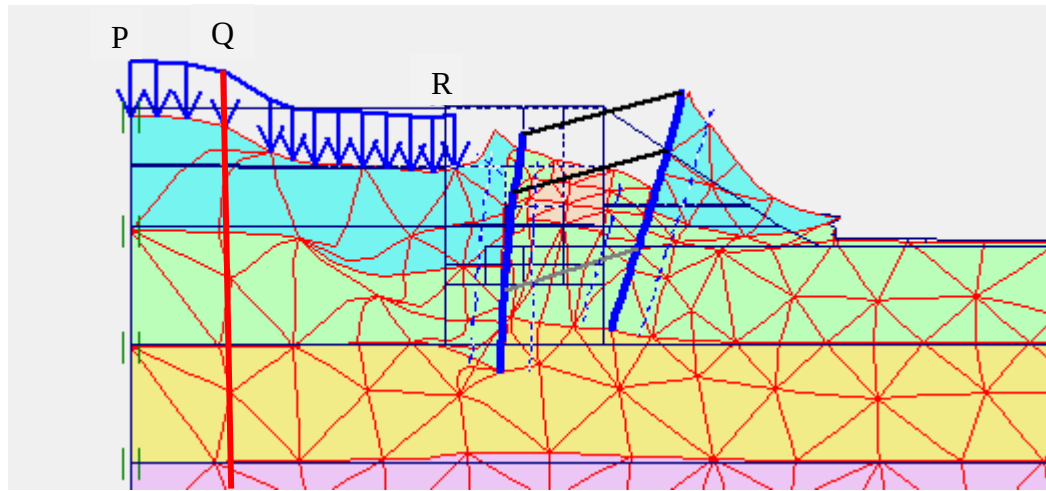


Figure 4.14: Deformed mesh of stage 05 of System 02.

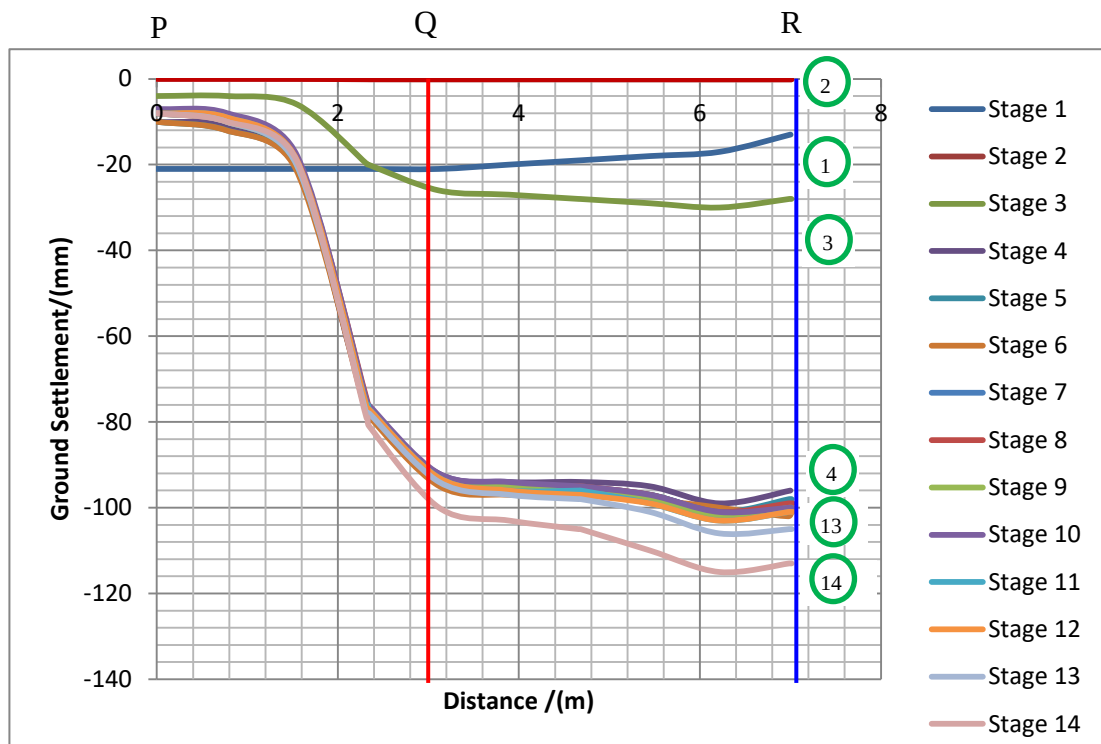


Figure 4.15: Maximum ground settlements along the Distance for System 02

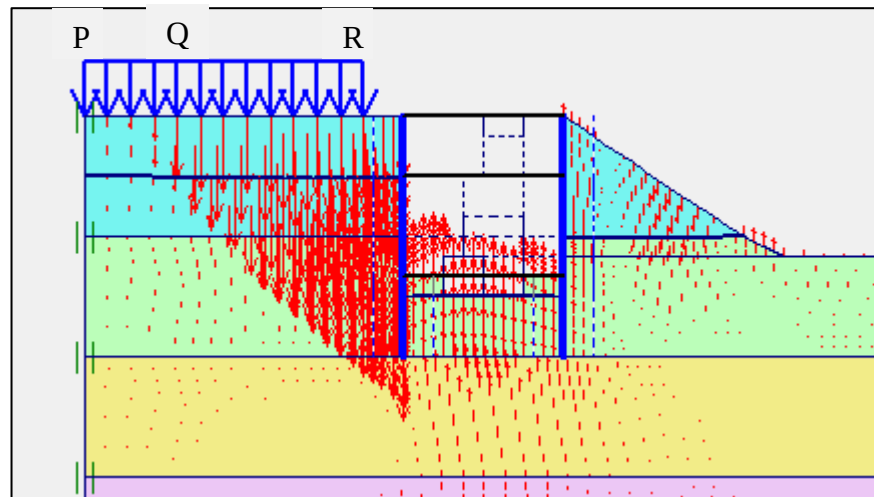


Figure 4.16: Vertical Settlement Profile of stage 05 of System 02.

Accordingly, when the sheet piles were installed at stage 01, the maximum ground settlement of 20mm was observed at 4m distance from the edge of shoring which represents the consolidation settlement due to the surcharge activated.

Installation of “H” iron prop at stage 02, any ground settlements were not observed. But, commencement of excavation of 1.5m depth and installation of second prop at stage 03, ground settlement has been increased to 25mm. Another 1.5m excavation at stage 04 resulted to increase the ground settlement to 90mm.

It is observed that there is a rapid rate of increasing in vertical settlement during stages 03 and 04. This trend of rapid increasing of the settlements has been ceased after stage 04. Apparently, the installation props have resulted in mitigating the settlement increase an even though the subsequent stage associated with excavation of further 1.5m (i.e. excavation to 4.5m depth level), there is no further increasing trend of the settlements and thus the effect of propping is highlighted.

However, the allowable settlement for the safety of the surrounding structures has been set up as 40mm and it is evident that implementation of system 02 exceeds this threshold limit and therefore, implementation of system 02 is not recommended.

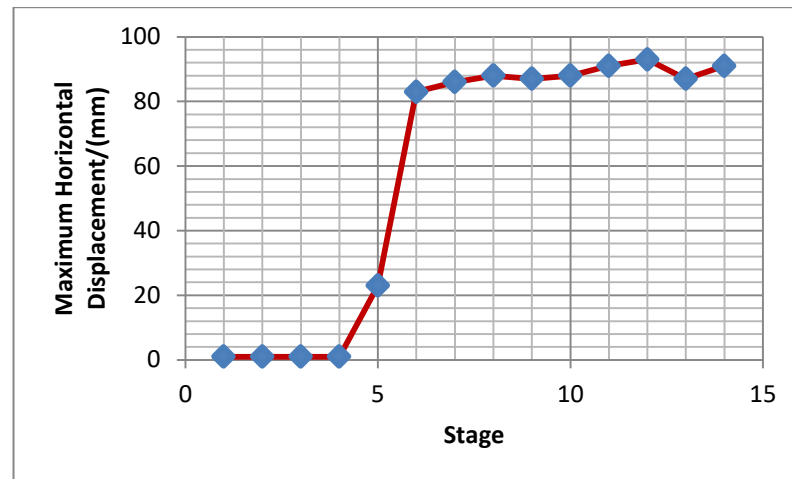


Figure 4.17: Maximum Horizontal displacement at damaged structure location for each stage of System 02.

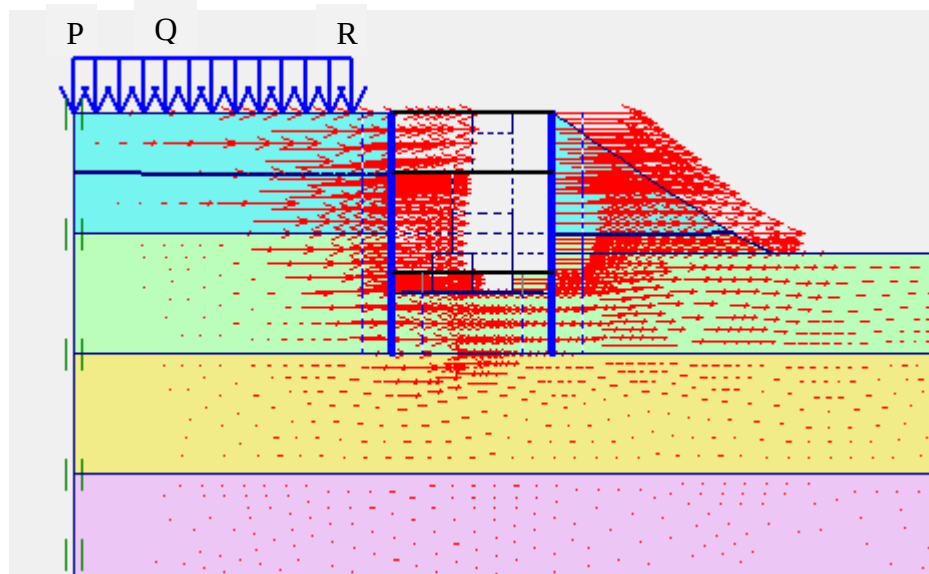


Figure 4.18: Horizontal displacement Profile of stage 05 of System 02.

The corresponding variation of horizontal displacement along the construction stages is shown in Figure 4.17 and the variation is closely in relation with the vertical settlement. The excavation activities at stages 04 and 05 show a significant increase in horizontal displacement with a rapid gradient similar to the vertical settlements. The amount of displacement after the excavation is 85mm, which then shows a minimum increment of displacement with a total displacement of 90mm after the entire construction.

It is observed that the amount of settlements is substantially reduced than System 01 due to the increased bracings and yet both vertical settlement and horizontal displacements are well above the allowable limits.

4.5.3 System 03.

The stage identification for the PLAXIS model for system 03 is as follows. The Plaxis 2D model for system 03 includes 8 construction stages. 9m long sheet piles have been used where it propped at sheetpile top level.

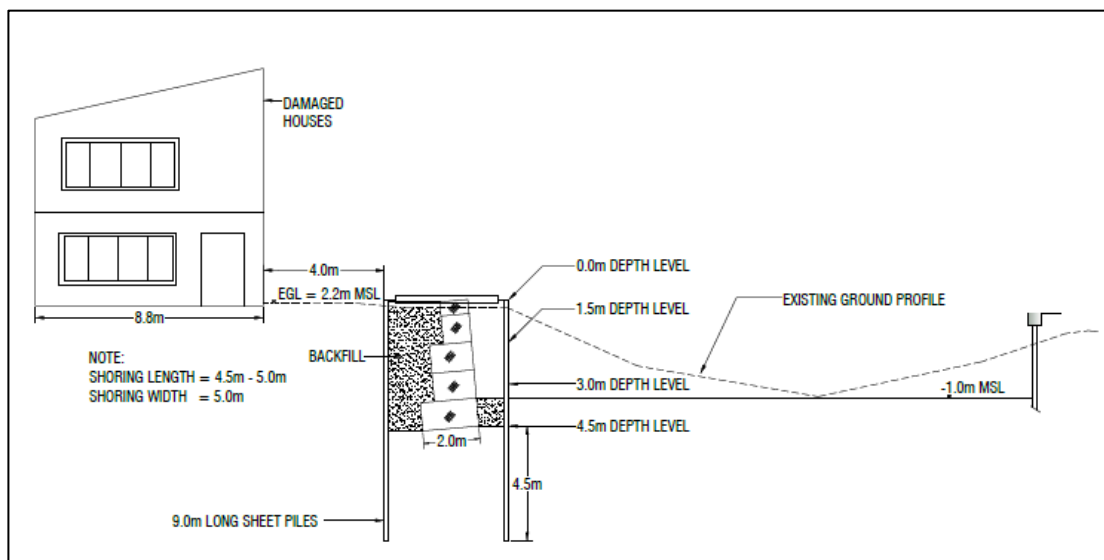


Figure 4.19: Schematic Diagram of System 03 used for the Plaxis Model .

The stages of the plaxis model which represents the construction sequence can be described as below.

At the first stage, 9m long sheet piles were installed. Then 0.5m depth excavation was done and “H” iron prop has been installed at 0.0m depth level as the 2nd stage. Thereafter, excavation was done upto gabion bottom level (i.e. 4.5m depth level) while dewatering is in progress at the 3th stage. Then, gabion construction was commenced. At the 4th stage, 2mx1m gabion was laid and backfilling around the gabion was done. Two boxes of 1.5mx1m gabion boxes were laid on top of 2mx1m gabion box and backfilled at the 5th stage.

Then 1mx1m gabion box was laid on top of 1.5mx1.0m gabion box and backfilled at 6th stage. Afterthat, “H” iron prop has been ejected and 0.5mx1m gabion box was laid on top of 1mx1m gabion box at 7th stage. Finally sheet piles were removed at the 8th stage.

Table 4.9: Construction sequence used for system - 3

Stage Identification	Description
Stage 01	Installation of Sheet Piles
Stage 02	Excavation of 0.5m and install Prop at 0.0m depth level
Stage 03	Excavation of 4.0m depth – upto 4.5m depth level
Stage 04	Laying 1 st row of gabion (2mx1m) and backfilling around the gabion
Stage 05	Laying 2 nd and 3 rd rows of gabion (1.5mx1m) and backfilling
Stage 06	Remove prop
Stage 07	Laying 4 th row of gabion (1.0mx1.0m) and 5 th row of gabion (0.5mx1.0m)
Stage 08	Backfilling around the gabion
Stage 09	Remove sheet piles

The ground settlement profile for each stage of construction from the edge of sheet pile towards 7m distance in land side is obtained from the Plaxis model and plotted as shown in Figure 4.21. The vertical line at 3m indicates the boundary of the damaged house.

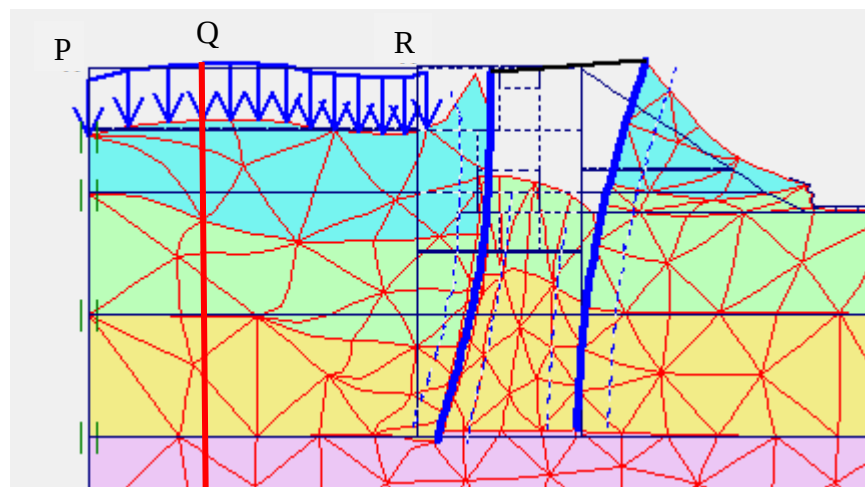


Figure 4.20: Deformed mesh of stage 03 of System 03.

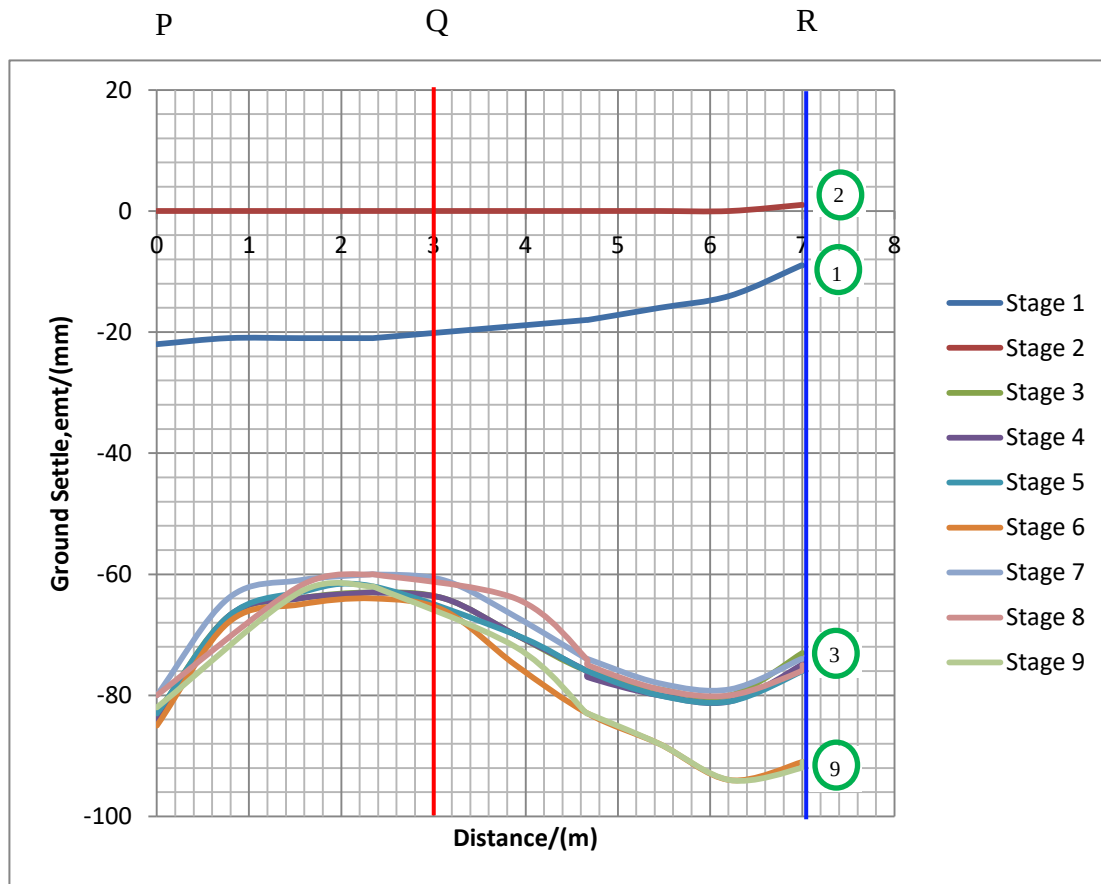


Figure 4.21: Maximum ground settlements along the Distance for System 03

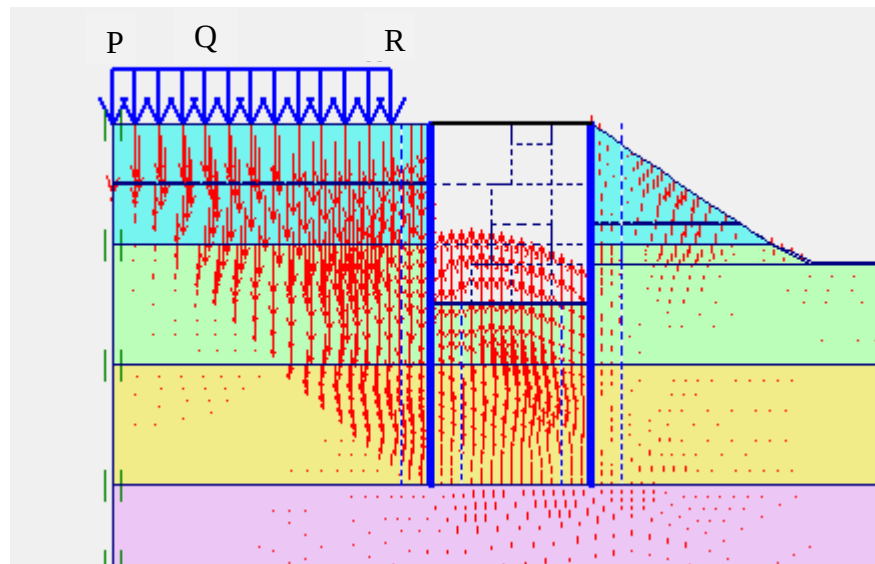


Figure 4.22: Vertical Settlement Profile of stage 03 of System 03.

The deformed mesh and vertical settlement profile for stage 03 was shown in Figure 4.20 and Figure 4.22.

The system 03 involved with the setting up of the props at ground level in stage 02. Therefore, even though there is a considerable rate of settlement during stage 04 (i.e. stage relevant to excavation), there is a reduction of total settlement by approximately 40mm than system 02. The rate of settlement is ceased once the excavation is completed and construction of gabion is initiated.

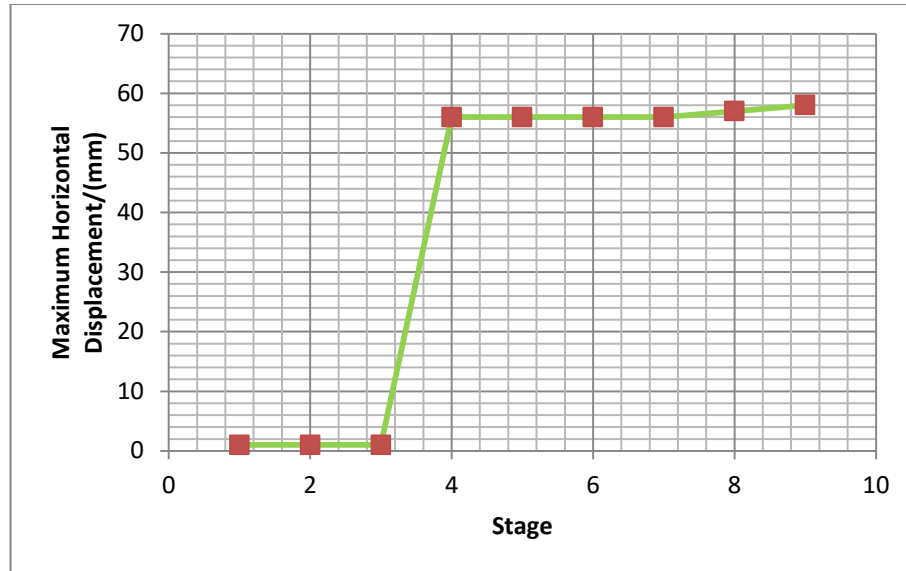


Figure 4.23: Maximum Horizontal displacement at damaged structure location for each stage of System 03.

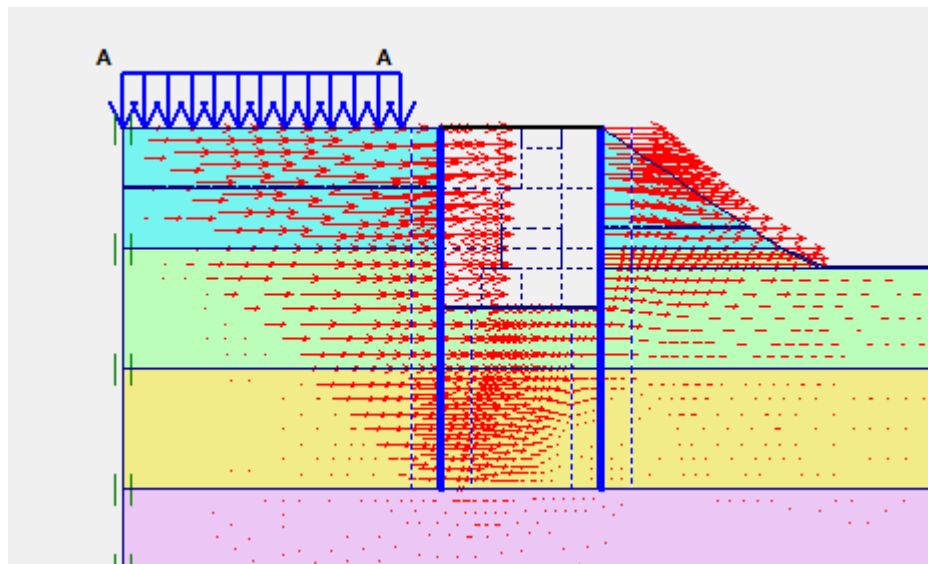


Figure 4.24: Horizontal Displacement Profile of stage 03 of System 03.

The horizontal settlement shown in Figure 4.23 indicates a similar pattern as per previous stages where the horizontal displacement is abruptly increased during the excavation followed by a subdued increase in displacement afterwards. The displacement increases rapidly during the excavation, up to 58mm and the increased resistance of 9m sheet pile was able to hold the displacement graph near constant after the excavation. The amount of displacement is reduced even more than System 02 and yet it exceeds the allowable limit similar to the status of vertical settlements.

In essence, even though the settlements are slightly lower than the System 02, the settlements during the implementation are still considerably higher than the allowable limits and therefore, system 03 also not recommended concerning the safety of the buildings.

4.5.4 System 04.

The Stage identification for the PLAXIS model for System 04 is as follows. The Plaxis 2D model for System 04 includes 14 stages which have 9m long sheet piles propped at 1.5m intervals.

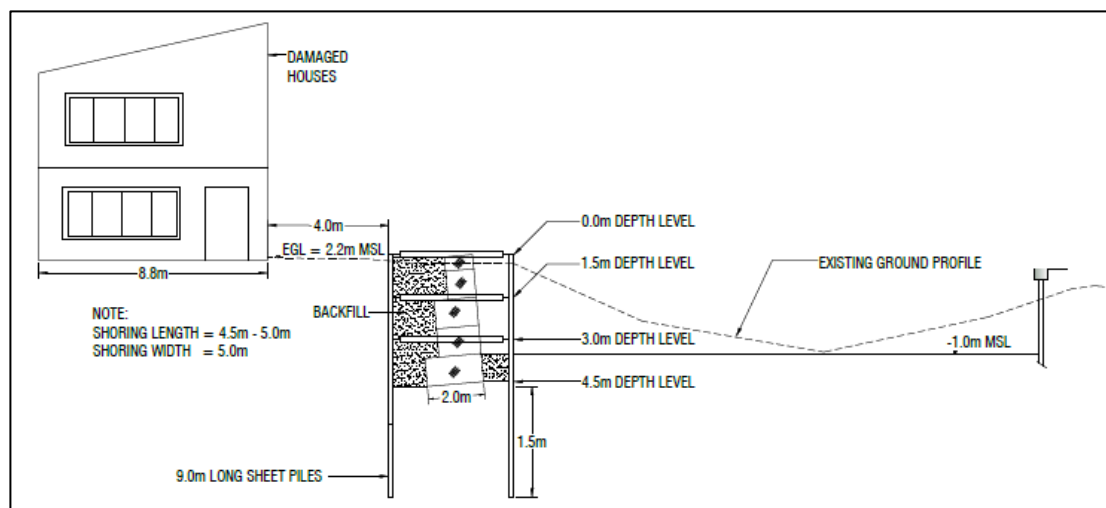


Figure 4.25: Schematic Diagram of System 04 used for the Plaxis Model .

The stages of the plaxis model which represents the construction sequence can be described as below.

Table 4.10: Construction sequence used for system - 4

Stage Identification	Description
Stage 01	Installation of Sheet Piles install Prop at 0.0m depth level
Stage 02	Excavation of 1.5m and install second prop at 1.5m depth level
Stage 03	Excavation of 1.5m depth upto 3.0m depth level and install third prop at 3.0m depth level
Stage 04	Excavation of 1.5m depth upto 4.5m depth level
Stage 05	Laying 1 st row of gabion (2mx1m) and backfilling around the gabion Laying 2 nd and 3 rd rows of gabion (1.5mx1m)
Stage 06	Remove prop at 3.0m depth level.
Stage 07	Backfilling around the gabion upto 1.5m depth level
Stage 08	Laying 4 th row of gabion (1mx1m) and laying of 5 th row of gabion (0.5mx1.0m) Remove prop at 1.5m depth level
Stage 09	Remove prop at 0.0m depth level and remove sheet piles

At the first stage the 9m long sheet piles were driven upto 9m depth and then “H” iron prop has been installed at 0.0m depth level. Excavation of 1.5m depth upto 1.5m depth level was introduced and second prop has been installed as 2nd stage.

Another 1.5m depth has been excavated and dewatered as the 3rd stage up to 3.0m depth level and 3rd prop has been installed at same 3.0m depth level. Then another 1.5m depth has been excavated and dewatered up to 4.5m depth level which reaches the design bottom level of gabion structure. This was done at the 4th stage.

Gabion construction was started thereafter. At the 5th stage, 2mx1m gabion box was laid and backfilled the both sides. Then 1.5mx1m gabion box was laid on top of 2mx1m gabion box and another 1.5mx1.0m gabion box was laid up to 1.5m depth level as the 6th stage while the dewatering was in progress.

Afterthat, the prop at 3.0m depth level was removed and backfilling was done at land side upto 1.5m depth level at the 7th stage. Then 1.0mx1.0m gabion box was laid on top of 1.5mx1.0m gabion box and 0.5mx1.0m box was laid on top of it. Thereafter, prop at 1.5m depth level was removed at 8th stage and continued backfilling up to

0.0m depth level. After, the prop at 0.0m depth level was removed and finally the sheet piles were removed at 9th stage.

The ground settlement profile for each stage of construction from the edge of sheet pile towards 7m distance in land side is obtained from the Plaxis model and plotted as shown in Figure 4.27. The vertical line at 3m (at Q) indicates the boundary of the damaged house.

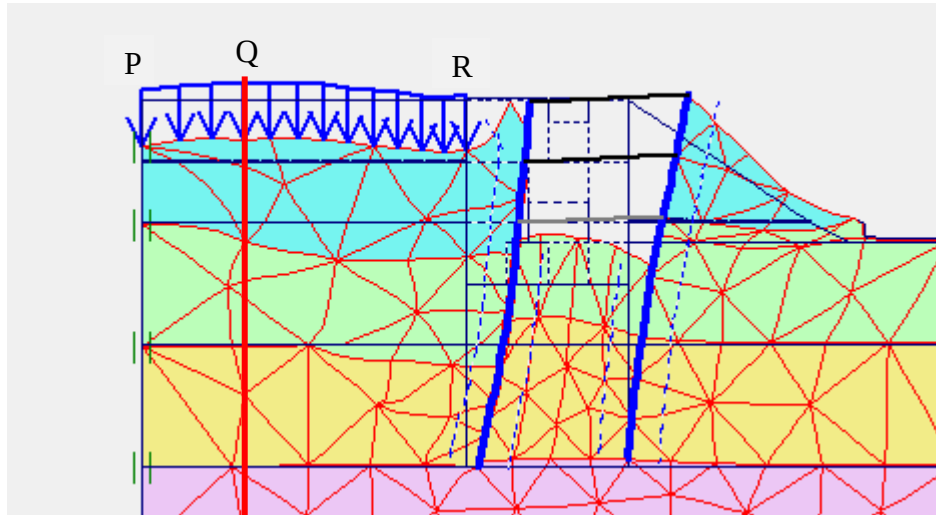


Figure 4.26: Deformed mesh of stage 04 of System 04.

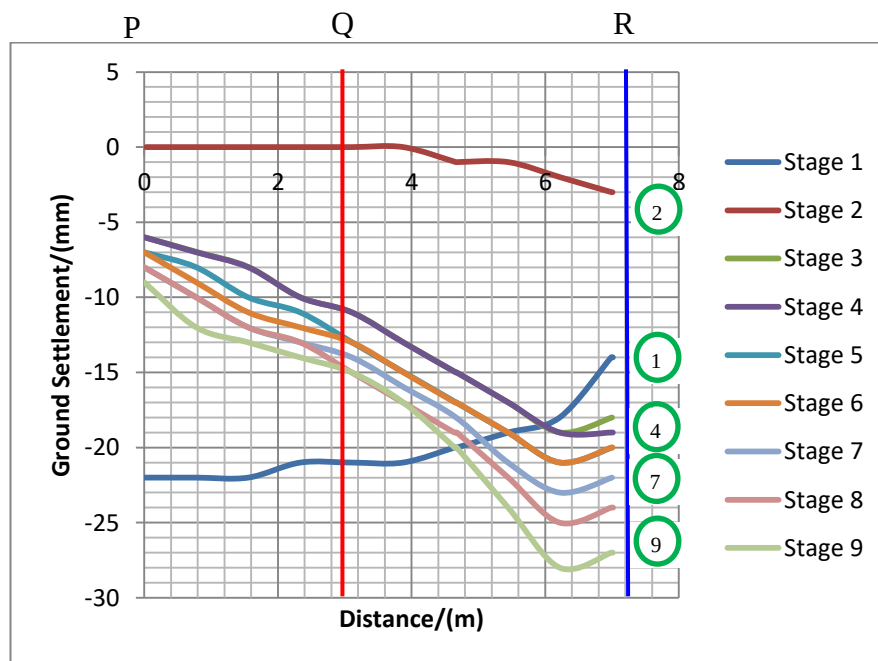


Figure 4.27: Maximum ground settlements along the Distance for System 04.

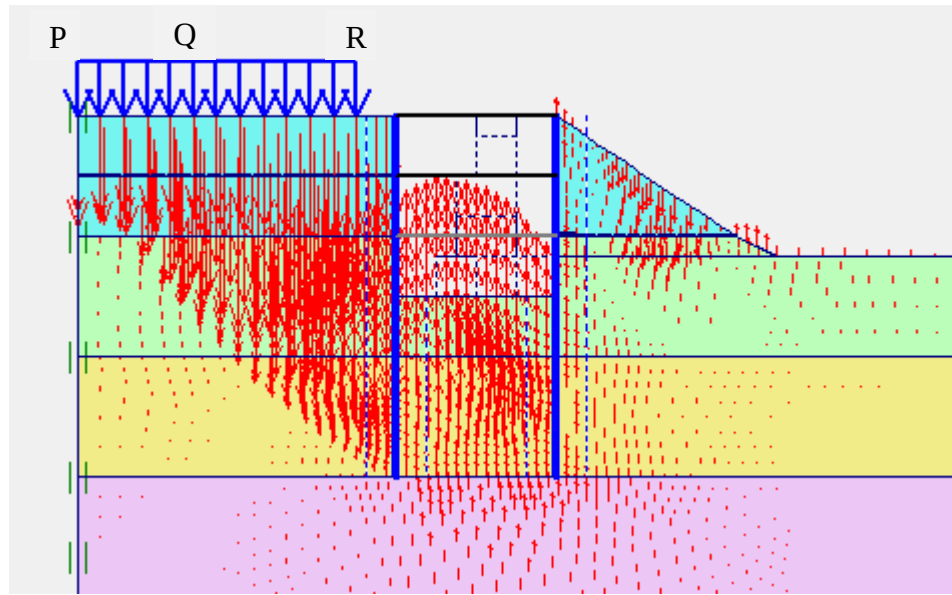


Figure 4.28: Vertical Settlement Profile of stage 04 of System 04.

The system 04 involves propping at 03 levels. Stages 01, 02 and 03 represent the installation of props at 1.5m intervals. The stage 04 is associated excavation upto 41.5m depth and there is a slight increase in settlements in this stage which is understandable. A slight increase in settlements are observed during the next stages. When the overall settlement is considered, the total settlement is within the allowable limits of (i.e. 40mm) along the entire construction sequence.

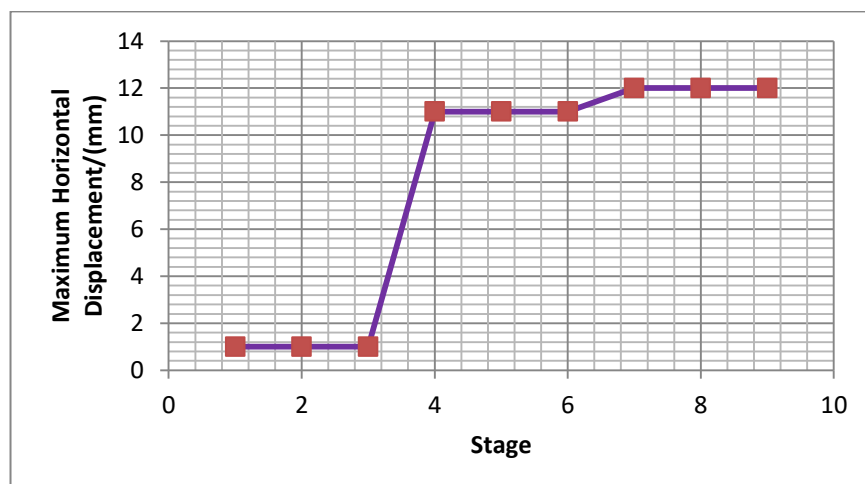


Figure 4.29: Maximum Horizontal displacement at damaged structure location for each stage of System 04.

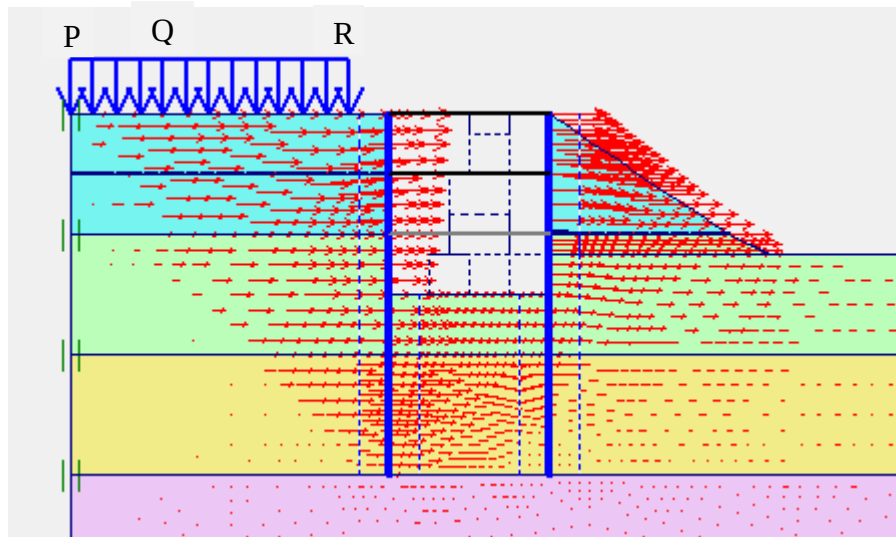


Figure 4.30: Horizontal Displacement Profile of stage 04 of System 04.

The variation in horizontal displacement is shown in Figure 4.29. Similar to all previous systems, the horizontal displacement increases with a higher increment during the excavation stages and yet the amount of displacement after excavation is only 11mm due to the 9m sheet piles with increased bracings in system 04. The displacement after total construction is 12mm and has significantly reduced than all the previous systems. Similar to ground settlements, it can be observed that horizontal displacements also below the allowable limits throughout the entire construction sequence and hence the structural safety is further reassured considering the horizontal displacements with the arrangement of system 04.

Therefore, the system 03 is thus identified as the most recommended construction sequence to be implemented for the safety of the structures.

It is evident that system 01 is very unstable and the ground settlements were always excessive than the allowable settlement of 40mm. The settlements increase exponentially during the mid construction stages simulating the actual damage occurred during the construction.

System 02 is more stable than system 01 where the ground settlements were approximately constant after stage 04. However the maximum stage settlements were still higher than the allowable limits and therefore, there is still a risk of structural damage.

System 03 is even more stable than system 02 where the maximum stage settlements were lesser. Similar to system 02, the ground settlements reached a peak values during the mid construction and then the variation was minimal. However, since the maximum stage settlements are higher than the allowable limits the risk of structural damage is not fully addressed.

System 04 can be indentified as the most stable and most conservative system where maximum stage settlements are always below the limits. A very mild increase in settlement can be indentified along the construction and yet there is no risk of structural damage as the settlement curve is always below the threshold.

Therefore system 04 was identified as the arrangement as the most suitable temporary shoring sytem for the location as per the analysis on construction sequance.

CHAPTER 05: CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The excavation induced ground settlements during construction at urban areas were becoming a major issue for multiple canal improvement projects in Colombo, Sri Lanka and it was revealed during the literature review that it is a common engineering problem in Sri Lanka. Since almost all the infrastructure projects in urban cities require deep excavations during its construction projects, excavation induced ground settlements is a major risk of property damage for most cities. The geotechnical root causes of each cases differ and however the hypotheses of this study were set up that these damages could have been prevented by a proper analysis on construction sequence being conducted prior to the implementation.

Consequently, this case study related to property damages during the bank protection at St Sebastian North canal was analyzed with the objective whether the damage could have been prevented if adequate attention has been paid to the Construction sequence. The finite element analysis for the case was conducted for 04 construction sequences with each sequence consists of multiple stages.

The results of ground settlements and horizontal displacements are shown in Figure 4.9 to Figure 4.29. Accordingly, it is observed that implementation of System 01, System 02 and System 03 induce excessive ground settlements and horizontal displacements during intermediate construction stages and that is an implication that implementation of these 3 systems would cause severe ground settlements that would yield the structural damages to nearby properties. This simulate the actual cause of the damage happened on the ground.

Both system 01, system 02 and system 03 failed for the stage of excavation up to 4.5 m depth and dewatering. This might correspond to the actual structural damages happened. However, in system 04, ground settlement and horizontal displacement values are within the allowable limit along all the construction stages which can be regarded as allowable. Therefore, had these analyses been conducted earlier, the obvious choice would be to go for system 04 and however, that was not picked up as there were no analysis on construction stages. The conclusions can be lined up as follows.

- The system 01, System 02 and system 03 showed excessive settlements at intermediate stage of construction up to 4.5 m depth and dewatering.
- System 04 showed that all settlements and horizontal displacements are within allowable limits along all stages.
- Based on the analysis on construction sequence, the system 04 is the design choice for the implementation.
- The root cause of the damaged cause during the construction at St Sebastian North canal can be identified as dewatering induced surface settlements when dewatering after excavated to 4.5 m depth.
- The temporary shoring arrangement of system 04 is the suitable system to maintain the surface settlements within allowable range. The study illustrated that the driven depth of the system alone will not be adequate. The lateral support system plays a very critical part.
- Based on the analysis on construction stage, system 04 would be the design choice therefore it is concluded that inadequate analysis on construction sequence has resulted the structural damage.

5.2 Recommendations.

Based on the study outcomes the following recommendations are made for the future design in order to prevent the settlement induced damages during the constructions. The following recommendations are made for the projects involved with deep excavations in urban areas.

- The stability analysis of the final stage is not alone adequate to ensure the safety of the constructions.
- It is recommended to always carry out an analysis on the different construction sequences and evaluate the anticipated ground settlements.
- Based on the analysis, the most conservative and cost-effective construction sequence shall be adopted as the construction methodology.
- If there are any unavoidable ground settlements are detected during the analysis on construction sequence, ground improvement measures shall be taken prior to the excavation.
- The excavation shall only be started subjected to the approval of the construction sequence analysis report.

- Installation of ground monitoring points should be done prior to the construction.
- A comprehensive risk assessment shall be completed with the construction sequence analysis report and high risk areas shall be identified prior to the excavation.

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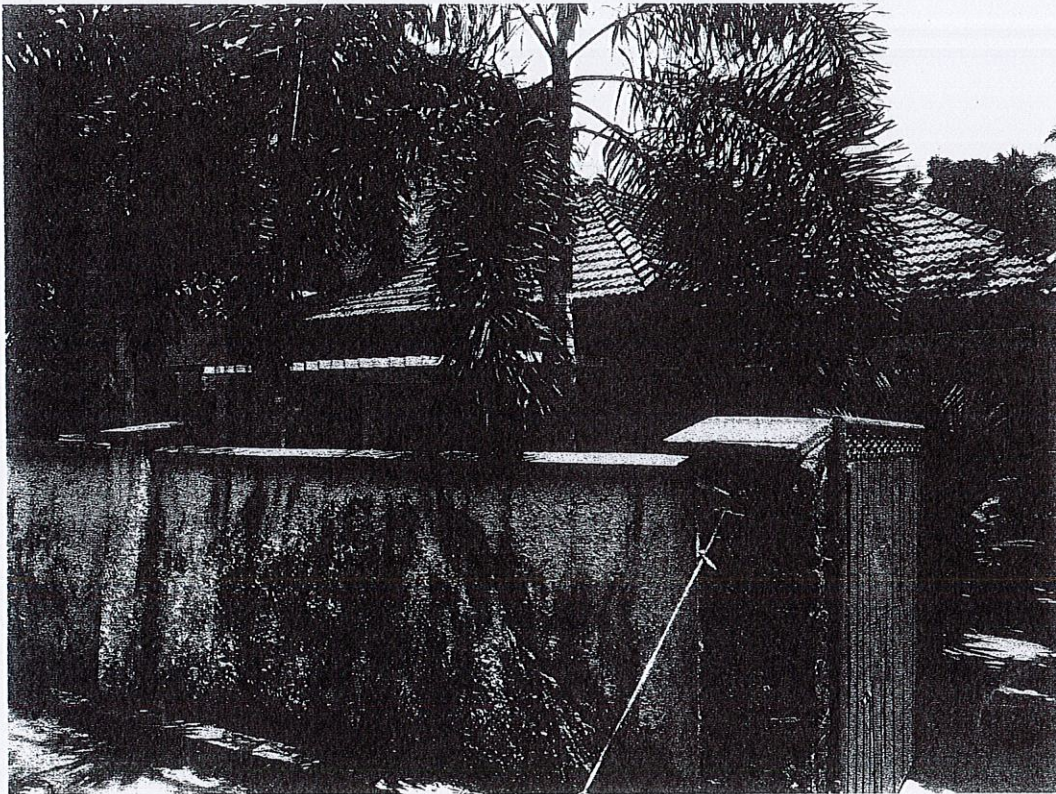
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Annexure I

**DAMAGE ASSESSMENT REPORT/COST ESTIMATE REPORT ON
No. 52, WALLAMPITIYA ROAD, AMBATHALE AT MULLERIYAWA**

Report No. NBRO/PMD/CR-034/2019



**PROJECT MANAGEMENT DIVISION
NATIONAL BUILDING RESEARCH ORGANISATION**
(Ministry of Public Administration and Disaster Management)

99/1, JAWATTA ROAD, COLOMBO 05.

Phone: 011-2588946 Fax: 011-2502611

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June 2019

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DAMAGE ASSESSMENT REPORT/COST ESTIMATE REPORT ON
No. 52, WALLAMPITIYA ROAD, AMBATHALE AT MULLERIYAWA

REPORT No: NBRO/PMD/2019/CR-034

1. INTRODUCTION

1.1 Mr. Chaminda Wijesinghe, Project Manager, E & C Kumathi Joint Venture (Client) informed the Director Project Management Division, National Building Research Organisation (NBRO) by his letter under Reference No. E&C/GEN/MADIW/W/19/58 dated 15/03/2019 that:

- (a) E & C Kumathi Joint Venture have being awarded Improvement of Madiwela Canal Improvement Project from Sri Lanka Land Reclamation & Development Corporation (SLLRDC).
- (b) during the construction, a third party property was damaged.
- (c) Based on a request from E & C Kumathi Joint Venture Sri Lanka Insurance Corporation has suggested to obtain a property damage assessment report/Cost estimate from NBRO

1.2 Accordingly, a joint site inspection of the said premises was undertaken by NBRO on 04/04/2019 in order to ascertain the prevailing site context:

Officers participated:

NBRO:

- Eng. Kavindu Anjana Weerasekara - Engineer/NBRO
- Mr. E.N. Marasinghe - MA-Tech/NBRO

Client:

- Mr. Chaminda Wijesinghe - Project Manager
E & C Kumathi Joint Venture

House Owner:

- Mr. Nalaka Pathberiya

1.3 Following documents were submitted to NBRO by the Client:

- Pre-crack survey report carried out in the affected house on 2016.
- Soil Investigation for the Improvement to Madiwela East Diversion Scheme- Stage 2, Ambathale carried out by Expert Soil Engineering Laboratories.
- Repair estimate given by S. J. Constructions on 14/10/2018.
- Detailed Repair Estimate given by the Contractor (BOQ format) with a sketch of the damaged property.

2. GENERAL OBSERVATIONS**2.1 Site location**

Location of the building was identified in the Google image as shown below:



Figure 01 – Satellite view of the damaged house

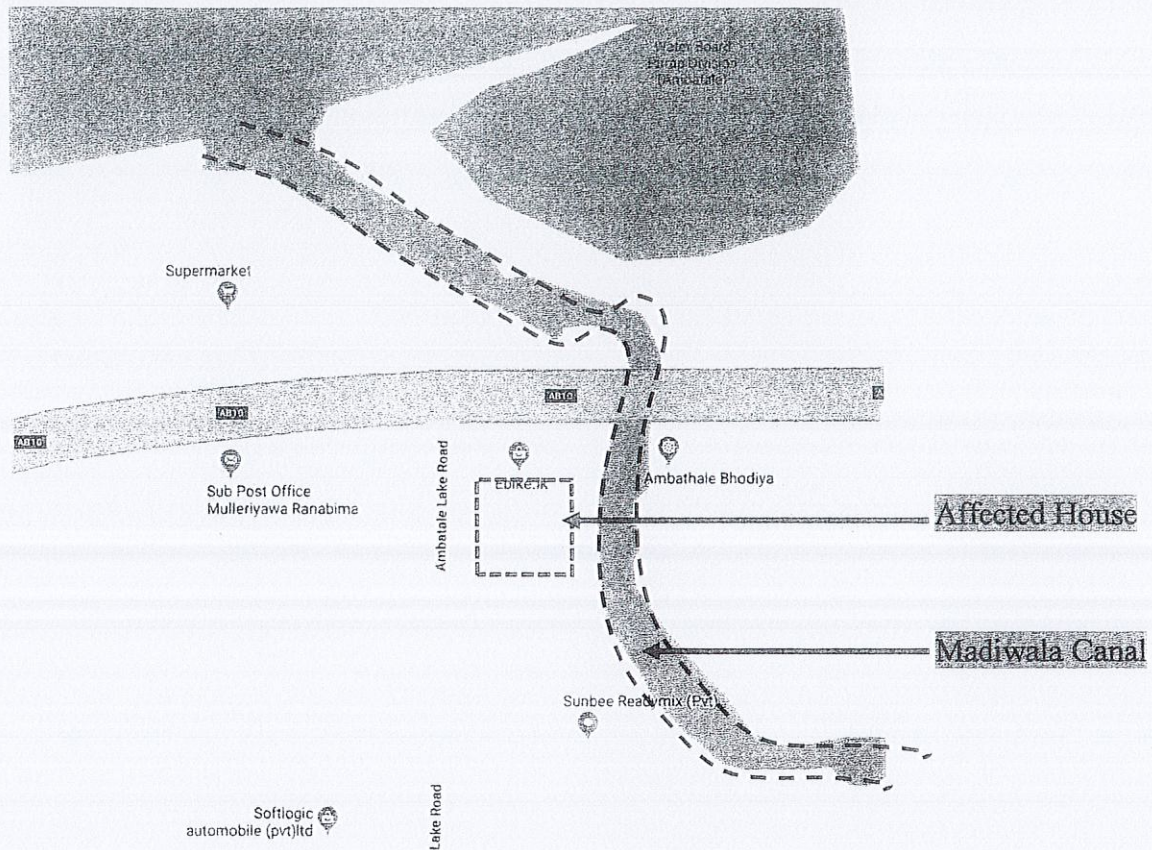


Figure 02 – Location of the damaged house

2.2 Details of the Existing buildings and site

The damaged house was situated closer to the Madiwela Canal. It was a single storied building with load bearing walls and a Calicut tile covered roof. Recently a two storied building was constructed adjacent to the affected house towards the canal side. As per the details given by the client the depth of the canal was 6m from the existing ground level.

A Gabion wall was constructed using interlock sheet piles and the construction of it was finished in November 2018. After the construction, interlock sheet piles were left driven in to the ground to avoid further damage to the properties. As informed by the client and the house owner, the canal area is regularly prone to flooding.



Figure 03 – Front view of the Building and the canal (Source: Google Street View)



Figure 04 – Front view of the Building (Source: Google Street View)

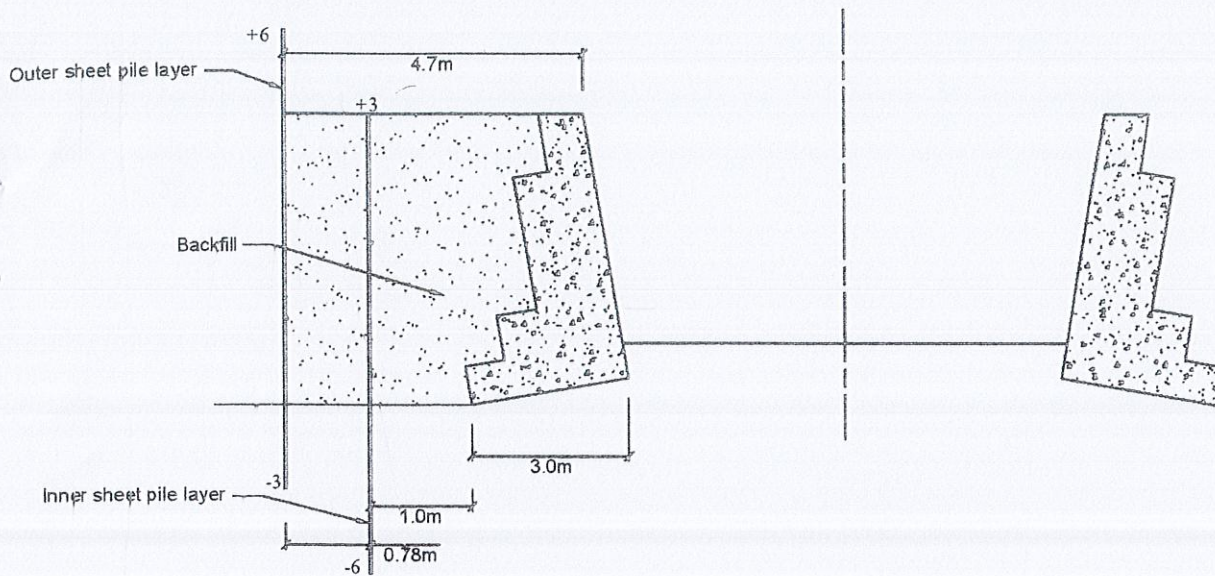


Figure 05 – Cross section of the Gabion Wall

3. DETAILED OBSERVATIONS

3.1 Building

- a) There were cracks of width lesser than 1cm at many locations of the house
 - There were cracks starting from the bottom corners of the window openings

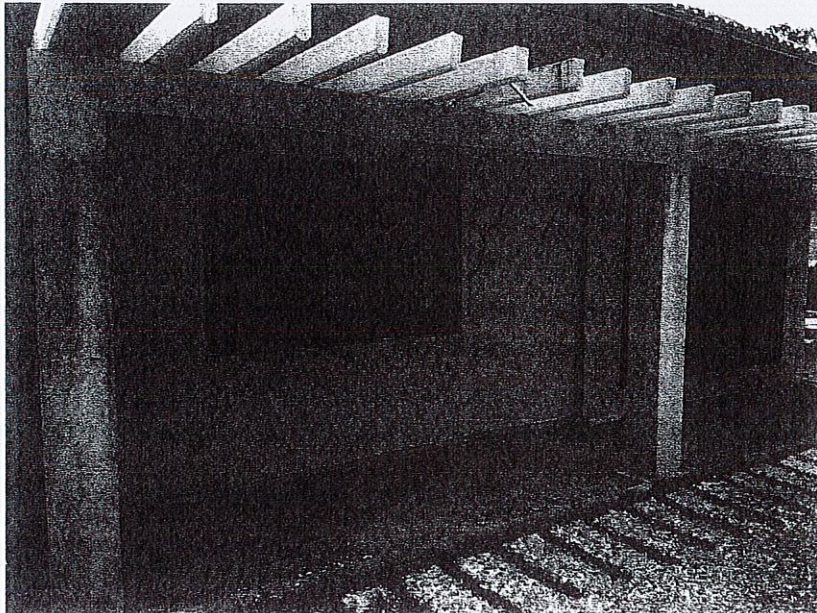


Figure 06 - Cracks at the front side of the building

- There were vertical cracks at wall corners of interior rooms of the house. Most of them were starting from the floor.

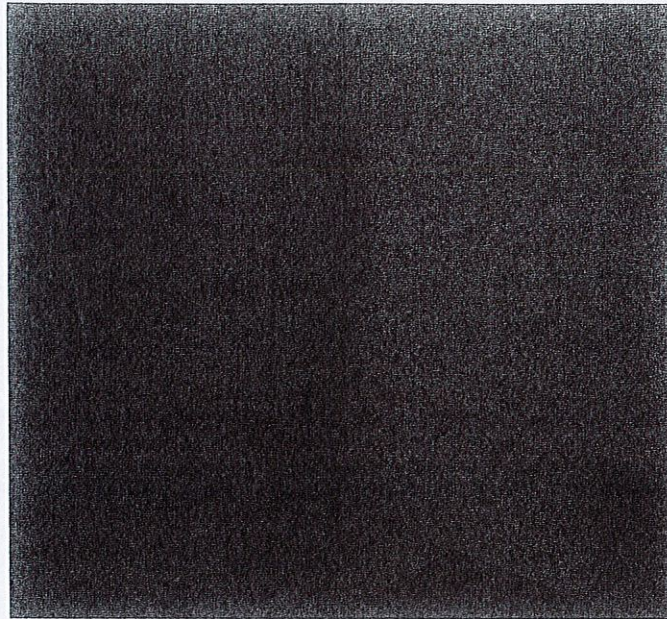


Figure 07 – Vertical cracks at wall corners

- b) A major crack of width more than 1cm was observed at many walls across the house. That crack was observed to be widening at the top of the wall.

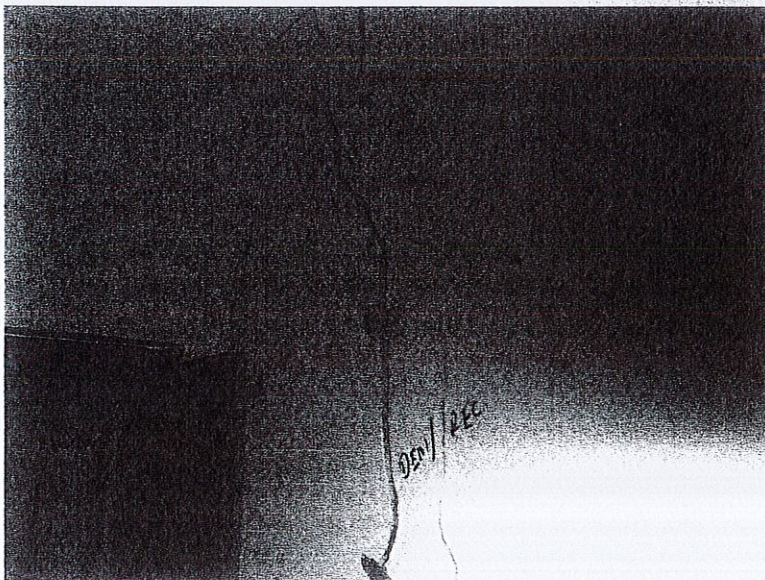


Figure 08 – Vertical cracks of width more than 2mm

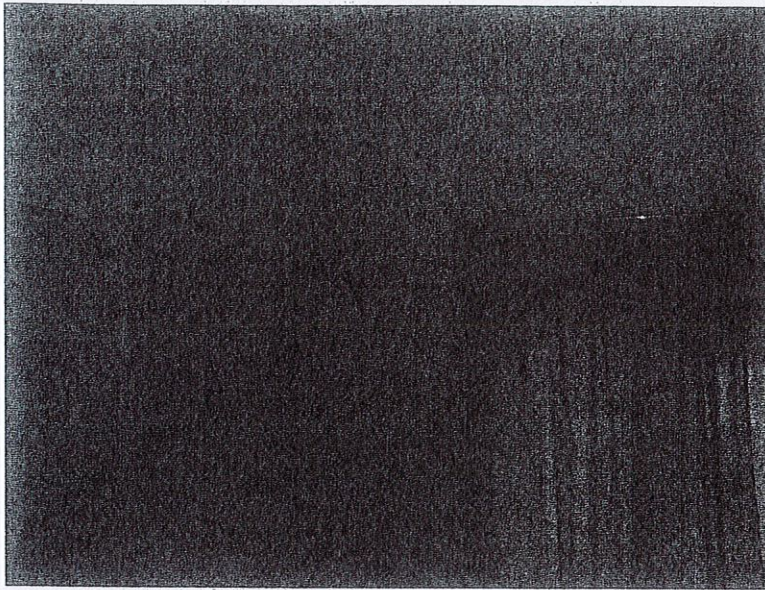


Figure 09 – Vertical cracks which are widening at the top

- c) A ground crack was observed to be running through the house parallel to the boundary between two adjacent buildings.

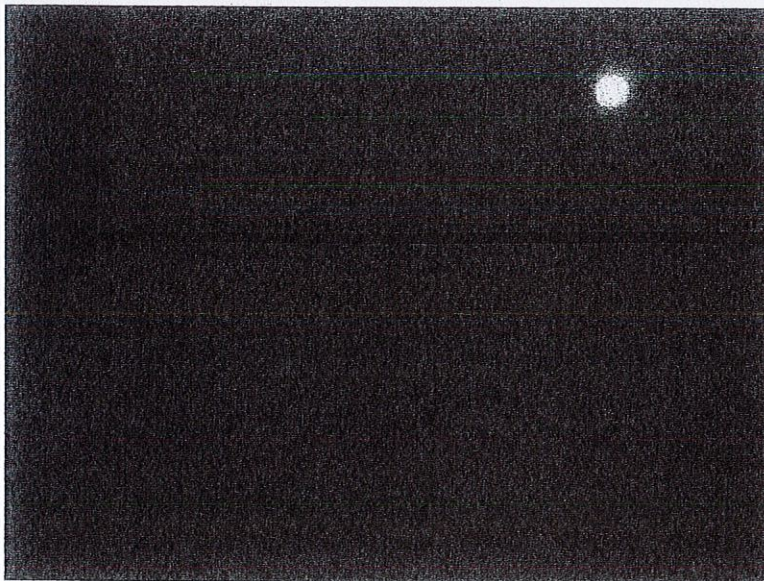


Figure 10 – Tension crack running across the house

- d) When using the water bubble leveler, a small tilt in the floor was observed towards the two storey building side.

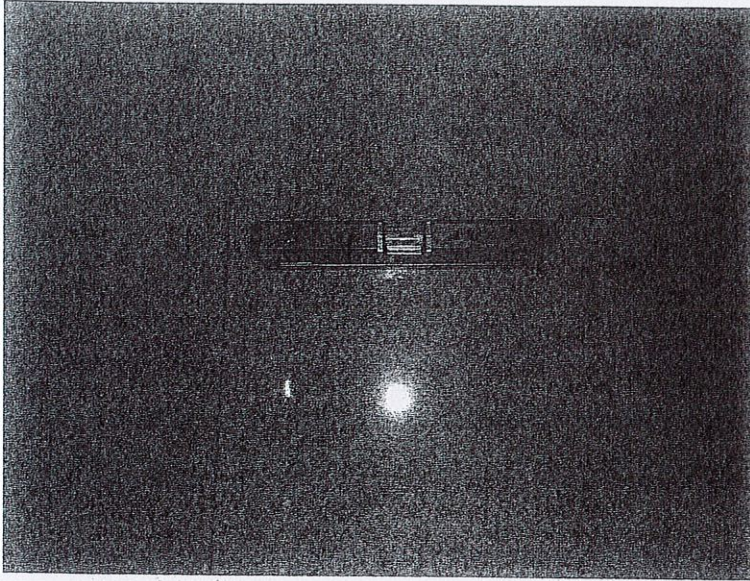


Figure 11 – Tilt observed in the floor of the house

- e) It was observed that the single storied building was connected to the two storied building structurally.

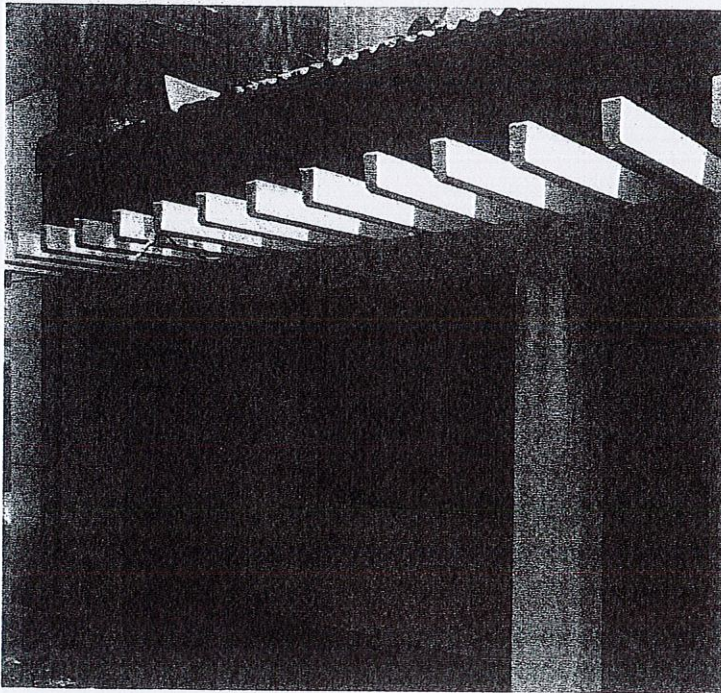


Figure 12 – Two buildings were connected with each other.

3.2 Site

- (a) Distance from the retaining wall to the edge of the building was approximately 15m.
- (b) There was a ground crack along the 'retaining wall- building' boundary. As informed by the client, at the early stages, that gap was filled with sand and covered with cement grout at the top.

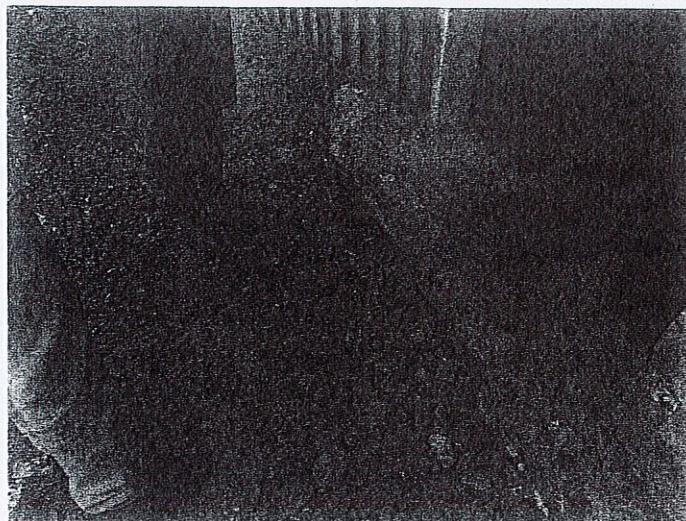


Figure 13 – Vertical cracks at wall corners

- (c) A new tension crack had appeared in the ground between the two storied building and the retaining wall.

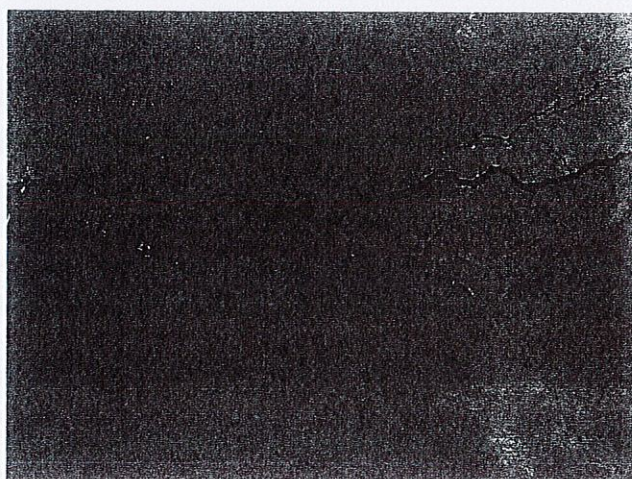


Figure 14 – New tension crack near the two storied building

4. ANALYSIS

- (a) According to the information provided by the contractors, the retaining wall was constructed using two sheet pile layers. Inner sheet pile layer was driven to a depth of 9m from the Ground level. Outer layer was driven to a depth of 12m and sheet piles were driven keeping spaces between each other. After the construction of the retaining wall, space between sheet pile layers and the retaining wall was filled with an Aggregate Base Course.
- (b) It is recommended in the soil investigation report to adopt steel sheet piles which are driven to the layer of 'completely weathered rock': i.e. to a depth of around 17m below borehole elevation. But sheet piles were only driven to a depth of 12m where a very weak residual soil layer is present. Therefore they are not supported at the ends to withstand pressure applied by the soil mass.
- (c) As sheet piles are not driven up to the recommended depths and a controlled fill was not used to fill the gap between the sheet pile wall and the gabion wall, it can be concluded that the sheet pile layer is tilted.
- (d) Along with the sheet pile layers, two storied building is tilted towards the canal side.
- (e) According to the site observations, single storied building is connected to the two storied house. Therefore the settlement two storied building had undergone is affecting the single storied building.

5. CONCLUSION AND RECOMMENDATIONS.

- (a) To avoid further propagation of cracks, two storied building should be separated from the single storied building completely.
- (b) Walls and foundation in the two storied building side should be demolished up to the floor crack propagated across the house.
- (c) Soil under the demolished part should be compacted by filling with a new soil layer with high bearing capacity.
- (d) Foundation should be reconstructed by introducing tie beams as shown in **Annex I** with the strict supervision of a qualified engineer.
- (e) All the minor cracks in the house should be repaired according to the methodologies mentioned in **Annex II** and **Annex III**.

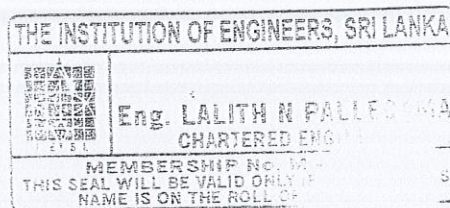
- (f) All of the above recommendations are for the damages observed at the time of site visit. Considering the soil profile in the area and the tilting of the sheet pile layer, further settlements are possible even with a strengthened foundation.
- (g) Cost Estimate for the repairs of the damaged building is attached in Annex IV.

Prepared by:

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Engineer / Project Management Division

Checked by:

Eng. L.N. Pallegama
Structural Consultant/ Project Management Division

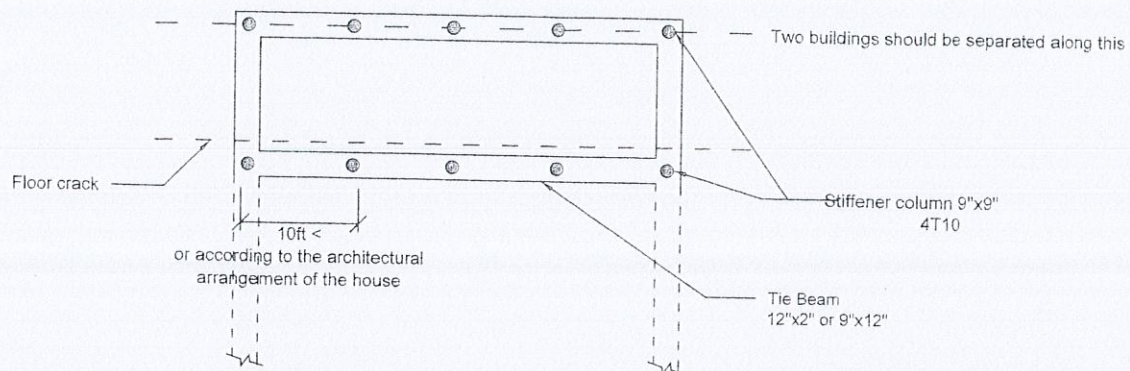


Certified by:

Eng. J.K. Jayawardena
Director
Project Management Division
National Building Research Organisation
Director – Project Management Division

Date: 11.06.2019.

ANNEX I



DRWG. TITLE
DAMAGE ASSESSMENT REPORT ON PROPERTY
AT NO.52, WELLAMPITIYA RD,
MULLERIYAWA
E & C KOMUTHI JV

NATIONAL BUILDING RESEARCH ORGANISATION			
80, PRIVATE ROAD, COLOMBO 08			
TEL: 011 266966, 011 266184			
PROJECT	DATE	17-05-2003	
CO-ORDINATOR	APPROVED	FOR SR	
DESIGNER	FOR SR		
PREPARED	FOR SR	NO. 52, WELLAMPITIYA RD, COLOMBO 08	
CHECKED	FOR SR	SR OF SR	
DATE	17-05-2003		

ANNEX II

Crack which does not pass through (one side cracks) can be repaired as follows.

1. Wire brush 3inch width on both sides of the crack for whole length.
2. Cut a U-groove along the crack as per following dimensions based on its width.
3. For crack width below 2mm – 5mm wide, 5mm deep U groove.
4. Fully clean the groove and the wire brush area and remove dust.
5. Carefully gun the X-Shield StrongSeal or equivalent with a smooth, continuous bead.
6. Remove extra sealant by a scraper.
7. Apply X-shield Acrylcoat or equivalent using brush, roller or airless spray as per the specifications provided by the manufacturer.
8. Finish with applying paint on top of the Acrylcoat layer matching to background.

ANNEX III

Cracks which pass through the wall need to be repaired as follows.

1. Install plastic nozzles and dispense MIT VE Tropical or equivalent to the crack from both sides.
2. After that install 100 mm wide X-Wrap GFRP fabric strip (or equivalent) with X-Wrap Laminating Adhesive (or equivalent).
3. Finish with applying paint on top of the Acrylcoat layer matching to background color.

ANNEX IV

X-Shield GlassFiber Fabric

High strength glass UD fabric

201480116

Product Description

X-Shield GlassFiber Fabric is a glass fiber fabric uni-directional sheet designed for the strengthening of structural members against tensile, shear and impact forces.

Advantages

- High strength
- Easy to use

Uses

As an external reinforcement system for the structural strengthening of:

- Beams
- Columns
- Slabs
- Tunnels
- Circular, curved and complicated details

Laboratory Test Data

Property	Typical Results	
	GFW430	GFW920
Areal Weight	450g/m ²	935g/m ²
Design Thickness	0.17mm	0.36mm
Tensile Strength	2300MPa	2300MPa
Tensile E-modulus	76GPa	76GPa
Elongation at break	3.1%	3.1%

Sizes

GFW430 - 80m Length x 50cm Width

GFW920 - 40m Length x 50cm Width

Packaging

GFW430 - 40m²/roll

GFW920 - 20m²/roll

Storage

X-Shield GlassFiber Fabric should be stored in dry conditions and protected from direct sunlight; the shelf life under such conditions will be unlimited.

Installation Guidelines

X-Calibur provides detailed method statements on all its products for use in various applications. These must be referred to prior to starting work.

Health and Safety

This product is not considered hazardous.

Authorized Technical Specialist

Please note that only X-Calibur Authorized Technical Specialists ('ATs') are permitted to change any of the information in this data sheet or to provide written recommendations concerning the use of this product. Visit www.x-calibur.us for a full list of X-Calibur ATs.

Datasheet Validity

X-Calibur makes modifications to its product datasheets on a continuous basis. Please check the datasheet update section on www.x-calibur.us to ensure you have the latest version.

Warranties

X-Calibur supplies products that comply with the properties shown on the current datasheets. In the unlikely event that products supplied are proved not to comply with these properties, then we will replace the non-compliant product or refund the purchase price. X-Calibur does not warrant or guarantee the installation of the products as it does not have control over the installation or end use of the products. Any suspected defects must be reported to X-Calibur in writing within five working days of being detected. X-Calibur Construction Chemistry Inc. **makes no warranty as to merchantability or fitness for a particular purpose and this warranty is in lieu of all other warranties express or implied.** X-Calibur Construction Chemistry Inc. shall not be liable for damages of any sort including remote or consequential damages, down time, or delay.

X-Wrap Lamination Adhesive

Lamination resin for X-Wrap Fabrics

70200214

Product Description

X-Wrap Lamination Adhesive is an easy to apply, high performance epoxy resin designed as an impregnant for fabric reinforcement; the resulting laminate offers excellent strengthening properties coupled with long term durability.

It has been formulated as a solvent-free, semi-thixotropic system which allows easy mixing and fibre impregnation combined with minimum drainage and sag on vertical surfaces.

It is available in two grades standard and tropical.

Advantages

- Solvent free
- Zero VOC
- Two grades standard and tropical
- High strength
- Strong adhesion
- Can be used in vertical and horizontal applications

Uses

As the lamination resin for X-Wrap Fabrics

Laboratory Test Data

Property	Standard	Tropical
Viscosity 20°C	5000 cps	7000 cps
Pot Life	40 mins at 25C	40 mins at 35C
Tensile Strength 7d/20°C	45MPa	
Flexural Strength 7d/20°C	60MPa	
Compressive Modulus 7d/20°C	1.67GPa	
Glass Transition Temperature	65C	

Theoretical Coverage

0.5 to 0.6 kg/m²

Packaging

5 kg packs

Shelf Life

18 months when stored below 30C (86F) under shade in a dry environment.

Installation Guidelines

X-Wrap Lamination Adhesive can only be applied by an X-Calibur Registered Contractor using trained personnel and the correct equipment. X-Calibur does not allow any contractor on to the register unless they satisfy strict selection criteria. For each application X-Calibur will provide a detailed method statement that the Registered Contractor will work to.

Application

Refer to method statement for the particular application.

Mixing

Mix X-Wrap Lamination Adhesive using the following technique. Add the hardener 'Part B' into the base 'Part A' and mix using a slow speed drill (500 rpm) with an X-Shield Coating Mixer Paddle for 3 minutes or until both components have fully dispersed and are uniform in color. Be sure to rotate the mixer throughout the drum. Mix only full packs.

Health and Safety

This product is for industrial use only by trained operatives. It is potentially hazardous if not used correctly. Please refer to the Material Safety Data Sheet (MSDS) prior to the purchase and use of this product. The MSDS can be obtained via our website www.x-calibur.us

Authorized Technical Specialist

Please note that only X-Calibur Authorized Technical Specialists (ATSS) are permitted to change any of the information in this data sheet or to provide written recommendations concerning the use of this product. Visit www.x-calibur.us for a full list of X-Calibur ATSS.

Datasheet Validity

X-Calibur makes modifications to its product datasheets on a continuous basis. Please check the datasheet update section on www.x-calibur.us to ensure you have the latest version.

Warranties

X-Calibur supplies products that comply with the properties shown on the current datasheets. In the unlikely event that products supplied are proved not to comply with these properties, then we will replace the non-compliant product or refund the purchase price.

X-Calibur does not warrant or guarantee the installation of the products as it does not have control of the installation or end use of the products. Any suspected defects must be reported to X-Calibur in writing within five working days of being detected. X-Calibur

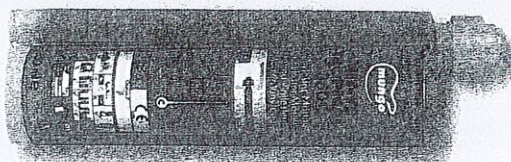
Construction Chemistry Inc. **makes no warranty as to merchantability or fitness for a particular purpose and this warranty is in lieu of all other warranties express or implied.** X-Calibur Construction Chemistry Inc. shall not be liable for damages of any sort including remote or consequential damages, down time, or delay.

MIT-VE Tropical Vinylester mortar, styrene free



Features

- European Technical Approval Option 7 for non-cracked concrete
- LEED - Test Report
- Application also in damp and water-filled drill holes
- Variable setting depth
- Slow curing
- Styrene free and low odour
- Colour of mortar: grey
- Indoor (zinc plated) and outdoor (stainless steel) applications



Applications

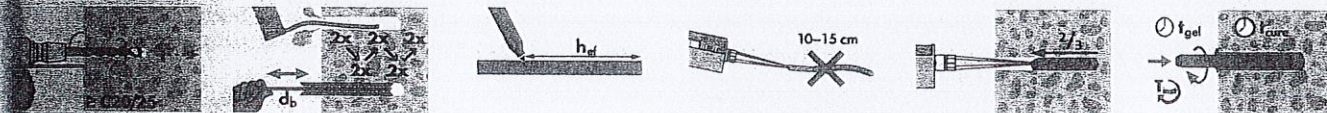
Steel constructions, wooden constructions, façades, façade scaffolds, railings, high-racks, machines, staircases, ladders, cable trays, canopies, hand-rails, consoles



Temperatures

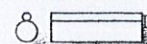
	10°C	10-20°C	20-25°C	25-30°C	30-35°C	35-40°C	40-45°C	>45°C
Working time, in minutes (t_{gel})	30	15	10	7.5	5	3.5	2.5	2.5
Curing time, in minutes (t_{cure})	5h	5h	145	85	50	40	35	12

Installation



Article code	Description	Languages	Content	Box content	Quantity per pallet
1710007	Cartridge incl. 2 mixers	DE/GB/IT	350	12	1440

MIT-VE Tropical Vinylester mortar, styrene free, 350 ml cartridge



X-Tite StrongSeal

Super adhesion high performance MS adhesive and joint sealant

50540310

Product Description

X-Tite StrongSeal is an extremely elastic moisture curing sealant designed for use in metal architecture, curtain wall construction and concrete, floor & wall joints subject to movement. X-Tite StrongSeal's adhesion to difficult surfaces permits its use on anodized metal, and coatings. X-Tite StrongSeal will not stain limestone, marble, concrete or other absorbent substrates. X-Tite StrongSeal's unique polyether chemistry eliminates outgassing on green concrete and "suntanning" or discoloration after exposure to ultraviolet light.

Advantages

- Single component; no mixing required
- Easy to use
- Durable resilient seal
- Suitable for a wide variety of substrates
- Non staining
- Short tack free time
- Can be over painted
- Waterproof and weather proof

Uses

- Expansion joints
- Pre-cast concrete
- Block and masonry
- Curtain walling
- Window and door frames
- Wooden flooring
- Adhesive
- Cove joints
- Transportation
- Weather sealing

Laboratory Test Data

Property	Typical Results
Elongation at Break (ASTM D412)	550 to 600%
Hardness Shore A (ASTM C661)	20 ± 3
Shear Strength (ASTM D1002)	150 Psi
Low Temperature Flex (ASTM D816)	Minus 10°F pass 1/4" mandrel
Shrinkage	No measurable shrinkage after 14 days
Service Temperature	Minus 40°F to 200°F continuous service
Movement Accommodation Factor (BS 6093)	50%

Volatile Organic Content

VOC = <19 g/L

Specification Compliance

ASTM C 920, Type S, Grade NS, Class 25.
ISO 11600 Class F25HM
Federal Specification TT-S-00230-C type II, Class A
Corps of engineers CRD-C-541, Type II, Class A
Meets CARB and SCAQMD VOC requirements.
Conforms to California Proposition 65
Conforms to USDA Requirements for non-food contact

Color

White, Grey and Black.

*Color matching is available in batch quantity only

Packaging

10.1 oz (300 ml) - Cartridges
20 oz (825 ml) - Sausages
5 gallon pails and 50 gallon drums available by special order.

Shelf Life

One year from date of manufacture when stored below 30C for cartridges and sausages. Pails and drums have a shelf life of three months. High temperatures and high relative humidity will significantly reduce shelf life.

Installation Guidelines

X-Calibur provides detailed method statements on all its products for use in various applications. These must be referred to prior to starting work. The information below is a summary intended for guidance only.

Surface Preparation

Concrete substrate must be structurally sound. Loose or unsound concrete should be removed. Surfaces must be entirely free of oil, grease, paint, corrosion deposits, dust, laitance or other surface deposits.

Joint Preparation

Joint surfaces should be clean, dry and free from all contamination including: dirt, oils, grease, tar, wax, rust and any other substance that may inhibit the sealant's performance.

Joint Design

Install all joint applications per ASTM and SWRI recommendations and guidelines. Joints shall be designed with a depth to width ratio of 1:2 (joint depth one-half the width). Control the depth of the sealant by using a polyethylene backer rod that is 25% larger than the joint opening at standard temperature. To prevent three-point adhesion use a backer rod or bond breaker tape to ensure proper joint movement and a long lasting weatherproof seal. Where the joint configuration will not permit a backer rod, X-Calibur recommends that an alternative bond breaker be used.

Concrete

Prior to application remove any residual contamination by mechanical abrasion, sand blasting or power washing. On green concrete, remove all release agents, friable and loose concrete. Dry all visible and standing water prior to applying X-Tite StrongSeal. Install an appropriate backer rod to avoid three-point bonding.

Metal

Prepare all metal to ensure maximum adhesion. Remove all rust, scale and residue by wire brushing to a bright metal sheen. Remove films, loose or inappropriate coatings and oils with an appropriate solvent such as alcohol. *

*X-Calibur recommends that coated substrates be tested for adhesion prior to starting a project. Please contact Technical Services for specific application guidelines and recommendations.

Wood

Wood should be clean, sound and dry prior to sealant application. Allow treated wood to weather for six months prior to application. Remove all coatings and paint (or test for compatibility) to ensure proper bonding. Do not use on fire retardant lumber.

Priming

In most instances X-Tite StrongSeal will not require a primer. However, certain applications or substrates may require a primer to ensure a long lasting bond and weatherproof seal. It is the applicator's responsibility to determine the need for a primer. X-Calibur recommends a primer be used for any application where prolonged immersion is anticipated.

Application Instructions

Remove all dirt, oil, loose paint, frost and other contamination from all working surfaces with alcohol. DO NOT USE petroleum solvents such as mineral spirits or xylene. Maintain X-Tite StrongSeal at room temperature before applying to ensure easy gunning and leveling. Test and evaluate to ensure adequate adhesion. Carefully gun the sealant with a smooth, continuous bead. If tooling is needed, do so within fifteen minutes of application.

Clean Up

Wet sealant can be removed using a solvent such as alcohol. Cured X-Tite StrongSeal can be removed by abrading or scraping the substrate.

Limitations

In areas where prolonged chemical exposure is anticipated, contact Technical Services for recommendations. Allow treated wood to "cure" for six months prior to application per APA guidelines.

Do not use in areas subject to continuous immersion.

Do not store in elevated temperatures.

Remove all coatings and sealers before application.

Do not apply at temperatures below 30F.

Health and Safety

This product is for industrial use only by trained operatives. It is potentially hazardous if not used correctly. Please refer to the Material Safety Data Sheet (MSDS) prior to the purchase and use of this product. The MSDS can be obtained via our website www.x-calibur.us

Authorized Technical Specialist

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Datasheet Validity

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Warranties

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www.x-calibur.us

X-Shield AcrylCoat

Single component elastic acrylic waterproof coating

20510510

Product Description

X-Shield AcrylCoat is a high performance, multi function elastic acrylic coating used for waterproofing. A solar reflective grade is available on request.

Composition

X-Shield AcrylCoat is a single component, solvent free coating manufactured using a high performance acrylic co-polymer, fillers and wetting agents.

Advantages

- Waterproof
- Elastic
- UV stable
- Durable
- Breathable
- Can be applied to one day old concrete
- Environmentally friendly
- Non-toxic

Uses

- Wet area waterproofing
- Roof waterproofing
- Restoration of metal roofs
- Restoration of fiber cement roofs

Specification Compliance

Classified under BS EN 1062-1:2002 as Class C₁ carbon dioxide permeability.

Laboratory Test Data

Property	Typical Results
Water permeability	Nil at 10m head
Tensile strength	>1 .0MPa (145psi)
Elongation	>350%
Water vapor transmission	2.3g/m ² .h
Rapid chloride permeability	Negligible

The above data relates to X-Shield AcrylCoat after 28 days curing.

Color

Grey and white.

Theoretical Coverage

0.5ltr per coat, 2 coats recommended to give 1mm wft.

Packaging

19 Ltr pails.

Shelf Life

12 months when stored below 30C (86F) under shade in a dry environment.

Installation Guidelines

X-Shield AcrylCoat should be applied by experienced applicators. X-Calibur provides detailed method statements on all its products for use in various applications. These must be referred to prior to starting work. The information below is a summary intended for guidance only.

Surface Preparation

The substrate must be structurally sound. Loose or unsound concrete should be removed and made good. Surfaces must be entirely free of oil, grease, paint, corrosion deposits, dust, laitance or other surface deposits. The surface should be prepared by light grit blasting or high pressure water blasting to produce a lightly exposed aggregate surface. Any bug holes should be filled with a suitable filler.

Priming

The surface should be primed using X-Shield AcrylCoat diluted with 20% clean potable water. Apply the primer coat at the rate of 0.3kg/m² using roller, brush or airless spray. Allow to dry completely (8 hrs at 25C) before applying subsequent coats.

Application

Stir well prior to use. Apply using brush, roller or airless spray. Apply at 1 kg/m² per coat. Apply subsequent coats once the coat beneath has completely dried. Subsequent coats should be applied in the perpendicular direction (at 90°) to the previous coat. The coating can be reinforced using X-Shield CrackMat in areas subject to movement.

Limitations

Do not add water to the mixed coating.
Temperature of the mixed coating should be less than 30°C at time of mixing.
Do not apply to substrates that have free water on them.
Not suitable for use with acids.
Do not apply below 5C.
Avoid excessive application.
Avoid skin contact.
Do not discard into the water system.
Apply only on to slabs that have a waterproofing system installed in order to prevent blistering due to osmosis.

Health and Safety

This product is for industrial use only by trained operatives. It is potentially hazardous if not used correctly. Please refer to the Material Safety Data Sheet (MSDS) prior to the purchase and use of this product. The MSDS can be obtained via our website www.x-calibur.us.

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www.x-calibur.us



COST ESTIMATE FOR REPAIRS TO DAMAGED BUILDING FOR
MR. NALAKA PATHBERIYA AT NO:27, WELLAMPITIYA ROAD, AMBATHALE
MULLERIYAWA
NBRO/PMD/2019/CR-034

ITEM NO	DESCRIPTION	UNIT	QTY	RATE (RS:)	AMOUNT (RS:)
A	<u>PRELIMINARY & GENERAL ITEMS</u>				
1	Provide Lump Sum for necessary protective measures including scaffoldings approximately 20'-0" height from ground level up to roof level to clearing the debris from the existing building.(cat walks, safety nets & belts and brasings)	LS			50,000.00
2	Allow Lump Sum for providing accomadations for house owners during the rectification period.	LS			200,000.00
3	Provide Lump Sum for construction of temperary hut for storage facilities.	LS			25,000.00
4	Allow Lump Sum for providing necessary safety arrangements for adjoining building during the repairing period. [Shorings/ supportings as per the Engineer's specifications]	LS			300,000.00
B	<u>REMOVALS & DEMOLISHING</u>				
1	Demolishing of the existing damaged/ cracked 9" thick brick wall and clearing the debris away from premises. [Approximately 5 Cubes] Rate shall include for necessary supports for tool as directed and making good for existing and adjoining walls and structures.	Item		30,000.00	30,000.00
2	Demolishing of the existing RRM foundation and clearing the demolished items away from the premises. [Approximately 1.5 Cubes] Rate shall include for making good for existing/ adjoining foundation.	Item		10,000.00	10,000.00
3	Demolishing of the existing damaged/ cracked floor area including floor tiles and floor concrete and clearing all debris away from the premises. [Approximately 2.0 Cubes]	Item		40,000.00	40,000.00
4	Removal of the damaged/ decayed timber/ asbestos ceiling with timber frame work and clearing the debris away from premises. [Approximately 5.0 Sqrs]	Item		25,000.00	25,000.00
5	Removal of the existing electrical wiring and fittings including all the accessories and stacking them carefully to reuse after the repairing is completed.	Item		5,000.00	5,000.00

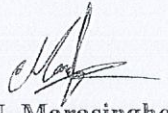
EM O	DESCRIPTION	UNIT	QTY	RATE (RS:)	AMOUNT (RS:)
3.6	Removal of the existing doors and windows and stacking them carefully to reuse after the repairing is completed.	Item		10,000.00	10,000.00
3.7	Removal of existing furniture items such as tables, chairs, computer tables and almairas etc. with due care as directed by the Employer and covered them with a suitable material to protect them from dust and etc...	LS			15,000.00
C	<u>EXCAVATION & EARTH WORK</u>				
2.1	Machine/ manual excavation for wall foundation, columns, footings in any material (except rock blasting/sledging) up to 5'-0" depth, part return fill in and rammed and disposal of surplus excavated materials as directed.	Cube	3.67	4,500.00	16,515.00
2.2	Back filling with selected existing soil/ imported soil and compacted well with using a machine rammer in working order.	Cube	7.00	6,000.00	42,000.00
D	<u>CONCRETE WORKS</u>				
3.1	Supply and placing of lean concrete [C15/20] for the base of the stiffer columns and floor curing concrete for 7 days.	Cube	4.75	43,865.00	208,358.75
3.2	Supply and placing of concrete [C25/20] for columns and footings. Rate shall include for curing concrete for 7 days.	Cube	13.43	63,675.00	855,155.25
E	<u>FORM WORK</u>				
3.1	Smooth finished form work for sides of columns and plinth beam including dismantling in 15mm thick plywood.	Sqr	7.15	15,600.00	111,540.00
F	<u>REINFORCEMENTS</u>				
3.1	Tor steel reinforcement bent to shape, laid in position and tied with 16 B.W.G. Aneal binding wire.	kg	375.00	230.00	86,250.00
3.2	Mild steel reinforcement bent to shape, laid in position and tied with 16 B.W.G. Aneal binding wire.	kg	125.00	220.00	27,500.00
G	<u>MASONRY WORKS</u>				
3.1	Random rubble (6"-9") masonry in cement mortar 1:5 in wall foundation.	Cube	5.87	42,450.00	249,181.50
3.2	9" thick brick work in cement and sand 1:8 in walls.	Cube	6.50	55,000.00	357,500.00
H	<u>FLOOR TILING</u>				
3.1	Matt finished floor tiles of size 24" x 24" of "Rocell" or approved equivalent matching to the existing tiles, paying on 1:3 cement sand bedding and pointed with tile grout to match the tiles. (Prime cost of a tile is Rs. 1050/=)				

EM ID	DESCRIPTION	UNIT	QTY	RATE (RS:)	AMOUNT (RS:)
	Rate shall include for necessary skirting works. [Colour & design to be approved by the Employer]	Sqr	4.25	44,000.00	187,000.00
	<u>PLASTERING WORKS</u>				
1	Plastering 5/8" thick to walls in cement and sand 1:5 finished semi-rough.	Sqr	17.00	8,210.00	139,570.00
2	Applying two coats of skim coat (Not exceeding 3 mm thick) to walls including preparation of surface.	Sqr	17.00	2,895.00	49,215.00
3	Plastering 5/8" thick to reveals width not exceeding 6" in cement and sand 1:5 finished smooth with skim coat.	Lft	85.00	265.00	22,525.00
	<u>PAINTING WORKS</u>				
1	Painting walls with one coat of alkali resistant wall primer including preparing surface.	Sqr	17.00	1,872.00	31,824.00
2	Painting walls with two coats of weathershield paint including preparing surface.	Sqr	4.00	3,790.00	15,160.00
3	Painting walls with two coats of emulsion paint including preparing surface.	Sqr	13.00	3,375.00	43,875.00
	<u>CEILING WORKS</u>				
1	Asbestos flat ceiling fixed with coves & fillets to local Micro fillet (Eucalyptus Microcoris) or Balau, Bitis or Kempas timber frame work of 94mm x 45mm @ 4' - 0" C/C , 45mm x 45mm @ 2' - 0" C/C, size to be 1 ½" x ½" and cove moulding size to be 2"x 2".	Sqr	7.00	45,070.00	315,490.00
	<u>ROOF WORKS</u>				
1	Rearrangement of existing roof with calicut pattern tiles using existing timber frame work comprising of 165mm x 45mm plate, 94mm x 70mm wall plate, 94mmx45mm rafters @ 18" and 94mm x 45mm reepers @ 12" C/C and application of preservative. Rate shall include for necessary ridge tiles and valance/ barge boards, gutters and down pipes in working [Approximately 7.0 Sqr]	Item		500,000.00	500,000.00
	<u>MISCELLANEOUS WORKS</u>				
1	Refixing of the exsisting doors, windows and fan lights including necessary locks and other accessories in working order.	Item		50,000.00	50,000.00
2	Rearrangement of exsisting electrical wirings including DB panel, fittings and other accessories in working order after repairing is completed.	Item		50,000.00	50,000.00

ITEM NO	DESCRIPTION	UNIT	QTY	RATE (RS:)	AMOUNT (RS:)
3	Rearrangement of existing removed furniture items such as tables, chairs, computer tables and almairas etc...at necessary locations after the repairing is completed.	Item		20,000.00	20,000.00
	Total				4,088,659.50
	Add 10% Contingency				408,865.95
	Grand Total				4,497,525.45

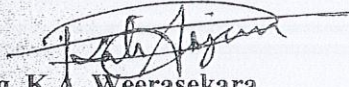
Say 4,500,000.00

Prepared by:



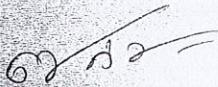
C.N. Marasinghe
Management Assistant - Technical
Project Management Division

Checked by:



Eng. K.A. Weerasekara
Civil Engineer
Project Management Division

Certified by:



Eng. J.K. Jayawardena
Director - Project Management Division

Date: 11.06.2019

Annexure II



SRI LANKA LAND RECLAMATION & DEVELOPMENT CORPORATION

Engineering Materials Testing Laboratory

FIELD BOREHOLE LOG

Project	MCUDP- Improvements to Madiwela East Diversion Scheme - Stage III				
Location	As per the drawing (Annex I)			Hole No	BH - 01
Date of Drilling	19 th to 21 st Sep 2015	Drill Machine	D 45 (Rotary)	Casing size	NW
Ground Elevation	4.908 MSL	Depth of Hole	19.40 m	Angle from Vertical:	0°
Drilled by	J.V.N. Kumara	Logged by	K.W.P. Chandika	SPT Hammer Type:	Automatic Drop Hammer
Water Level in BH	1.658 MSL	Weather	Rainy		

Elevation (MSL)	Depth (m)	Thickness (m)	Colour	Description / Classification	Sample Type	Standard Penetration Test				
						Depth (m)	Number of Blows (N)			
							(N)	10	20	30
4.808	0.10	0.10	Dark Brown	Organic clay mixed with gravel & roots	DS					
3.908	1.00	1.90	Brown	Firm to Soft slightly Gravelly Sandy CLAY. Possible Filled material.	SPT	1.00	06			
						15 -	05			
						15 -	03			
2.908	2.00					2.00	02			
		2.80	Brown	Very Soft Slightly (fine) Sandy CLAY.	SPT	15 -	01			
						15 -	01			
						15 -	01			
1.908	3.00					3.00	<01			
					SPT	15 -	01			
						15 -				
1.408	3.50									
		2.80	Brown	Very Soft Slightly (fine) Sandy CLAY.	UD	4.00	No SPT due to UD sampling			
0.108	4.80									
-0.092	5.00									



SRI LANKA LAND RECLAMATION & DEVELOPMENT CORPORATION
Engineering Materials Testing Laboratory
FIELD BOREHOLE LOG

Project	MCUDP- Improvements to Madiwela East Diversion Scheme - Stage III				
Location	As per the drawing (Annex I)			Hole No	BH - 01
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Drilled by	J.V.N. Kumara	Logged by	K.W.P. Chandika	SPT Hammer Type:	Automatic Drop Hammer
Water Level in BH	1.658 MSL	Weather	Rainy		

Elevation (MSL)	Depth (m)	Thickness (m)	Colour	Description / Classification	Sample Type	Standard Penetration Test				
						Depth (m)	Number of Blows (N)			
						15 -	01			
						15 -	01			
					SPT	15 -				
		2.00	Brownish Black	Plastic Clayey Pseudo-fibrous PEAT		6.00	01			
						15 -	SW			
					SPT	15 -	01			
					UD					
-1.892	6.80									
-2.092	7.00				SPT	7.00	13			
						15 -	04			
						15 -	06			
						15 -	07			
-3.092	8.00	2.20	Yellowish Brown	Medium dense Clayey Gravelly (Medium to Coarse) SAND		8.00	10			
					WS	15 -	04			
						15 -	05			
						15 -	05			
-4.092	9.00					9.00	11			
					SPT	15 -	03			
						15 -	05			
						15 -	06			
-5.092	10.00		Dull Yellowish Brown	Medium dense slightly Gravelly Clayey (Medium to Coarse) SAND		10.00	18			



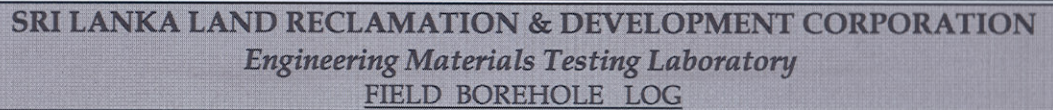
SRI LANKA LAND RECLAMATION & DEVELOPMENT CORPORATION
Engineering Materials Testing Laboratory
FIELD BOREHOLE LOG

Project	MCUDP- Improvements to Madiwela East Diversion Scheme - Stage III		
Location	As per the drawing (Annex I)	Hole No	BH - 01
Date of Drilling	19 th to 21 st Sep 2015	Drill Machine	D 45 (Rotary)
Ground Elevation	4.908 MSL	Depth of Hole	19.40 m
Drilled by	J.V.N. Kumara	Logged by	K.W.P. Chandika
Water Level in BH	1.658 MSL	Weather	Rainy
		SPT Hammer Type	Automatic Drop Hammer

Elevation (MSL)	Depth (m)	Thickness (m)	Colour	Description / Classification	Sample Type	Standard Penetration Test					
						Depth (m)	Number of Blows (N)				
							(N)	10	20	30	40
		2.00	Dull Yellowish Brown	Medium dense slightly Gravelly Clayey (Medium to Coarse) SAND	SPT	15 - 06 15 - 08 15 - 10					
-6.092	11.00				11.00	08					
		1.00		Loose slightly Gravelly Clayey (Medium to Coarse) SAND	SPT	15 - 03 15 - 04 15 - 04					
-7.092	12.00				12.00	18					
		3.00			SPT	15 - 03 15 - 08 15 - 10					
-8.092	13.00					13.00	10				
					Medium dense slightly Gravelly Clayey (Medium to Coarse) SAND	SPT	15 - 04 15 - 05 15 - 05				
-9.092	14.00					14.00	16				
						SPT	15 - 09 15 - 09 15 - 07				
-10.092	15.00					15.00	21				



Elevation (MSL)	Depth (m)	Thickness (m)	Colour	Description / Classification	Sample Type	Standard Penetration Test				
						Depth (m)	Number of Blows (N)			
							(N)	10	20	30
					SPT	15 - 07 15 - 10 15 - 11				
-11.092	16.00	2.00	Dull Yellowish Brown	Medium dense Clayey Gravelly (Medium to Coarse) SAND		16.00 19				
					SPT	15 - 07 15 - 09 15 - 10				
-12.092	17.00					17.00 20				
			Yellowish Brown	Medium dense Clayey very Gravelly (Medium to Coarse) SAND with Quartz	SPT	15 - 06 15 - 10 15 - 10				
-13.092	18.00					18.00 13				
					SPT	15 - 08 15 - 07 15 - 06				
-14.092	19.00					19.00 >50				
-14.242	19.15	0.15		Completely Weathered Rock	SPT	15 - 15 15 - >50				
-14.492	19.40	0.25		Completely to Highly Weathered Rock	CS	15 - -				



Elevation (MSL)	Depth (m)	Thickness (m)	Colour	Description / Classification	Sample Type	Standard Penetration Test				
						Depth (m)	Number of Blows (N)			
							(N)	10	20	30
					SPT	15 - 07 15 - 10 15 - 11				
-11.092	16.00	2.00	Dull Yellowish Brown	Medium dense Clayey Gravelly (Medium to Coarse) SAND		16.00 19				
					SPT	15 - 07 15 - 09 15 - 10				
-12.092	17.00					17.00 20				
					SPT	15 - 06 15 - 10 15 - 10				
-13.092	18.00		Yellowish Brown	Medium dense Clayey very Gravelly (Medium to Coarse) SAND with Quartz		18.00 13				
					SPT	15 - 08 15 - 07 15 - 06				
-14.092	19.00					19.00 >50				
-14.242	19.15	0.15		Completely Weathered Rock	SPT	15 - 15 15 - >50		Hammer rebounded with no penetration in 2nd 15cm		
-14.492	19.40	0.25		Completely to Highly Weathered Rock	CS	15 - -				

**SRI LANKA LAND RECLAMATION & DEVELOPMENT CORPORATION****Engineering Materials Testing Laboratory****FIELD BOREHOLE LOG**

Project	: MCUDP- Improvements to Madiwela East Diversion Scheme - Stage III		
Location	: As per the drawing (Annex I)	Hole No	: BH - 01
Date of Drilling	: 19 th to 21 st Sep 2015	Drill Machine	: D 45 (Rotary)
Ground Elevation	: 4.908 MSL	Depth of Hole	: 19.40 m
Drilled by	: J.V.N. Kumara	Logged by	: K.W.P. Chandika
Water Level in BH	: 1.658 MSL	Weather	: Rainy
		SPT Hammer Type:	Automatic Drop Hammer

NOTE

- ~ Borehole terminated on completely to highly weathered rock layer at 19.40m (-14.492 MSL) depth.
- ~ Levels are presented in here with reference to the Mean Sea Level (MSL).
- ~ SPT - Standard Penetration Test (Sample)
 - DS - Disturbed Sample
 - UD - Undisturbed Sample
 - CS - Core Sample
 - SW - Self Weight of the SPT hammer
- ~ Rainy condition prevailed throughout the testing period.

Logged by

Senior Testing Officer

Engineer

Certified by

Engineer

Engineering Materials Testing Laboratory
Sri Lanka Land Reclamation & Development Corporation

E M T Laboratory

S.L.L.R. & D.C.

No. 130A, Attidiya Road, Attidiya.

Tel: 2714409, 2714517(Fax)

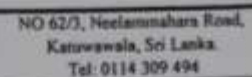
Reviewed by

Engineer

19 October 2015

Date

Annexure III



BH-08

1 of 3

Ground Water level	0.3
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	$X = 400988.9456$
--	-------------------

Coordinates	Y = 495101.3959
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Annexure IV



නාගරික සංවර්ධන, ජල සම්පාදන හා නිවාස පහසුකම් අමාත්‍යාංශය
நகர அபிவிருத்தி, நீர் வழங்கல் மற்றும் வீடமைப்பு வசதிகள் அமைச்சு
Ministry of Urban Development, Water Supply and Housing Facilities



කොළඹ අගනගර ආශ්‍රිත නාගරික සංවර්ධන ව්‍යාපෘතිය
கொழும்பு மாநகர அபிவிருத்தி திட்டம்
METRO COLOMBO URBAN DEVELOPMENT PROJECT

දුරකථන தொலைபேசி Telephone	+94-112-187047 +94-112-187048	ෆැක්ස් தொலைநகல் Fax	+94-112-186009	විද්‍යුත් තැපෑල மின்னஞ்சல் Email	informcudp@gmail.com	මගේ අංකය. எனது இல. My Reference.	MCUDP/06/SOC/NP/LA/01/2013	බිග් අංකය. உமது இல. Your No.
---------------------------------	----------------------------------	---------------------------	----------------	--	----------------------	--	----------------------------	------------------------------------

24.01.2020

Mrs. Indra Malani

No. 14/19, Sedawatta Road,

Wellampitiya.



Building Condition Technical Evaluation Report & Invoice

This has reference to your letter dated 07th January 2020. As per your request, we have sent the request letter to General Manager of Central Engineering Consultancy Bureau (CECB) and received the Building Condition Technical Evaluation Report & Invoice of this task.

Accordingly, we are forwarding the said report and Invoice for the payment of Rs. 54,000.00 of CECB for your necessary action please.

Project Director

Metro Colombo Urban Development Project

Cc: Dr. A.U. Gunasekara – Team Leader/ Client's Representative, NLPST Project

Signature

Mr. Zhang Yuan – Project Manager (China Geo Engineering)

විකල්පී කළමනාකරණ ඒකකය,
12 වන මහල, සෙත්තිරිපාය අදිශර II,
බත්තරමුල්ල, ශ්‍රී ලංකාව.

Project Management Unit,
12th Floor, Sethsiripaya (Stage II),
Battaramulla, Sri Lanka.

செயற்றிட்ட முகாமைத்துவ பிரிவு
12வது மாபி, “செத்சிரிபாய்” கட்டம் II
பத்தரமுல்லை, இலங்கை.



දුරකර්මය කාර්යයන් පිළිබඳ මධ්‍යම උපදේශක කාර්යාංශය
மத்திய பொறியியல் உசாதுணைப் பணியகம்
CENTRAL ENGINEERING CONSULTANCY BUREAU



415, බොද්ධාලෝක මාවත, කොළඹ 07, ශ්‍රී ලංකා
415, பொத்தாலோக மாவத்தை, கொழும்பு 07, இலங்கை
415, Bauddhaloka Mawatha, Colombo 07, Sri Lanka

දුරකර්මය / தொலைபேசி / Telephone 2668800 Fax : 2687369
E-Mail: cecb@cecbsl.com
website: www.cecblk

Your ref.

My ref. 2019/E9/01

17th January 2020

Eng. (Mrs.) A.H. Thushari
Project Director
Metro Colombo Urban Development Project
Project Management Unit
12th Floor, Sethsiripaya (Stage II)
Battaramulla

Dear Madam,

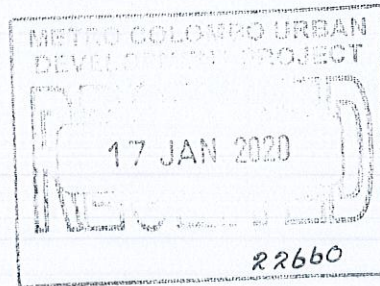
Building Condition Technical Evaluation Report of Dwelling House No. 14/19, Sedawatta Road, Wellampitiya
SUBMISSION OF REPORT

Please find enclosed herewith the Condition Assessment Report along with our invoice for the consultancy work.

Early settlement of this invoice is highly appreciated.

Yours faithfully,
CENTRAL ENGINEERING CONSULTANCY BUREAU

Eng. (Ms) S. W. Jayawardane
Additional General Manager (Designs - 1)



CENTRAL ENGINEERING CONSULTANCY BUREAU

No.415, Bauddhaloka Mawatha, Colombo 7, Sri Lanka.

Telephone

94 11 2668951

Fax

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E-mail

dec@cecbsl.com

Website

www.cecbsl.com

DESIGN 1 SECTION / Structural Design 3 Unit

INVOICE

VAT Registration No. 409000100-7000

Client : Project Director,
(Name & Address) Metro Colombo Urban Development Project,
Project Management Unit,
12th Floor ,
Sethsiripaya (Stage II),
Battaramulla.

Project : Building Condition technical evaluation
report of Dewelling house No.14/19
,Sedawatta Road , Wellampitiya

Project Invoice No : 1

Date : 17th January 2020

Project Cost (Rs) (Excl. Taxes)

Estimated :

Final :

Consultancy Fee (Excl. Taxes) Rs.50,000.00

Invoice No : I/SD 3/2020/03

Services Provided	Fee		Work completed	Amount (Rs)
	Basis	Amount (Rs)		
Condition evaluation report	LS	50,000.00	100%	50,000.00
Total Amount (Excl. Taxes)				50,000.00
VAT 8%				4,000.00
Total Amount (Incl. Taxes)				54,000.00
Less: Payments Received				
Reference	Invoice No	Date Received	Cheque No.	Amount
Total Amount Due (Incl. Taxes)				54,000.00
Total Amount Due in words Rupees Fifty Four Thousand Only				
The Tax Invoice will be issued at the time of Payment Received				
Please draw your cheque in favour of CENTRAL ENGINEERING CONSULTANCY BUREAU				
Signature Name Date	Prepared by	Checked by	Authorized by	
	N.M.Kalpage	Acct.H M S Malkanthi	Eng.S.W.Jayawardane, AGM(D1)	
	17-Jan-2020	17-Jan-2020	17-Jan-2020	

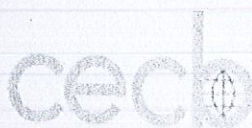
**BUILDING CONDITION
TECHNICAL EVALUATION REPORT
OF
DWELLING HOUSE**

**AT NO. 14/19, SEDAWATTA RODA,
WELLAMPITIYA**



Report No: CB/SR/2020/01

January 2020



Central Engineering Consultancy Bureau (CECB)

415, Bauddhaloka Mawatha, Colombo07

Sri Lanka

Tel: +0094 11 266 88 00, Fax: +0094 11 2668967

BUILDING CONDITION TECHNICAL EVALUATION REPORT OF DWELLING HOUSE NO 14/19, SEDAWATTA RODA, WELLAMPITIYA.

A. Background and Introduction

- A.1 This report is submitted by the Central Engineering Consultancy Bureau (CECB) in fulfillment of the assignment given by the Project Director, Metro Colombo Urban Development Project.
- A.2 According to the Project Management Unit of North Lock Pumping Station Project (PMU/NLPST), some structural damages to above mentioned dwelling house (located at no 14/19, Sedawatta road, Wellampitiya) had been observed during construction works of the adjoining North Lock Pumping station Project. Location map is shown in Fig. A.1.
- A.3 Part of the dwelling house consists of two story structure with reinforced concrete column-beam frames, reinforced concrete slab and timber framed asbestos roof, while the rest is of single storied structure, by the side of two story part, with masonry walls and timber framed asbestos roof. As per pre-condition survey report done in 2017, the dwelling house had been constructed approximately 6 years before the survey.
- A.4 Reported construction operation activities taken place at the NLPST site are sheet pile driving (to a depth of 12 m by vibro method), dewatering (lowering ground water to a depth of around 6 m), movement of heavy equipment etc.
- A.5 Scope of this report is to provide condition evaluation report with recommendations by reviewing the site visit observations done by the CECB on 17/12/2019 along with the observation records (Details of crack survey on 19/11/2017, Report on damage inspection on 21/12/2018 and Results of ground settlement monitoring points) provided by the PMU/NLPST.

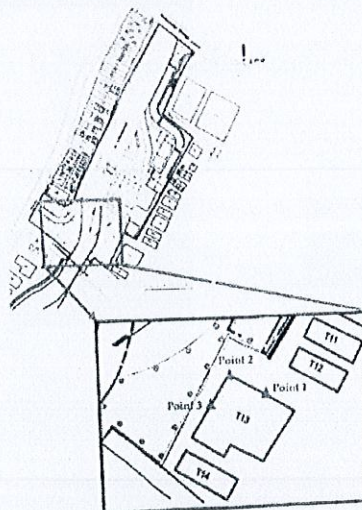


Fig. A.1 Location map of house No 14/19 (marked as Block No. T13) adjacent to construction site.

B. Visual Observations by the CECB on 17th December 2019

Some photographs taken at the site visit are presented below;



Fig. B.1 Front and rear views of No. 14/19 dwelling house

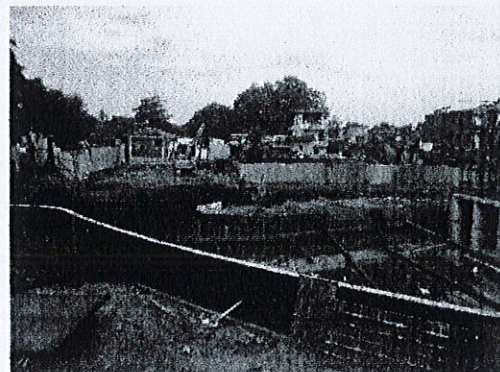
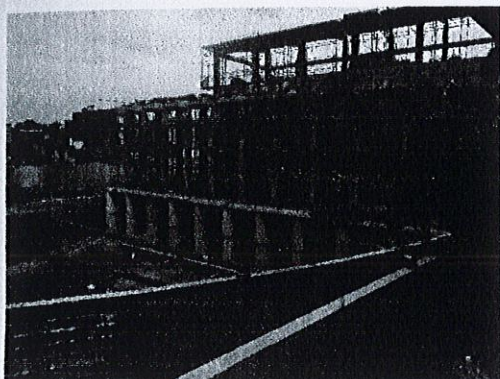


Fig. B.2 Adjacent construction of Pumping Station



Fig. B.3 Wall and floor cracks in single storied front (living) area

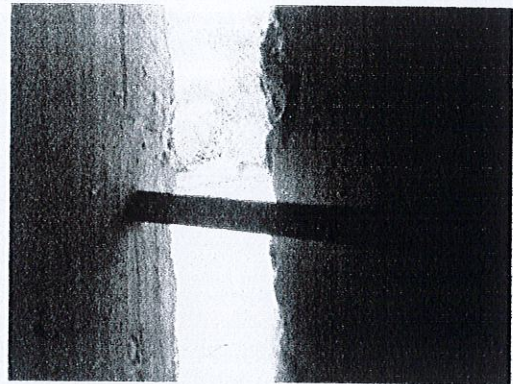
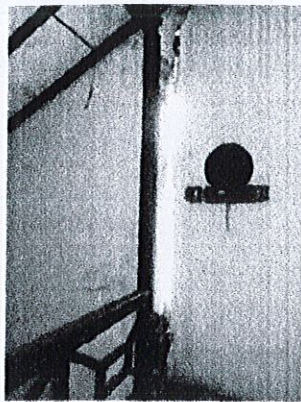


Fig. B.4 Wide open crack at upper floor wall

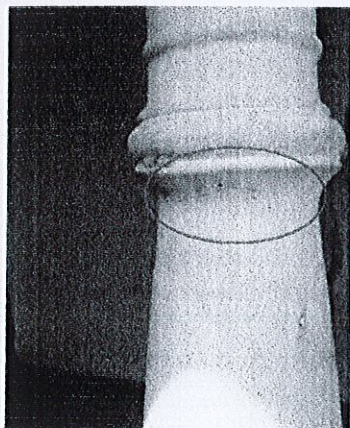


Fig. B.5 Cracks in front varandha columns and arrangement of roof rafters, some of which are about to dislodge from their connections

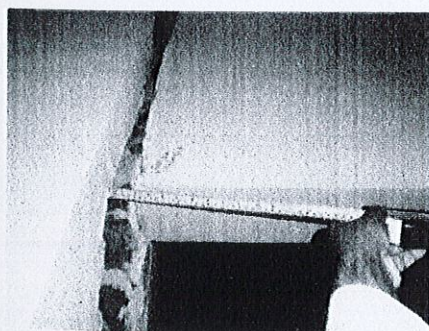


Fig. B.6 Cracks in walls at single story area (at rear and by the side of two story portion).

C. Summary of crack monitoring data done by the PMU/NLPST, starting from 19th Nov. 2017.

- C.1 At the time of pre-condition survey of adjacent buildings, the said dwelling unit (No. 14/19) had been registered as Block No. T13, and the layout plan is reproduced here in figure A.1. As per the crack survey carried out on 19th November 2017, following information can be gathered.
- i. All cracks had been on masonry walls. No records of cracks on ground floor slab (on grade), upper floor slab or RC frames (column/beam).
 - ii. In 1st floor walls, cracks had been observed only at 5 locations, out of which 2 locations had been on balcony walls. All the 1st floor wall cracks were within 1-2 mm widths.
 - iii. In Ground floor walls, cracks had been recorded at 29 locations, and only one (horizontal) crack was in order of 4mm crack width and the rest were within 1-3 mm crack widths. Out of 29 cracks in Ground floor walls, 45% of the cracks were diagonal, 41% of cracks were vertical and the rest had been horizontal.
- C.2 As per the ground settlement monitoring observations during 19th May 2018 to 12th October 2019, rear side of the building had undergone maximum settlement of 101 mm. According to the readings of the 3 settlement monitoring points, the structure had tilted, affecting verticality of the building. Differential settlement between two closer-by monitoring points had been 17 mm (=101 mm-84 mm)
- C.3 As per the inspection of 19th and 20th December 2018, further widening of existing cracks, appearing of new cracks on walls and appearing of cracks on ground floor slab (on grade) had been reported.
- C.4 According to the borehole data provided by the PMU/NLPST, subsurface condition (up to 8m depth) consists of loose sand layers with very low SPT values (predominantly around 1 to 3).

D. Concluding remarks

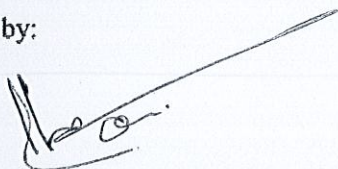
- D.1 According to crack survey and settlement monitoring carried out by PMU/NLPST, it is very likely that the said dwelling house (No. 14/19) has been tilted/settled due to ground settlement caused by movement/vibratory compaction of loose sand strata during dewatering, excavation and sheet pile driving.
- D.2 It was observed that the two storied part of the dwelling unit has been tilted towards rear side of the building. This tilt can cause some additional stresses on columns and soil under footings at rear side of the building. Since there are not much evidence of cracks in reinforce concrete elements, it can be assumed that the whole two storied block is tilting as a one unit. Available data is not adequate enough to conclude whether the settlement has seized at present.
- D.3 Structural integrity of walls in this dwelling unit has been lost due to wide cracks in brick wall interconnections. Some roof connections are at vulnerable state as they dislodged due to tension forces caused by tilt/settlements.
- D.4 Allowable settlement of a building will depend on rigidity of structure, type of construction, presence of sensitive finishes etc. However, as a general rule, a total settlement of 25 mm and a differential settlement of 20 mm between columns (under un-factored load) can be considered safe for buildings supported on isolated pad footings placed on sand. As per the settlement monitoring points, maximum settlement of this building was 101 mm, which is beyond acceptable limits.

- D.5 Accordingly, the present condition, this dwelling house is not structurally safe for the occupants, unless adequate precautions are taken for restoration. Main steps in restoration will consist of arresting further settlement by securing the foundation on competent strata, bringing back the verticality of the structure, repairing wall cracks and restoring integrity, repairing roof connections and relaying of ground floor slab/finishes.
- D.6 When considering subsurface soil profile, the best engineering solution to arrest further settlement/tilts of the structure is to support the existing foundations by means of underpinning with micro piles. Underpinning of foundation is a specialized work that should be performed by an experienced contractor. However, to bring the tilted structure to original position will be a rigorous process, which require jacking-up the affected foundations.
- D.7 It is unlikely that cost of retrofitting which includes underpinning with micro piling and jacking-up will be comparative for a two storied dwelling unit such as this, when compared with demolition and re-built option.

E. Recommendation

- E.1 Due to foundation settlements and loss of integrity, as per present status, dwelling house No. 14/19 is structurally unsafe for the occupants. Due to high technical involvement and cost in retrofitting methods such as underpinning and jacking-up foundation/slab, demolition and rebuilding is recommended as feasible option.
- E.2 It is recommended to consider re-usable material in the existing dwelling house when implementing rebuilding option.
- E.3 Prior to construction works, it is recommended to get advices from a geotechnical engineer on type of foundations and level of soil improvement required for this site.

Prepared by:



Eng. D.L.T.R. Peiris (Specialist Engineer – Structural Designs)

Structural Renovation/SD3/CECB

Annexure V

METRO COLOMBO URBAN DEVELOPMENT PROJECT (MCUDP)

**DAMAGE ASSESSMENT FOR THE
STRUCTURAL DISTRESS OCCURRED AT
JINANANDA CHILDREN'S HOME**

Submitted To:

**THE PROJECT DIRECTOR,
METRO COLOMBO URBAN DEVELOPMENT PROJECT (MCUDP)**

Prepared by:
CENTRAL ENGINEERING CONSULTANCY BUREAU
415, Bauddhaloka Mawatha,
Colombo 7



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2. DESCRIPTION OF THE BUILDING

2.1. GENERAL

Chandigarh Children's Home is situated on Durgam Chaudhary Road, Chandigarh. The building is a three-story building with two semi basement levels (Figure 1). External walls are made of brickwork and the basement levels are made of concrete. The building is situated on a plot of 1000 sq. m.

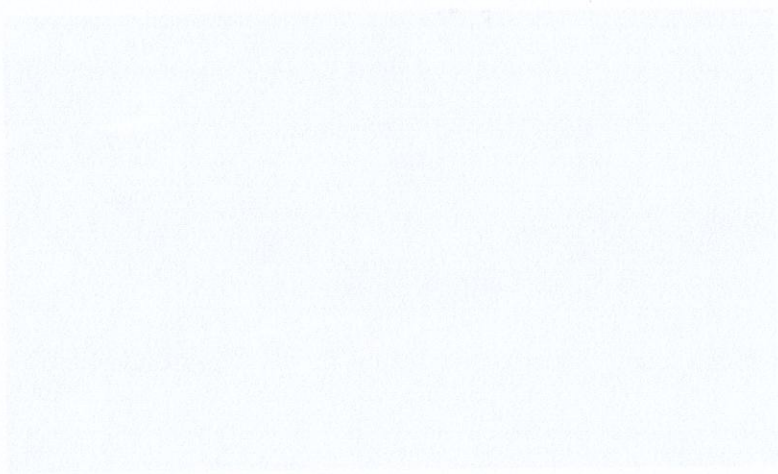


Figure 1 - Location of the Building

1. INTRODUCTION

The Project Director of Metro Colombo Urban Development Project (MCUDP) had requested Central Engineering Consultancy Bureau (CECB) to submit an Engineering Proposal regarding the Jinanada Children's home which had been damaged during the conveyance improvement works at canal; by his letter MCUDP/W/07A/01 dated 03.04.2017.

According to the information received, a proposal had been made by MCUPD to construct flow guidance walls at the upstream of the Galle road bridge of Wellawatta canal in order to streamline the flow and improve the conveyance capacity of the canal, where the bridge width is considerably less than the upstream and downstream canal width. However during the implementation of the project, the existing canal bank had become unstable due to the excavation causing damages to the existing building at the canal bank.

As per the request of the Project Director of Metro Colombo Urban Development Project (MCUDP), affected building was investigated on 18th of April 2017 by CECB and this report is submitted to describe the condition of existing building and to propose retrofitting/repairing methods that can be incorporated. Tentative cost estimate is also given based on preliminary investigations carried out and data received.

2. DESCRIPTION OF THE BUILDING

2.1. GENERAL

Jinananda Children's Home is situated on Dhammarama Road, Wellawatta. The Children's home is a three storey building with two semi basement levels (Figure 1). Ground floor is facing Dhammarama Road and the Basement levels adjoin with the Wellawatta Canal (Figure 2 & Figure 3)



Figure 1 - Location of the Building

During the discussions had when investigations were carried out, it was disclosed that though the entire building connected as a single structure at ground floor which faces Dharmmarama road, the basement levels had been added later to the building that had been built at ground level.



Figure 2 - Ground floor orientation of the building



Figure 3 - Orientation of lower basement and upper basement

2.2. STRUCTURE

The building approximately of 58m in length, 10m in width and 10m in height from its lower basement foundation level, has been used to provide accommodation for children, facilitate classes for children and conduct religious rites.

The structure has three main levels namely, lower basement level, upper basement level and the ground floor level. The building consists of reinforced concrete frame structures in filled with masonry. Timber has been used for the roof with calicut tiles being used as the roofing material of the building. Structural drawings of the building were unavailable and constructions done were found as haphazard.

There are structural columns in different sizes to carry the gravity load of the building and we were informed that some columns in lower basement has equipped with shallow foundations constructed on quarry dust filled concrete cylinders, bored into the hard soil strata. Majority of beam spans range from 3.5 m to 6 m, having different beams depths. Slab thickness of floors are generally varies from 150-200 mm.

There is a brick retaining wall in the lower basement level to support the earth fill below the part of upper basement. Sheet piles constructed along the berm had provided the passive resistance required to maintain the stability of the canal bank where the building foundations were rested.

3. ASSESSMENT METHODOLOGY AND INSPECTION

The objective of condition assessment of existing structure was to produce a brief report on the severity of damage caused to the building and to provide approximate cost estimation for retrofitting/repairing. Further, the inspection was aimed to notify the relevant authorities about the retrofitting methods that could be adopted to enhance the usability of the building.

Assessment was carried out by an initial inspection of the structure.

4. INITIAL INSPECTION AND OBSERVATION

4.1 LOWER BASEMENT (+2.8m MSL)

Lower basement of the building had been affected severely due to the canal bank instability. It was clearly visible that the structure had leaned to the side due to this instability.

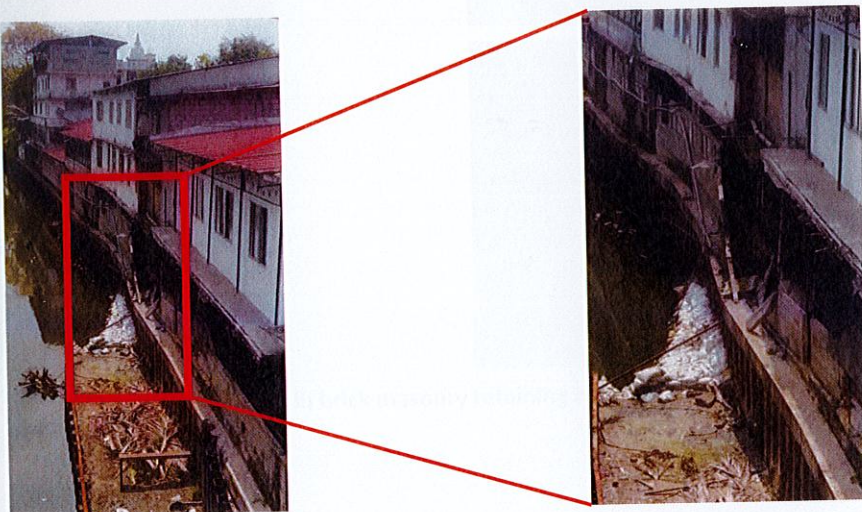


Figure 4 - Location where the canal bank instability had occurred

Three columns in the lower basement near the affected area were identified as significantly worn off. It was noticed that a shear slip had occurred in the ground of this vicinity.

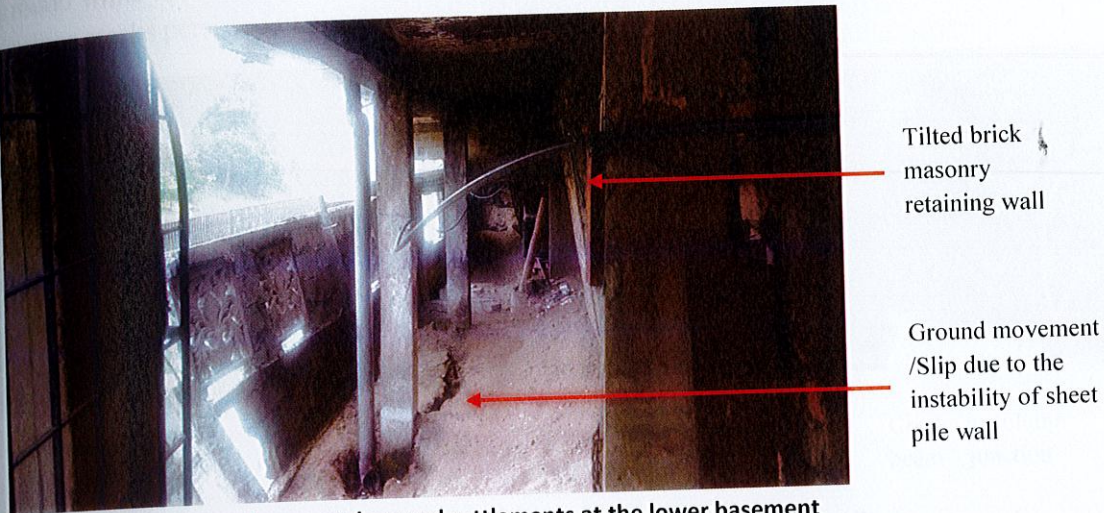


Figure 5 - Obsolete columns and ground settlements at the lower basement

Brick retaining wall in the lower basement area had also leaned/tilted due to the canal bank instability. Excessive lateral force exerted on the brick retaining wall and the poor construction practices adopted could also be a reason for this distortion.

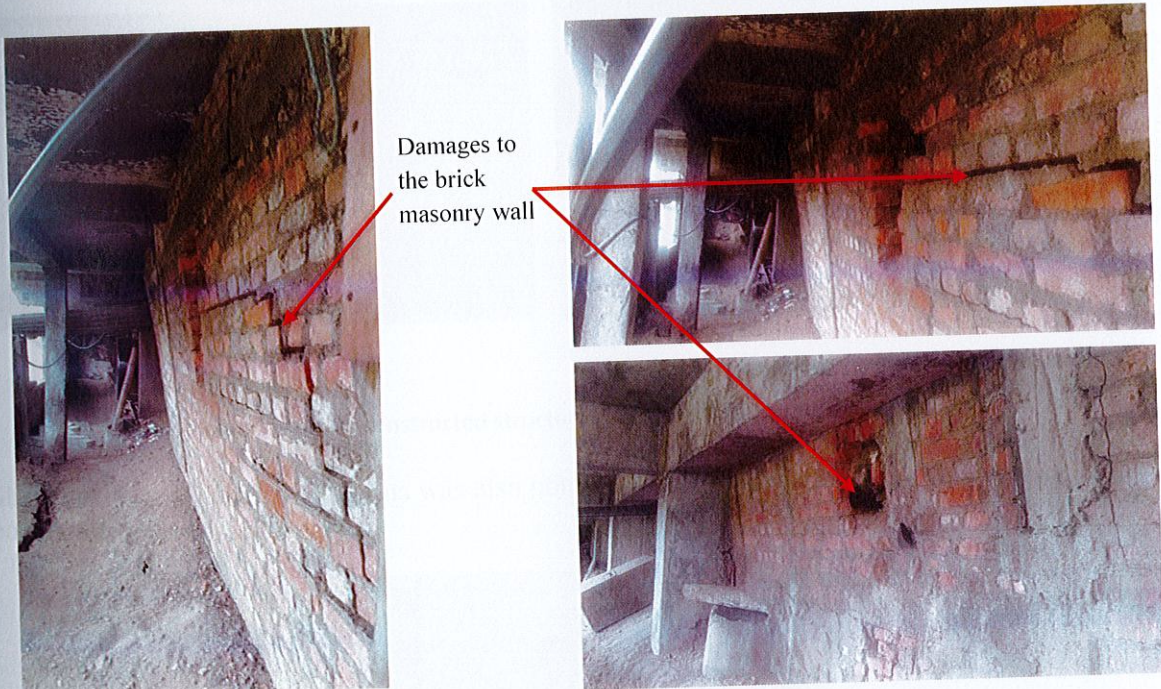


Figure 6 - Damages in brick masonry retaining wall at lower basement

Cracks were found in some of beams, columns and walls. Unaccomplished constructions work were found in the lower basement. Most of those could be due to the settlements occurred in ground while some appears presently could suspected as enlargement of previous hairline cracks / minor cracks persisted in poorly constructed structural elements.

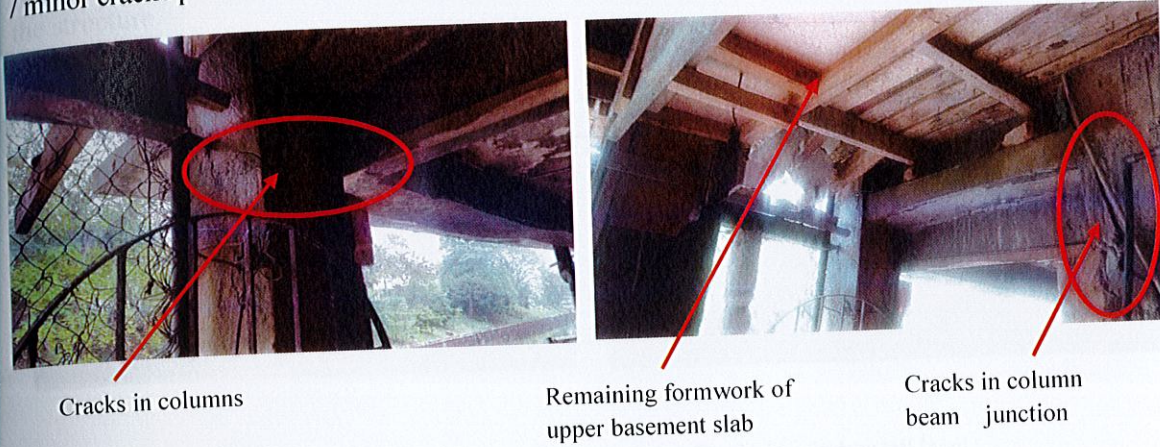


Figure 7 - Cracked structural elements in the lower basement floor

Since beams and columns supporting the upper basement area were found to be in poor construction quality, rectification of those would also be necessary if a proper restoration is done.

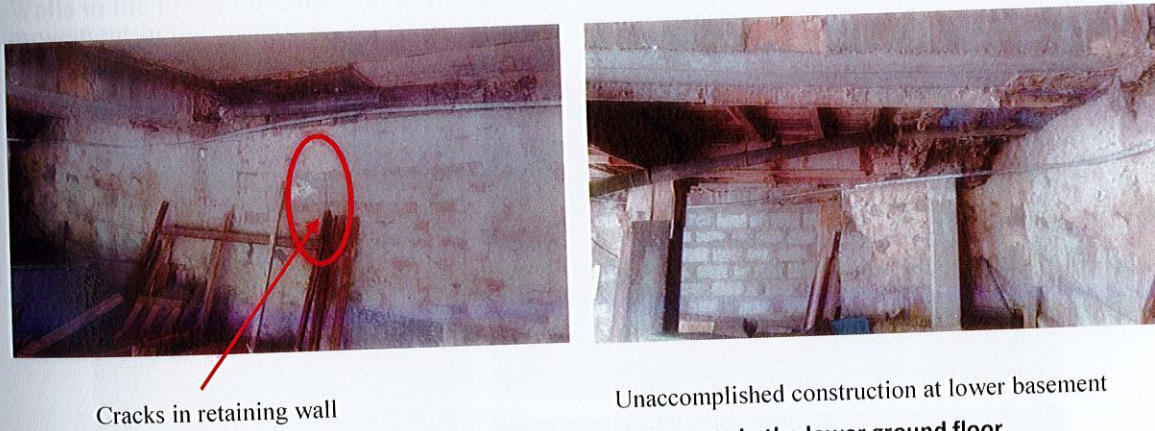


Figure 8 - Poorly constructed structural elements in the lower ground floor

Slight spalling of concrete columns was also noted in the lower basement level, again due to the poor construction quality.



Figure 9 - Slight spalling of concrete in columns

4.2 UPPER BASEMENT (+5.8m MSL)

In upper basement area damage had mainly taken place in brick walls due to the movement of the structure.

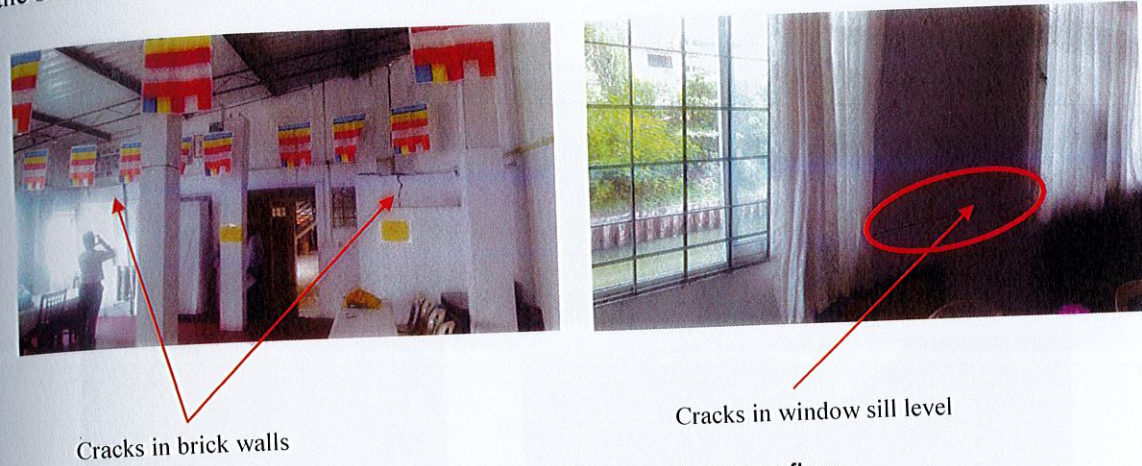


Figure 10 - Cracks observed in upper basement floor

Walls in the upper basement level wash rooms are cracked severely due to the differential movement in the structure.



Figure 11 - Cracks observed in upper basement wash room walls

4.3 GROUND FLOOR (+8.8m MSL)

In ground floor, walls and the slab panels of latter construction had shifted from initially constructed structure due to the movement caused with the settlement. The highest displacements of structural elements were found in the ground floor level.



Figure 12 - Shifted walls and slabs observed in the ground floor

5. METHODS OF RETROFITTING / REPAIRING

Recommendations shall be undertaken in below mentioned repair methods.

1. Restore the canal bank stability.
2. Demolition and reconstruction of settled columns.
3. Demolition and reconstruction of lower basement retaining wall.
4. Demolition and reconstruction of cracked walls.
5. Rectifying the poorly constructed beams, slabs and columns in the lower basement.
6. Rectifying the columns which had subjected for spalling.
7. Anchoring the floor slab where it has subjected to displacements and shifted.
8. Check the level of floor slabs.

5.1 RESTORE THE CANAL BANK STABILITY

Before starting rectification for the building, it is necessary to restore the canal bank stability. This can be done by driving a set of sheet piles along the berm as close as possible to the existing sheet piles of the berm and restore the berm to provide the required lateral passive pressure to stabilize the main sheet pile line. The completion of work shall be carried out as per the instructions / details already given by the geotechnical unit of CECB on this regard.

5.2 DEMOLITION AND RECONSTRUCTION OF SETTLED COLUMNS

Settled columns in the lower basement must be demolished and reconstructed. Before removing the column it is necessary to be certain that the removal of columns would not overstress the beams and slabs above. Therefore it is necessary to take appropriate measures (such as propping the beams) before removing the columns.

Columns shall be constructed on sand/quarry dust filled concrete cylinders driven up to a depth of 3.5 meters where the hard soil could be found according to the soil investigation report, as the underlying soil layer had already been subjected to settlement.

5.3 DEMOLITION AND RECONSTRUCTION OF LOWER BASEMENT RETAINING WALL

Since the brick retaining wall in the lower basement had been tilted and cracked, reconstruction of the retaining wall is required.

The construction can be proceed stage by stage having installed temporary lateral supports to the existing retaining wall if existing wall is demolished, or alternatively new wall could be constructed in touched with existing wall.

It is advisable to construct a concrete retaining wall having adequate foundation depth and width, instead random rubble masonry wall due to the prevailing space restriction at the particular location.

5.4 DEMOLITION AND RECONSTRUCTION OF CRACKED WALLS

Cracked walls in all three levels should be demolished and reconstructed as the damage occurred is beyond repair limits.

Wash room in the upper basement area shall also be demolished completely and reconstructed. Replacing of bathroom fittings in the wash room will also be required.

5.5 RECTIFYING THE POORLY CONSTRUCTED BEAMS, SLABS AND COLUMNS IN THE LOWER BASEMENT

As beams, slabs and columns in the lower basement area were found as poorly constructed, it is recommended to rectify those structural elements with appropriate re-strengthening methodology for the safety of the occupants in building's life span.

However, structural capacity of existing beam could not be determined simply, as there were no structural drawings available. Therefore, a detailed investigation backed by laboratory investigations would only reveal the actual requirements for retrofitting. This was not covered in this report since it was not the objective of this assessment.

5.6 RECTIFYING THE COLUMNS WHICH HAD SUBJECTED FOR SPALLING

Few columns which the concrete had spalled were identified in lower basement level. Hence it is required to replace the spalled concrete both to reinstate the original form and to provide adequate structural capacity, durability and fire resistance.

5.7 ANCHORING THE FLOOR SLAB WHERE IT HAS SUBJECTED TO DISPLACEMENTS AND SHIFTED

Upper basement and Ground floor level floor slabs had been shifted away from the initial constructed section of the building (described in section 2.1 of this report) due to the movement occurred in the structure. Therefore providing reinforcement anchors from shifted slabs to existing non shifted structural elements is required to repair the damages and to provide structural integrity.

5.8 CHECK THE LEVEL OF FLOOR SLABS

Level of all slab panels should be checked to identify any excessive settlement or a deflection. Appropriate rectification works shall be adopted after detailed assessment of floor slab structural capacity, if significant distortion of slabs were noted.

Tentative Engineer's Estimate for the repair cost is attached in Annexure 1.

6. RECOMMENDATIONS

This report only outlines the initial inspection outcomes and suggests brief methodologies possible to rectify damages. A detailed investigation, execution of engineered reconstruction methodology based on the outcomes of such investigation with proper design and detailing of structural elements are necessary if building has to be restored properly. It is recommended to evacuate the children and other personnel until the building is completely restored, if they are still occupying the building.

ANNEXURE 1

TENTATIVE ENGINEER'S ESTIMATE PROPOSAL FOR
STRUCTURAL DISPUTE OF JINANADA CHILDREN'S HOME
AT WALLAWATHA

SUMMARY

Item	Description	Quantity	Unit	Rate	Amount
1	Excavation for foundation	100	m ³	150	15000
2	Foundation concrete	100	m ³	200	20000
3	Reinforcement steel	100	m ³	100	10000
4	Formwork	100	m ²	50	5000
5	Backfilling	100	m ³	50	5000
6	Labour	100	man	100	10000
7	Transport	100	km	100	10000
8	Material	100	kg	100	10000
9	Tools	100	kg	100	10000
10	Other	100	kg	100	10000
11	Subtotal				100000
12	Grand Total				100000

[Signature]
Engineer
Jinanada Children's Home
Wallawatha

DEMOCRATIC SOCIALIST REPUBLIC OF SRI LANKA
MINISTRY OF DEFENCE AND URBAN DEVELOPMENT
PROPOSAL FOR STRUCTURAL DISPUTE OF JINANADA CHILDREN'S HOME AT
WALLAWATHA
TENTATIVE ENGINEER'S ESTIMATE
SUMMARY

Item	Description	Amount LKR
1	DEMOLISH AND RECONSTRUCTION WORK (a) Interlock Sheet pile Concrete retaining wall (b) Columns in +2.8m MSL (c) brick Retaining wall and construction of Concrete Retaining wall (d) Fixing 10mm rebars (e) Brick wall and toilet Area in +5.8m MSL	7,675,733.81
2	MISCELLANEOUS WORK	500,000.00
	SUB TOTAL (1)	8,175,733.81
	Add - Physical Contingencies 10%	817,573.38
	SUB TOTAL (2)	8,993,307.19
	Add - VAT (%) 15%	1,348,996.08
	GRAND TOTAL INCLUDING VAT	10,342,303.27

BREAK DOWN OF TENTATIVE ENGINEER'S ESTIMATE STRUCTURAL DISPUTE OF JINANADA CHILDREN'S HOME AT WALLAWATHA

Item	Discription	Quantity	Unit	Rate	Amount	Total Amount
1	Demolish and reconstruction interlock Sheet pile Concrete retaining wall					
	Demolition	34.00	m ³	11,050.00	375,700.00	
	interlock sheet pile	85.00	m ²	27,540.00	2,340,900.00	
	Concrete	34.00	m ³	19,000.00	646,000.00	
	Formwork	85.00	m ²	1,500.00	127,500.00	3,490,100.00
2	Demolish and reconstruction Columns in +2.8m MSL					
	Demolition	1.50	m ³	11,050.00	16,575.00	
	Concrete	1.50	m ³	19,000.00	28,500.00	
	Formwork	12.00	m ²	1,800.00	21,600.00	
	Reinforcement	150.00	kg	225.00	33,750.00	100,425.00
3	Demolish and reconstruction of Exsisting brick Retaining wall and construction of Concrete Retaining wall in +2.8m MSL					
	Demolition	110.00	m ²	719.89	79,188.02	
	Concrete	45.00	m ³	19,000.00	855,000.00	
	Formwork	220.00	m ²	1,800.00	396,000.00	
	Reinforcement	4050.00	kg	225.00	911,250.00	2,241,438.02
4	Fixing 10mm rebars with using HILTI RE-500	630	nr	650.00	409,500.00	409,500.00
5	Demolish and reconstruction of brick wall and toilet Area in +5.8m MSL					
	Demolition	150.00	m ²	719.89	107,983.66	
	150m Block wall	150.00	m ²	2,171.91	325,787.14	
	plaster	300.00	m ²	935.00	280,500.00	
	painting	300.00	m ²	650.00	195,000.00	
	Toilet	3	nr	175,000.00	525,000.00	1,434,270.79
						7,675,733.81

Annexure VI

Project		SOIL INVESTIGATION AT THE LEFT BANK SIDE OF WELLAWATTA CANAL					Borehole No		BH_01								
Client		Edward and Christie Company					Sheet		1 of 1								
Location		Wellawatta	Rig	YWE-D45	Rock Level	m	G.W.L.	0.05	m								
Date of Started		20/3/2017	Method	WB	Terminated Level	10.00	Coordinates		**								
Date of Finished		20/3/2017	Weather	Hot	Elevation (MSL)	**											
Depth (m)	Soil Cond	Soil NO	Soil Type	Ground water Level	Legend	Soil Description	Depth	Field Records (SPT)/(CR,RQD)				Logged By : Samitha					
								15cm	15cm	15cm	Σ	Supervised Samitha					
								Penetration Resistance									
								5	10	15	20	25	30	35	40	50	
0.00						Ground level											
0.50	X	S1	Bulk				0 to 0.45										
0.50	X	S2	SPT			Loose, Light brown fine to medium SAND (Fill)	0.5 to 0.95										
1.00							1.5										
1.50	X	S3	SPT			Loose, Dark Grey CLAYEY SAND	1.5 to 2.95	1	1	1	2						
2.00							2.5										
2.50	X	S4	SPT			Loose, Light Grey medium to coarse CLAYEY SAND	2.5 to 2.95	6	3	6	9						
3.00							3.5										
3.50	X	S5	SPT				3.5 to 3.95	48									
4.00								10cm HB									
4.50	X	S6	SPT			Very dense, Grey SILTY SAND	4.5 to 4.95	45									
5.00								9cm HB									
5.50	X	S7	SPT			Whitish Grey SANDY CLAY	5.5 to 5.95	4	2	3	5						
6.00																	
6.50	X	S8	SPT			Very stiff, Whitish Grey medium to coarse SANDY CLAY	6.5 to 7.95	2	7	10	17						
7.00																	
7.50	X	S9	SPT			Very dense, Whitish Grey medium to coarse CLAYEY SAND	7.5 to 7.95	14	17	17	34						
8.00																	
8.50	X	S9	SPT			Terminated at 10m	8.5 to 10	14	17	17	34						
9.00																	
9.50																	
10.00																	

Sample Key

SPT - Standard Penetration Test

WB-Wash Sample

UD- Undisturbed Sample

GWL - Ground Water Level observed inside the

Borehole, after the saturation

NR - Not Recovered

HB - Hammer Bounce

FD - Free Drop

GEOTECH (PVT) LIMITED

13/1, PEPILIYANA
MAWATHA, KOHUWALA,
NUGEGODA, SRILANKA

Test Key

D - Disturbed Sample

SS - SPT Sample

W - Water Sample

Cr - Core Recovery (%)

RQD - Rock Quality Designation (%)

Remarks - Existing ground level considered as the zero level

JOB NO :- G/5044

Checked By

(%)	
ation (%)	By
considered as the zero	Kern
G/50-44	

Project		SOIL INVESTIGATION AT THE LEFT BANK SIDE OF WELLAWATTA CANAL				Borehole No		BH_03	
Client		Edward and Christie Company				Sheet		1 of 1	
Location	Wellawatta	Rig	YWE-D45	Rock Level	m	G.W.L	0.05	m	
Date of Started	20/3/2017	Method	WB	Terminated Level	0.70	Coordinates		**	
Date of Finished	20/3/2017	Weather	Hot	Elevation (MSL)	**	Coordinates		**	

Depth (m)	So. Cond	So. NO	So. Type	Ground water Level	Legend	Soil Description	Depth	Field Records (SPT)/(CR,RQD)				Penetration Resistance							
								15cm	15cm	15cm	N	5	10	15	20	25	30	35	40
0.00						Ground level													
0.00	X	S1	Bulk			Dark Brown SANDY CLAY with Debris	0 to 0.45												
						Terminated at 0.7m	0.7												
1.00																			
2.00																			
3.00																			
4.00																			
5.00																			
6.00																			
7.00																			
8.00																			
9.00																			
10.00																			

SPT - Standard Penetration Test WS - Wash Sample UID - Undisturbed Sample GWL - Ground Water Level observed inside the Borehole, after the saturation NR - Not Recovered HBI - Hammer Bounce FD - Free Drop		 GEOTECH (PVT) LIMITED 13/1, PEPILIYANA MAWATHA, KOHUWALA, NUGEGODA, SRILANKA		DT - Disturbed Sample SS - SPT Sample W - Water Sample Cr - Core Recovery (%) RQD - Rock Quality Designation (%) Remarks - Existing ground level considered as the zero level JOB NO - G/5044		Checked By - J Drilled By - Kuma	
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Annexure VII

Steel Sheet Pile

Steel Grades

Standard Designation: NF 35 520 (FRANCE) Circulaire 590 2 (BELGIUM)						Other steel grades comparable properties been supplied in compliance with the specifications and recommendations of the following standards					
Grade	Minimum yield point N/mm ²	Minimum tensile strength N/mm ²	Minimum elongation Lo-5.65 So %	Bend test 180° e < 16mm/ e > 16mm		Euronorm	USA	Canada	G.B.	G.F.R.	Japan
						25 - 72	ASTM	CSA G. 4021	BS 4360	EAU	A. 5528
E 240 SP	240	390	28	1e	1.5e	Fe 360 B				SiSo 37	
PAE 250	250	390	26		1e						
PAE 270	270	420	25		1.5e	Fe 430 B	A 328	Gr.260 W	Gr.43 A	SiSo 45	
E 270 SP	270	440	25	1e	1.5e						
PAE 300	300	450	23		2e	Fe 490 B	A 572 Gr.42	Gr.300 W			SY 295
PAE 320	320	470	23		2e						
E 320 SP	320	490	23	1.5e	2e	Fe 510 B	A 572 Gr.50	Gr.350 W	Gr.50 A	SiSo S	
PAE 360 2)	360	490	22		2.5e						
E 360 SP	360	510	22	2e	3e		A 572 Gr.55				SY 390
PAE 390 2)	390	510	20		2.5e						
E 390 SP*	390	550	20	2e	3e		A 572 Gr.60	Gr.400 W			
PAE 420 2)	420	520	19		3e						
E 430 SP*	430	570	17	3e	4e						
MERCOR 360	360	490	22		2.5e						

* These grades may contain dispersoid elements within the following ranges:
Mo - 0.01-0.06%, V - 0.02-0.10%

1) If all other conditions are respected a tensile strength of 20 N/mm² lower than the minimum indicated is acceptable.
Addition of copper: on request

2) For qualities with a guaranteed minimum yield point equal or higher than 360 N/mm² the carbon content in the heat analysis is limited to 0.22% and the manganese content to 1.6%.

Steel Qualities

	Ultimate Stress		Minimum Yield Stress				Minimum Elongation on 200mm	
			Up to and including 16mm thick		Over 16mm up to and including 40mm thick		Up to and including 9mm thick	Over 9mm thick
	kg/mm ²	N/mm ²	kg/mm ²	N/mm ²	kg/mm ²	N/mm ²	%	%
BS.4360: 1986 (Mild Steel) Grade 43A	43.8/59.1	430/580	28.0	275	27.0	265	16	20
BS.4360: 1986 (High yield Steel) Grade 50A	49.9/65.2	490/640	36.2	355	35.2	345	15	18
Similar to ASTM - A328	48.2 min.	482.6 min.	27.0	265	27.0	265	-	17

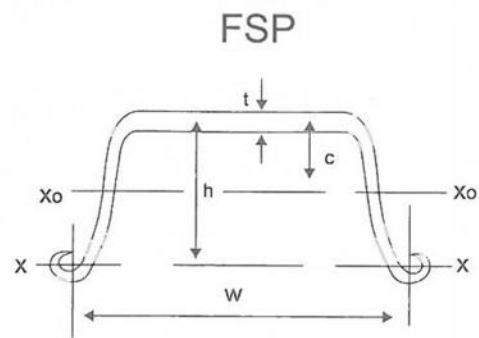
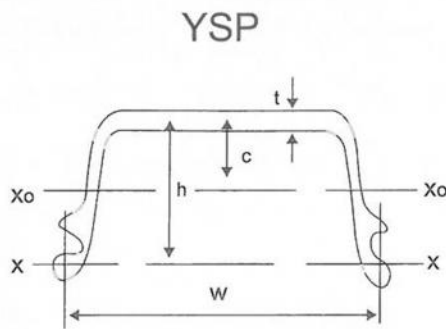
Notes:
COPPER BEARING STEEL - all the grades of steel are available with 0.20% up to 0.35% or over 0.35% up to 0.50% Copper content.
1 N/mm² = 0.101972 kg/mm²

Recommended working stresses for Steel Sheet Piling

Class of Work	BS.4360: 1986 Grade 43A ASTM A328 (Mild Steel)		BS.4360: 1986 Grade 50A (High yield Steel)	
	kg/mm ²	N/mm ²	kg/mm ²	N/mm ²
Permanent	14.2	140	18.3	180

Notes:
For guidance only.
Stresses in temporary conditions occurring during construction may be varied at the discretion of the client's engineer.

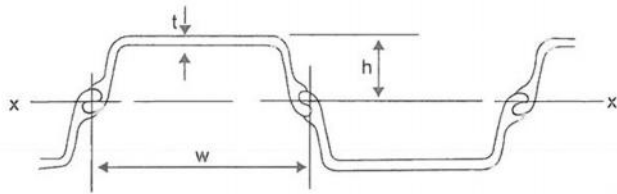
Steel Sheet Piles - U Type



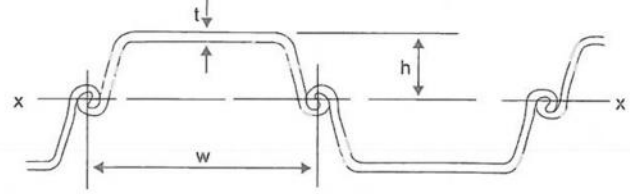
Designation	Dimensions						Sectional Area				Surface Area			
	W		h		t		per pile		per wall width		per pile		per wall width	
	mm	in	mm	in	mm	in	cm ²	in ²	cm ² /m	in ² /ft	m ² /m	ft ² /ft	m ² /m ²	ft ² /ft ²
YSP I	400	15.7	75	2.95	8.0	0.315	46.49	7.206	116.2	4.589	1.15	3.77	1.44	1.44
YSP U-5	400	15.7	80	3.15	7.6	0.299	45.21	7.008	113.0	5.338	1.17	3.84	1.47	1.47
FSP IA	400	15.7	85	3.35	8.0	0.315	45.21	7.008	113.0	5.338	1.21	3.97	1.51	1.51
YSP II	400	15.7	100	3.94	10.5	0.413	61.18	9.483	153.0	7.228	1.24	4.07	1.55	1.55
FSP II	400	15.7	100	3.94	10.5	0.413	61.18	9.483	153.0	7.228	1.33	4.36	1.66	1.66
YSP U-9	400	15.7	110	4.33	9.3	0.366	55.01	8.527	137.5	6.496	1.29	4.23	1.61	1.61
FSP IIA	400	15.7	120	4.72	9.2	0.362	55.01	8.527	137.5	6.496	1.34	4.40	1.68	1.68
YSP III	400	15.7	125	4.92	13.0	0.512	76.42	11.85	191.0	9.022	1.33	4.36	1.66	1.66
FSP III	400	15.7	125	4.92	13.0	0.512	76.42	11.85	191.0	9.022	1.44	4.72	1.80	1.80
YSP U-15	400	15.7	150	5.91	12.2	0.480	74.40	11.53	186.0	8.788	1.43	4.69	1.78	1.78
FSP IIIA	400	15.7	150	5.91	13.1	0.516	74.40	11.53	186.0	8.788	1.44	4.72	1.80	1.80
YSP IV	400	15.7	155	6.10	15.5	0.610	96.99	15.03	242.5	11.46	1.47	4.82	1.84	1.84
FSP IV	400	15.7	170	6.69	15.5	0.610	96.99	15.03	242.5	11.46	1.61	5.28	2.01	2.01
YSP U-23	400	15.7	175	6.89	14.7	0.579	94.21	14.60	235.5	11.12	1.56	5.12	1.94	1.94
FSP IVA	400	15.7	185	7.28	16.1	0.634	94.21	14.60	235.1	11.11	1.57	5.15	1.96	1.96
YSP V	420	16.5	175	6.89	22.0	0.866	134.0	20.77	319.0	15.07	1.59	5.22	1.99	1.99
FSP VL	500	19.7	200	7.87	24.3	0.957	133.8	20.74	267.6	12.64	1.75	5.74	1.75	1.75
FSP VIL	500	19.7	225	8.86	27.6	1.09	153.0	23.72	306.0	14.46	1.83	6.00	1.83	1.83

Steel Sheet Piles - U Type

YSP



FSP

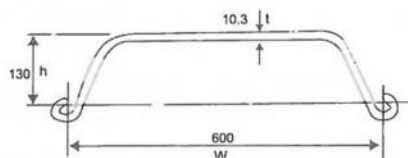


Centre		weight				Moment of Inertia				Radius of Gyration		Section Modulus			
C		per pile		per wall width		per pile		per wall width		per pile		per pile		per wall width	
cm	in	kg/m	lbs/ft	kg/m ²	lbs/ft ²	cm ⁴	in	cm ⁴ /m	in ⁴ /ft	cm	in	cm ³	in ³	cm ³ /m	in ³ /ft
2.64	1.04	36.5	24.5	91.2	18.7	429	10.3	3,820	28.0	3.04	1.20	66.4	4.05	509	9.47
2.78	1.09	35.5	23.9	88.8	18.2	454	10.9	4,220	30.9	3.17	1.25	64.7	3.95	527	9.80
3.45	1.36	35.5	23.9	88.8	18.2	598	14.4	4,500	33.0	3.64	1.43	88.0	5.37	529	9.84
3.62	1.43	48.0	32.3	120	24.6	986	23.7	8,690	63.6	4.01	1.58	121	7.38	869	16.2
4.04	1.59	48.0	32.3	120	24.6	1,240	29.8	8,740	64.0	4.50	1.77	152	9.28	874	16.3
3.86	1.52	43.2	29.0	108	22.1	1,070	25.7	9,680	70.9	4.42	1.74	120	7.32	880	16.4
4.72	1.86	43.2	29.0	108	22.1	1,460	35.1	10,600	77.6	5.15	2.03	160	9.76	880	16.4
4.72	1.86	60.0	40.3	150	30.7	1,920	46.1	16,400	120	5.01	1.97	196	12.0	1,310	24.4
4.90	1.93	60.0	40.3	150	30.7	2,220	53.3	16,800	123	5.39	2.12	223	13.6	1,340	24.9
5.71	2.25	58.4	39.2	146	29.9	2,700	64.9	22,800	167	5.12	2.02	238	14.5	1,520	28.3
5.84	2.30	58.4	39.2	146	29.9	2,790	67.0	22,800	167	6.12	2.41	250	15.3	1,520	28.3
5.85	2.30	76.0	51.1	190	38.9	2,690	88.7	31,900	234	6.15	2.42	311	19.0	2,060	38.3
6.45	2.54	76.1	51.1	190	38.9	4,670	112	38,600	283	6.94	2.73	362	22.1	2,270	42.2
6.51	2.56	74.0	49.7	185	37.9	4,380	105	39,400	389	6.81	2.68	330	20.1	2,250	41.8
7.45	2.93	74.0	49.7	185	37.9	5,300	127	41,600	305	7.50	2.95	400	24.4	2,250	41.8
6.15	2.42	105	70.6	250	51.2	5,950	143	55,200	404	6.67	2.63	433	26.4	3,150	58.6
6.94	2.73	105	70.6	210	43.0	7,960	191	63,000	461	7.71	3.04	520	31.7	3,150	58.6
8.09	3.18	120	80.6	240	49.2	11,400	271	86,000	630	8.63	3.40	680	41.5	3,820	71.1

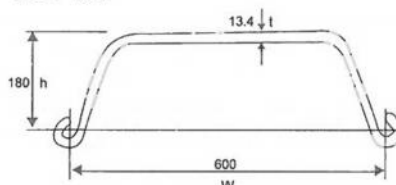
Steel Sheet Piles

Other available ranges :

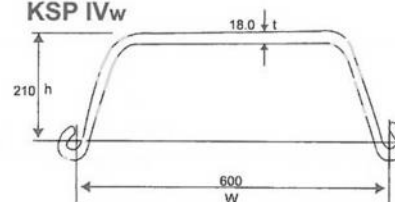
KSP II_w



KSP III_w



KSP IV_w



Section	Dimensions			Section Area		Unit weight		Moment of inertia		Modulus of section	
	W	h	t	A	A/m	M	M/m	I	I/m	Z _y	Z _y /m
	mm	mm	mm	cm ²	cm ² /m	kg/m	kg/m ²	cm ⁴	cm ⁴ /m	cm ³	cm ³ /m
KSP II _w	600	130	10.3	78.70	131.2	61.8	103	2,110	13,000	203	1,000
KSP III _w	600	180	13.4	103.9	173.2	81.6	136	6,220	32,400	376	1,800
KSP IV _w	600	210	18.0	135.3	225.5	106	177	8,630	56,700	539	2,700

Note : The sizes are also available in FSP sheet piles.
Sectional properties given per single pile, and per linear metre wall.

H T V B

Steel Sheet Pile

1) Grades Of Steel

Chemical composition and mechanical properties

Notation	Chemical Composition (%)									Tensile Strength (N/mm ²)	Yield Point (N/mm ²)	Elongation (%)	
	O	Si	Mn	P	S	Cu	Ni	O	Nb +V			Test Piece	
SY295	-	-	-	0.040 max.	0.040 max.	0.25 min.	-	-	-	490 min	294 min	JIS No. 1A	17 min
SY390	-	-	-	0.040 max.	0.040 max.	0.25 min.	-	-	-	539 min	392 min	JIS No. 1A	15 min
CR4C490	0.20 max.	0.55 max.	1.20 max.	0.070 - 0.15	0.040 max.	0.20 min.	0.65 max.	0.30 - 0.80	0.15 max.	JIS No. 1A	17 min		

Note: * Mo, Ti and other element added as necessary

2) Dimensional Tolerances

JIS A 5528 : 2000		
HOT ROLLED STEEL SHEET PILE "U" SHAPE		
Dimension		Tolerance
Height		±4%
Width (B)		+ 10 - 5
Thickness	t < 10	± 1.0
	10 ≤ t < 16	± 1.2
	t ≥ 16	± 1.5
Length (L)		+ Not Specified 0
Deflection	L ≤ 10m.	Full Length (M) x 0.12% max.
	L > 10m.	Full Length - 10m. x 0.10% + 12mm. max
Camber	L ≤ 10m.	Full Length (M) x 0.25% max.
	L ≥ 10m.	Full Length - 10m. x 0.20% + 20mm. max
Difference in vertically Cut Sections.		Within 4% of Width

Note: The deflection shall be in the direction parallel to the sheet pile wall and the camber shall be in the direction vertical to the sheet pile wall.

Annexure VIII

BRITISH STANDARD

**BS 4 : Part 1 :
1993**

Structural steel sections

Part 1. Specification for hot-rolled sections

Profilés en acier de construction
Partie 1. Spécification des profilés laminés à
chaud

Profile aus Baustahl
Teil 1. Spezifikation für warmgewaltzte Profile

UDC 669.14.018.29-423-122 : 693.814.1

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BS 4 : Part 1 : 1993

Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Iron and Steel Standards Policy Committee (ISM/-) to Technical Committee ISM/13, upon which the following bodies were represented:

BEAMA Ltd.
British Constructional Steelwork Association Ltd.
British Steel Industry
Health and Safety Executive
National Association of Steel Stockholders

This British Standard, having been prepared under the direction of the Iron and Steel Standards Policy Committee, was published under the authority of the Standards Board and comes into effect on 15 December 1993

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First published November 1903
Second edition August 1921
Third edition March 1932
Fourth edition February 1962
Fifth edition January 1972
Sixth edition November 1980
Seventh edition December 1993

The following BSI references relate to the work on this standard:
Committee reference ISM/13
Draft for comment 92/45671 DC

ISBN 0 580 21637 3

Amendments issued since publication

Amd. No.	Date	Text affected

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Foreword

This Part of BS 4 has been prepared under the direction of the Iron and Steel Standards Policy Committee and is a revision of BS 4 : Part 1 : 1980 which is withdrawn.

The major reason for this revision is to delete the requirements for tolerances on beams, columns and bearing piles which are now contained in BS EN 10034, published simultaneously with this standard. The opportunity has also been taken to delete derived properties from the tables in the previous edition of this standard.

The dimensions and masses of beams, columns and bearing piles have also been amended slightly to correct anomalies in tables 5 to 7 of BS 4 : Part 1 : 1980.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

1 Scope

This Part of BS 4 specifies the dimensions and mass of the following hot-rolled structural steel sections:

- universal beams
- universal columns
- universal bearing piles
- joists
- structural tees cut from universal beams
- structural tees cut from universal columns
- channels

Mass, length and depth tolerances for taper flange joists and channels are also given.

NOTE. Dimensional, mass, length and straightness tolerances for universal beams, columns and bearing piles are given in BS EN 10034.

2 Normative reference

This Part of BS 4 incorporates, by reference, provisions from another publication. The normative reference is cited at the appropriate point in the text and the publication is given on the inside back cover. Subsequent amendments to, or revisions of, this publication apply to this Part of BS 4 only when incorporated in it by updating or revision.

3 Designation

A section shall be designated by the serial size in millimetres (mm) and the mass per unit length in kilograms per metre (kg/m).

This form of designation shall be used in any enquiry and order.

Example

a beam of 920.5 mm × 420.5 mm at 388 kg/m is designated 914 × 419 × 388 beam.

4 Tolerances on universal beams, columns and bearing piles

Dimensional, mass, length and straightness tolerances on universal beams, columns and bearing piles shall be as given in BS EN 10034.

5 Mass and length tolerances for joists and channels

5.1 Mass

Tolerances shall be applied to the actual mass per unit length of the joist or channel.

The rolling tolerances shall be either :

- a) ± 2.5 % of the actual mass per unit length; or
- b) $+ 5$ % of the actual mass per unit length where a minimum mass per unit length is ordered.

5.2 Length

Joists or channels ordered as 'specified' or as 'exact' lengths shall be supplied as follows.

- a) *Specified lengths*. When a joist or channel is to be cut to a specified length, it shall be cut to within ± 25 mm of that length. When a minimum length is specified, it shall be cut to within $^{+50}_0$ mm of that minimum length.
- b) *Exact lengths*. When a section is to be cut to an exact length, it shall be cold sawn to within ± 3.2 mm of that length.

6 Tolerance on depth of section for joists and channels

The tolerance on the nominal depth of section shall be as given in table 1.

Table 1. Tolerances on nominal depth of joists and channels

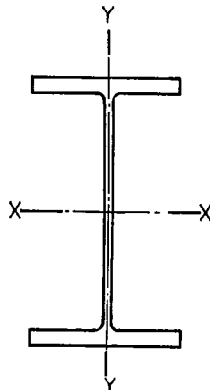
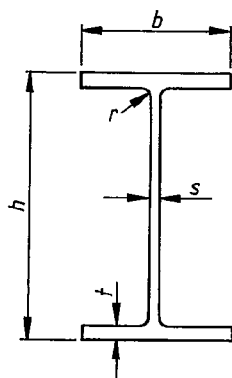
Nominal depth		Maximum permissible variation from specified depth	
Over	Up to and including		
mm	mm	mm	mm
—	305	+ 3.2	— 0.8
305	381	+ 4.0	— 1.6
381	432	+ 4.8	— 1.6

7 Dimensions of hot-rolled structural steel sections

The nominal dimensions of hot-rolled structural steel sections shall be as given in tables 2 to 8.

BS 4 : Part 1 : 1993

Table 2. Universal beams



Section designation	Mass per unit length kg/m	Depth h mm	Width b mm	Web s mm	Flange t mm	Root radius r mm
914 × 419 × 388	388.0	921.0	420.5	21.4	36.6	24.1
914 × 419 × 343	343.3	911.8	418.5	19.4	32.0	24.1
914 × 305 × 289	289.1	926.6	307.7	19.5	32.0	19.1
914 × 305 × 253	253.4	918.4	305.5	17.3	27.9	19.1
914 × 305 × 224	224.2	910.4	304.1	15.9	23.9	19.1
914 × 305 × 201	200.9	903.0	303.3	15.1	20.2	19.1
838 × 292 × 226	226.5	850.9	293.8	16.1	26.8	17.8
838 × 292 × 194	193.8	840.7	292.4	14.7	21.7	17.8
838 × 292 × 176	175.9	834.9	291.7	14.0	18.8	17.8
762 × 267 × 197	196.8	769.8	268.0	15.6	25.4	16.5
762 × 267 × 173	173.0	762.2	266.7	14.3	21.6	16.5
762 × 267 × 147	146.9	754.0	265.2	12.8	17.5	16.5
762 × 267 × 134	133.9	750.0	264.4	12.0	15.5	16.5
686 × 254 × 170	170.2	692.9	255.8	14.5	23.7	15.2
686 × 254 × 152	152.4	687.5	254.5	13.2	21.0	15.2
686 × 254 × 140	140.1	683.5	253.7	12.4	19.0	15.2
686 × 254 × 125	125.2	677.9	253.0	11.7	16.2	15.2
610 × 305 × 238	238.1	635.8	311.4	18.4	31.4	16.5
610 × 305 × 179	179.0	620.2	307.1	14.1	23.6	16.5
610 × 305 × 149	149.1	612.4	304.8	11.8	19.7	16.5
610 × 229 × 140	139.9	617.2	230.2	13.1	22.1	12.7
610 × 229 × 125	125.1	612.2	229.0	11.9	19.6	12.7
610 × 229 × 113	113.0	607.6	228.2	11.1	17.3	12.7
610 × 229 × 101	101.2	602.6	227.6	10.5	14.8	12.7

Table 2. Universal beams (*continued*)

Section designation	Mass per unit length kg/m	Depth <i>h</i> mm	Width <i>b</i> mm	Web <i>s</i> mm	Flange <i>t</i> mm	Root radius <i>r</i> mm
533 × 210 × 122	122.0	544.5	211.9	12.7	21.3	12.7
533 × 210 × 109	109.0	539.5	210.8	11.6	18.8	12.7
533 × 210 × 101	101.0	536.7	210.0	10.8	17.4	12.7
533 × 210 × 92	92.1	533.1	209.3	10.1	15.6	12.7
533 × 210 × 82	82.2	528.3	208.8	9.6	13.2	12.7
457 × 191 × 98	98.3	467.2	192.8	11.4	19.6	10.2
457 × 191 × 89	89.3	463.4	191.9	10.5	17.7	10.2
457 × 191 × 82	82.0	460.0	191.3	9.9	16.0	10.2
457 × 191 × 74	74.3	457.0	190.4	9.0	14.5	10.2
457 × 191 × 67	67.1	453.4	189.9	8.5	12.7	10.2
457 × 152 × 82	82.1	465.8	155.3	10.5	18.9	10.2
457 × 152 × 74	74.2	462.0	154.4	9.6	17.0	10.2
457 × 152 × 67	67.2	458.0	153.8	9.0	15.0	10.2
457 × 152 × 60	59.8	454.6	152.9	8.1	13.3	10.2
457 × 152 × 52	52.3	449.8	152.4	7.6	10.9	10.2
406 × 178 × 74	74.2	412.8	179.5	9.5	16.0	10.2
406 × 178 × 67	67.1	409.4	178.8	8.8	14.3	10.2
406 × 178 × 60	60.1	406.4	177.9	7.9	12.8	10.2
406 × 178 × 54	54.1	402.6	177.7	7.7	10.9	10.2
406 × 140 × 46	46.0	403.2	142.2	6.8	11.2	10.2
406 × 140 × 39	39.0	398.0	141.8	6.4	8.6	10.2
356 × 171 × 67	67.1	363.4	173.2	9.1	15.7	10.2
356 × 171 × 57	57.0	358.0	172.2	8.1	13.0	10.2
356 × 171 × 51	51.0	355.0	171.5	7.4	11.5	10.2
356 × 171 × 45	45.0	351.4	171.1	7.0	9.7	10.2
356 × 127 × 39	39.1	353.4	126.0	6.6	10.7	10.2
356 × 127 × 33	33.1	349.0	125.4	6.0	8.5	10.2
305 × 165 × 54	54.0	310.4	166.9	7.9	13.7	8.9
305 × 165 × 46	46.1	306.6	165.7	6.7	11.8	8.9
305 × 165 × 40	40.3	303.4	165.0	6.0	10.2	8.9
305 × 127 × 48	48.1	311.0	125.3	9.0	14.0	8.9
305 × 127 × 42	41.9	307.2	124.3	8.0	12.1	8.9
305 × 127 × 37	37.0	304.4	123.4	7.1	10.7	8.9

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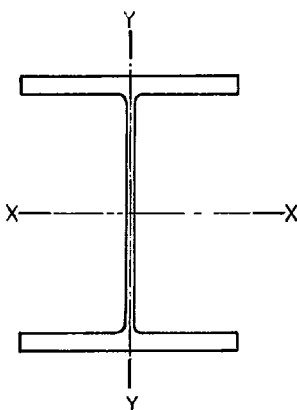
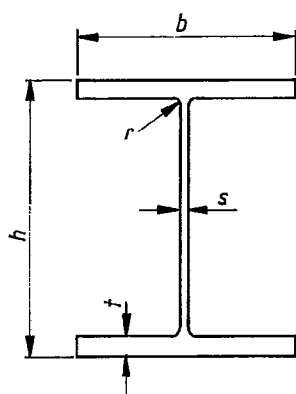
Table 2. Universal beams (*concluded*)

Section designation	Mass per unit length kg/m	Depth <i>h</i> mm	Width <i>b</i> mm	Web <i>s</i> mm	Flange <i>t</i> mm	Root radius <i>r</i> mm
305 × 102 × 33	32.8	312.7	102.4	6.6	10.8	7.6
305 × 102 × 28	28.2	308.7	101.8	6.0	8.8	7.6
305 × 102 × 25	24.8	305.1	101.6	5.8	7.0	7.6
254 × 146 × 43	43.0	259.6	147.3	7.2	12.7	7.6
254 × 146 × 37	37.0	256.0	146.4	6.3	10.9	7.6
254 × 146 × 31	31.1	251.4	146.1	6.0	8.6	7.6
254 × 102 × 28	28.3	260.4	102.2	6.3	10.0	7.6
254 × 102 × 25	25.2	257.2	101.9	6.0	8.4	7.6
254 × 102 × 22	22.0	254.0	101.6	5.7	6.8	7.6
203 × 133 × 30	30.0	206.8	133.9	6.4	9.6	7.6
203 × 133 × 25	25.1	203.2	133.2	5.7	7.8	7.6
203 × 102 × 23	23.1	203.2	101.8	5.4	9.3	7.6
178 × 102 × 19	19.0	177.8	101.2	4.8	7.9	7.6
152 × 89 × 16	16.0	152.4	88.7	4.5	7.7	7.6
127 × 76 × 13	13.0	127.0	76.0	4.0	7.6	7.6

NOTE 1. The nominal dimensions are based on the imperial values.

NOTE 2. The masses per metre run are derived from the cross-sectional area on the basis of 0.785 kg/cm² per metre run.

Table 3. Universal columns



Section designation	Mass per unit length kg/m	Depth h mm	Width b mm	Web s mm	Flange t mm	Root radius r mm
356 × 406 × 634	633.9	474.6	424.0	47.6	77.0	15.2
356 × 406 × 551	551.0	455.6	418.5	42.1	67.5	15.2
356 × 406 × 467	467.0	436.6	412.2	35.8	58.0	15.2
356 × 406 × 393	393.0	419.0	407.0	30.6	49.2	15.2
356 × 406 × 340	339.9	406.4	403.0	26.6	42.9	15.2
356 × 406 × 287	287.1	393.6	399.0	22.6	36.5	15.2
356 × 406 × 235	235.1	381.0	394.8	18.4	30.2	15.2
356 × 368 × 202	201.9	374.6	374.7	16.5	27.0	15.2
356 × 368 × 177	177.0	368.2	372.6	14.4	23.8	15.2
356 × 368 × 153	152.9	362.0	370.5	12.3	20.7	15.2
356 × 368 × 129	129.0	355.6	368.6	10.4	17.5	15.2
305 × 305 × 283	282.9	365.3	322.2	26.8	44.1	15.2
305 × 305 × 240	240.0	352.5	318.4	23.0	37.7	15.2
305 × 305 × 198	198.1	339.9	314.5	19.1	31.4	15.2
305 × 305 × 158	158.1	327.1	311.2	15.8	25.0	15.2
305 × 305 × 137	136.9	320.5	309.2	13.8	21.7	15.2
305 × 305 × 118	117.9	314.5	307.4	12.0	18.7	15.2
305 × 305 × 97	96.9	307.9	305.3	9.9	15.4	15.2
254 × 254 × 167	167.1	289.1	265.2	19.2	31.7	12.7
254 × 254 × 132	132.0	276.3	261.3	15.3	25.3	12.7
254 × 254 × 107	107.1	266.7	258.8	12.8	20.5	12.7
254 × 254 × 89	88.9	260.3	256.3	10.3	17.3	12.7
254 × 254 × 73	73.1	254.1	254.6	8.6	14.2	12.7

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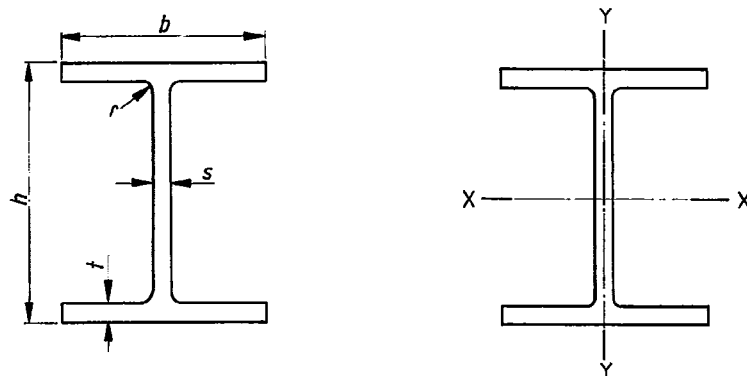
Table 3. Universal columns (*concluded*)

Section designation	Mass per unit length kg/m	Depth <i>h</i> mm	Width <i>b</i> mm	Web <i>s</i> mm	Flange <i>t</i> mm	Root radius <i>r</i> mm
203 × 203 × 86	86.1	222.2	209.1	12.7	20.5	10.2
203 × 203 × 71	71.0	215.8	206.4	10.0	17.3	10.2
203 × 203 × 60	60.0	209.6	205.8	9.4	14.2	10.2
203 × 203 × 52	52.0	206.2	204.3	7.9	12.5	10.2
203 × 203 × 46	46.1	203.2	203.6	7.2	11.0	10.2
152 × 152 × 37	37.0	161.8	154.4	8.0	11.5	7.6
152 × 152 × 30	30.0	157.6	152.9	6.5	9.4	7.6
152 × 152 × 23	23.0	152.4	152.2	5.8	6.8	7.6

NOTE 1. The nominal dimensions are based on the imperial values.

NOTE 2. The masses per metre run are derived from the cross-sectional area on the basis of 0.785 kg/cm² per metre run.

Table 4. Universal bearing piles



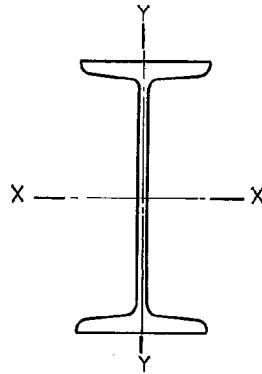
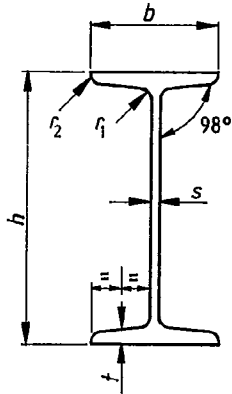
Section designation	Mass per unit length kg/m	Depth <i>h</i> mm	Width <i>b</i> mm	Web <i>s</i> mm	Flange <i>t</i> mm	Root radius <i>r</i> mm
356 × 368 × 174	173.9	361.4	378.5	20.3	20.4	15.2
356 × 368 × 152	152.0	356.4	376.0	17.8	17.9	15.2
356 × 368 × 133	133.0	352.0	373.8	15.6	15.7	15.2
356 × 368 × 109	108.9	346.4	371.0	12.8	12.9	15.2
305 × 305 × 223	222.9	337.9	325.7	30.3	30.4	15.2
305 × 305 × 186	186.0	328.3	320.9	25.5	25.6	15.2
305 × 305 × 149	149.1	318.5	316.0	20.6	20.7	15.2
305 × 305 × 126	126.1	312.3	312.9	17.5	17.6	15.2
305 × 305 × 110	110.0	307.9	310.7	15.3	15.4	15.2
305 × 305 × 95	94.9	303.7	308.7	13.3	13.3	15.2
305 × 305 × 88	88.0	301.7	307.8	12.4	12.3	15.2
305 × 305 × 79	78.9	299.3	306.4	11.0	11.1	15.2
254 × 254 × 85	85.1	254.3	260.4	14.4	14.3	12.7
254 × 254 × 71	71.0	249.7	258.0	12.0	12.0	12.7
254 × 254 × 63	63.0	247.1	256.6	10.6	10.7	12.7
203 × 203 × 54	53.9	204.0	207.7	11.3	11.4	10.2
203 × 203 × 45	44.9	200.2	205.9	9.5	9.5	10.2

NOTE 1. The nominal dimensions are based on the imperial values.

NOTE 2. The masses per metre run are derived from the cross-sectional area on the basis of 0.785 kg/cm² per metre run.

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Table 5. Joists with taper flanges

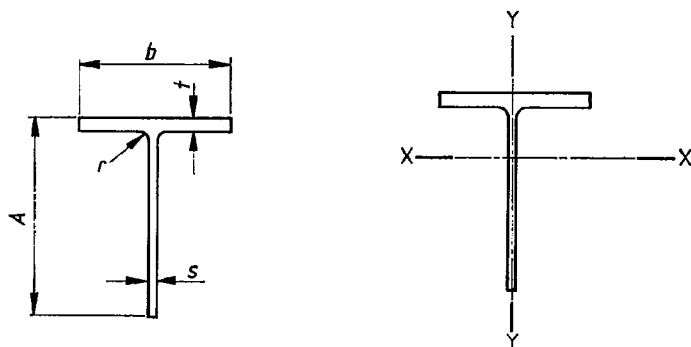


Section designation mm	Mass per unit length kg/m	Depth of section h mm	Width of section b mm	Thickness		Radius	
				Web s mm	Flange t mm	Root r ₁ mm	Toe r ₂ mm
254 × 203 × 82	82.0	254.0	203.2	10.2	19.9	19.6	9.7
254 × 114 × 37	37.2	254.0	114.3	7.6	12.8	12.4	6.1
203 × 152 × 52	52.3	203.2	152.4	8.9	16.5	15.5	7.6
152 × 127 × 37	37.3	152.4	127.0	10.4	13.2	13.5	6.6
127 × 114 × 29	29.3	127.0	114.3	10.2	11.5	9.9	4.8
127 × 114 × 27	26.9	127.0	114.3	7.4	11.4	9.9	5.0
127 × 76 × 16	16.5	127.0	76.2	5.6	9.6	9.4	4.6
114 × 114 × 27	26.9	114.3	114.3	9.5	10.7	14.2	3.2
102 × 102 × 23	23.0	101.6	101.6	9.5	10.3	11.1	3.2
102 × 44 × 7	7.5	101.6	44.5	4.3	6.1	6.9	3.3
89 × 89 × 19	19.5	88.9	88.9	9.5	9.9	11.1	3.2
76 × 76 × 15	15.0	76.2	80.0	8.9	8.4	9.4	4.6
76 × 76 × 13	12.8	76.2	76.2	5.1	8.4	9.4	4.6

NOTE 1. The nominal dimensions are based on the imperial values.

NOTE 2. The masses per metre run are derived from the cross-sectional area on the basis of 0.785 kg/cm² per metre run.

Table 6. Structural tees cut from universal beams



Section designation	Mass per unit length	Cut from universal beam (see table 2)	Width of section <i>b</i>	Depth of section <i>A</i>	Thickness		Root radius <i>r</i>
		Section designation			Web <i>s</i>	Flange <i>t</i>	
	kg/m	mm	mm	mm	mm	mm	mm
305 × 457 × 127	126.7	914 × 305 × 253	305.5	459.1	17.3	27.9	19.1
305 × 457 × 112	112.1	914 × 305 × 224	304.1	455.1	15.9	23.9	19.1
305 × 457 × 101	100.4	914 × 305 × 201	303.3	451.4	15.1	20.2	19.1
292 × 419 × 113	113.2	838 × 292 × 226	293.8	425.4	16.1	26.8	17.8
292 × 419 × 97	96.9	838 × 292 × 194	292.4	420.3	14.7	21.7	17.8
292 × 419 × 88	87.9	838 × 292 × 176	291.7	417.4	14.0	18.8	17.8
267 × 381 × 99	98.3	762 × 267 × 197	268.0	384.8	15.6	25.4	16.5
267 × 381 × 87	86.5	762 × 267 × 173	266.7	381.0	14.3	21.6	16.5
267 × 381 × 74	73.4	762 × 267 × 147	265.2	376.9	12.8	17.5	16.5
254 × 343 × 85	85.1	686 × 254 × 170	255.8	346.4	14.5	23.7	15.2
254 × 343 × 76	76.2	686 × 254 × 152	254.5	343.7	13.2	21.0	15.2
254 × 343 × 70	70.0	686 × 254 × 140	253.7	341.7	12.4	19.0	15.2
254 × 343 × 63	62.6	686 × 254 × 125	253.0	338.9	11.7	16.2	15.2
305 × 305 × 119	119.0	610 × 305 × 238	311.4	317.8	18.4	31.4	16.5
305 × 305 × 90	89.5	610 × 305 × 179	307.1	310.0	14.1	23.6	16.5
305 × 305 × 75	74.6	610 × 305 × 149	304.8	306.1	11.8	19.7	16.5
229 × 305 × 70	69.9	610 × 229 × 140	230.2	308.5	13.1	22.1	12.7
229 × 305 × 63	62.5	610 × 229 × 125	229.0	306.0	11.9	19.6	12.7
229 × 305 × 57	56.5	610 × 229 × 113	228.2	303.7	11.1	17.3	12.7
229 × 305 × 51	50.6	610 × 229 × 101	227.6	301.2	10.5	14.8	12.7

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Table 6. Structural tees cut from universal beams (*continued*)

Section designation	Mass per unit length kg/m	Cut from universal beam (see table 2)	Width of section <i>b</i> mm	Depth of section <i>A</i> mm	Thickness		Root radius <i>r</i>
		Section designation			Web <i>s</i> mm	Flange <i>t</i> mm	
210 × 267 × 61	61.0	533 × 210 × 122	211.9	272.2	12.7	21.3	12.7
210 × 267 × 55	54.5	533 × 210 × 109	210.8	269.7	11.6	18.8	12.7
210 × 267 × 51	50.5	533 × 210 × 101	210.0	268.3	10.8	17.4	12.7
210 × 267 × 46	46.0	533 × 210 × 92	209.3	266.5	10.1	15.6	12.7
210 × 267 × 41	41.1	533 × 210 × 82	208.8	264.1	9.6	13.2	12.7
191 × 229 × 49	49.1	457 × 191 × 98	192.8	233.5	11.4	19.6	10.2
191 × 229 × 45	44.6	457 × 191 × 89	191.9	231.6	10.5	17.7	10.2
191 × 229 × 41	41.0	457 × 191 × 82	191.3	229.9	9.9	16.0	10.2
191 × 229 × 37	37.1	457 × 191 × 74	190.4	228.4	9.0	14.5	10.2
191 × 229 × 34	33.5	457 × 191 × 67	189.9	226.6	8.5	12.7	10.2
152 × 229 × 41	41.0	457 × 152 × 82	155.3	232.8	10.5	18.9	10.2
152 × 229 × 37	37.1	457 × 152 × 74	154.4	230.9	9.6	17.0	10.2
152 × 229 × 34	33.6	457 × 152 × 67	153.8	228.9	9.0	15.0	10.2
152 × 229 × 30	29.9	457 × 152 × 60	152.9	227.2	8.1	13.3	10.2
152 × 229 × 26	26.1	457 × 152 × 52	152.4	224.8	7.6	10.9	10.2
178 × 203 × 37	37.1	406 × 178 × 74	179.5	206.3	9.5	16.0	10.2
178 × 203 × 34	33.5	406 × 178 × 67	178.8	204.6	8.8	14.3	10.2
178 × 203 × 30	30.0	406 × 178 × 60	177.9	203.1	7.9	12.8	10.2
178 × 203 × 27	27.0	406 × 178 × 54	177.7	201.2	7.7	10.9	10.2
140 × 203 × 23	23.0	406 × 140 × 46	142.2	201.5	6.8	11.2	10.2
140 × 203 × 20	19.5	406 × 140 × 39	141.8	198.9	6.4	8.6	10.2
171 × 178 × 34	33.5	356 × 171 × 67	173.2	181.6	9.1	15.7	10.2
171 × 178 × 29	28.5	356 × 171 × 57	172.2	178.9	8.1	13.0	10.2
171 × 178 × 26	25.5	356 × 171 × 51	171.5	177.4	7.4	11.5	10.2
171 × 178 × 23	22.5	356 × 171 × 45	171.1	175.6	7.0	9.7	10.2
127 × 178 × 20	19.5	356 × 127 × 39	126.0	176.6	6.6	10.7	10.2
127 × 178 × 17	16.5	356 × 127 × 33	125.4	174.4	6.0	8.5	10.2
165 × 152 × 27	27.0	305 × 165 × 54	166.9	155.1	7.9	13.7	8.9
165 × 152 × 23	23.0	305 × 165 × 46	165.7	153.2	6.7	11.8	8.9
165 × 152 × 20	20.1	305 × 165 × 40	165.0	151.6	6.0	10.2	8.9
127 × 152 × 24	24.0	305 × 127 × 48	125.3	155.4	9.0	14.0	8.9
127 × 152 × 21	20.9	305 × 127 × 42	124.3	153.5	8.0	12.1	8.9
127 × 152 × 19	18.5	305 × 127 × 37	123.4	152.1	7.1	10.7	8.9

Table 6. Structural tees cut from universal beams (concluded)

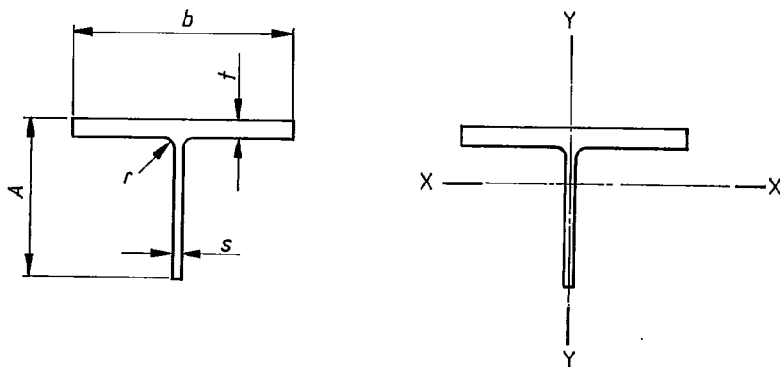
Section designation	Mass per unit length	Cut from universal beam (see table 2)	Width of section <i>b</i>	Depth of section <i>A</i>	Thickness		Root radius <i>r</i>
		Section designation			Web <i>s</i>	Flange <i>t</i>	
	kg/m	mm	mm	mm	mm	mm	mm
102 × 152 × 17	16.4	305 × 102 × 33	102.4	156.3	6.6	10.8	7.6
102 × 152 × 14	14.1	305 × 102 × 28	101.8	154.3	6.0	8.8	7.6
102 × 152 × 13	12.4	305 × 102 × 25	101.6	152.5	5.8	7.0	7.6
146 × 127 × 22	21.5	254 × 146 × 43	147.3	129.7	7.2	12.7	7.6
146 × 127 × 19	18.5	254 × 146 × 37	146.4	127.9	6.3	10.9	7.6
146 × 127 × 16	15.5	254 × 146 × 31	146.1	125.6	6.0	8.6	7.6
102 × 127 × 14	14.1	254 × 102 × 28	102.2	130.1	6.3	10.0	7.6
102 × 127 × 13	12.6	254 × 102 × 25	101.9	128.5	6.0	8.4	7.6
102 × 127 × 11	11.0	254 × 102 × 22	101.6	126.9	5.7	6.8	7.6
133 × 102 × 15	15.0	203 × 133 × 30	133.9	103.3	6.4	9.6	7.6
133 × 102 × 13	12.5	203 × 133 × 25	133.2	101.5	5.7	7.8	7.6

NOTE 1. The nominal dimensions are based on the imperial values.

NOTE 2. The masses per metre run are calculated from the cross-sectional area on the basis of 0.785 kg/cm² per metre run.

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Table 7. Structural tees cut from universal columns

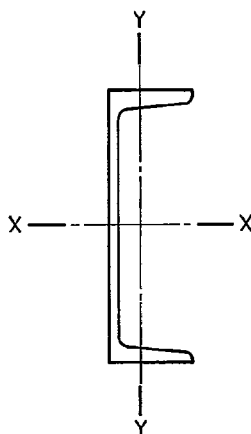
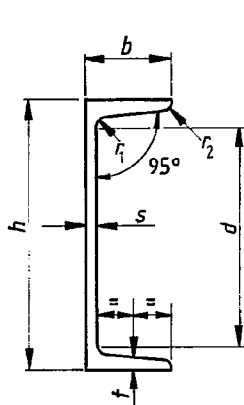


Section designation	Mass per unit length kg/m	Cut from universal beam (see table 2)	Width of section <i>b</i> mm	Depth of section <i>A</i> mm	Thickness		Root radius <i>r</i> mm
		Section designation mm			Web <i>s</i> mm	Flange <i>t</i> mm	
406 × 178 × 118	117.5	356 × 406 × 235	394.8	190.4	18.4	30.2	15.2
368 × 178 × 101	100.9	356 × 368 × 202	374.7	187.2	16.5	27.0	15.2
368 × 178 × 89	88.5	356 × 368 × 177	372.6	184.0	14.4	23.8	15.2
368 × 178 × 77	76.4	356 × 368 × 153	370.5	180.9	12.3	20.7	15.2
368 × 178 × 65	64.5	356 × 368 × 129	368.6	177.7	10.4	17.5	15.2
305 × 152 × 79	79.0	305 × 305 × 158	311.2	163.5	15.8	25.0	15.2
305 × 152 × 69	68.4	305 × 305 × 137	309.2	160.2	13.8	21.7	15.2
305 × 152 × 59	58.9	305 × 305 × 118	307.4	157.2	12.0	18.7	15.2
305 × 152 × 49	48.4	305 × 305 × 97	305.3	153.9	9.9	15.4	15.2
254 × 127 × 66	66.0	254 × 254 × 132	261.3	138.1	15.3	25.3	12.7
254 × 127 × 54	53.5	254 × 254 × 107	258.8	133.3	12.8	20.5	12.7
254 × 127 × 45	44.4	254 × 254 × 89	256.3	130.1	10.3	17.3	12.7
254 × 127 × 37	36.5	254 × 254 × 73	254.6	127.0	8.6	14.2	12.7
203 × 102 × 43	43.0	203 × 203 × 86	209.1	111.0	12.7	20.5	10.2
203 × 102 × 36	35.5	203 × 203 × 71	206.4	107.8	10.0	17.3	10.2
203 × 102 × 30	30.0	203 × 203 × 60	205.8	104.7	9.4	14.2	10.2
203 × 102 × 26	26.0	203 × 203 × 52	204.3	103.0	7.9	12.5	10.2
203 × 102 × 23	23.0	203 × 203 × 46	203.6	101.5	7.2	11.0	10.2
152 × 76 × 19	18.5	152 × 152 × 37	154.4	80.8	8.0	11.5	7.6
152 × 76 × 15	15.0	152 × 152 × 30	152.9	78.7	6.5	9.4	7.6
152 × 76 × 12	11.5	152 × 152 × 23	152.2	76.1	5.8	6.8	7.6

NOTE 1. The nominal dimensions are based on the imperial values.

NOTE 2. The masses per metre run are calculated from the cross-sectional area on the basis of 0.785 kg/cm² per metre run.

Table 8. Channels



Section designation	Mass per unit length	Depth of section <i>h</i>	Width of section <i>b</i>	Thickness		Radius		Depth between fillets <i>d</i>
				Web <i>s</i>	Flange <i>t</i>	Root <i>r</i> ₁	Toe <i>r</i> ₂	
	kg/m	mm	mm	mm	mm	mm	mm	mm
432 × 102 × 65	65.5	431.8	101.6	12.2	16.8	15.2	4.8	362.4
381 × 102 × 55	55.0	381.0	101.6	10.4	16.3	15.2	4.8	312.4
305 × 102 × 46	46.2	304.8	101.6	10.2	14.8	15.2	4.8	239.2
305 × 89 × 42	41.8	304.8	88.9	10.2	13.7	13.7	3.2	245.4
254 × 89 × 36	35.7	254.0	88.9	9.1	13.6	13.7	3.2	194.7
254 × 76 × 28	28.2	254.0	76.2	8.1	10.9	12.2	3.2	203.8
229 × 89 × 33	32.7	228.6	88.9	8.6	13.3	13.7	3.2	169.8
229 × 76 × 26	26.1	228.6	76.2	7.6	11.2	12.2	3.2	178.0
203 × 89 × 30	29.8	203.2	88.9	8.1	12.9	13.7	3.2	145.2
203 × 76 × 24	23.9	203.2	76.2	7.1	11.2	12.2	3.2	152.5
178 × 89 × 27	26.8	177.8	88.9	7.6	12.3	13.7	3.2	121.0
178 × 76 × 21	20.8	177.8	76.2	6.6	10.3	12.2	3.2	128.8
152 × 89 × 24	23.9	152.4	88.9	7.1	11.6	13.7	3.2	97.0
152 × 76 × 18	17.9	152.4	76.2	6.4	9.0	12.2	2.4	105.9
127 × 64 × 15	14.9	127.0	63.5	6.4	9.2	10.7	2.4	84.0
102 × 51 × 10	10.4	101.6	50.8	6.1	7.6	9.1	2.4	65.7
76 × 38 × 7	6.7	76.2	38.1	5.1	6.8	7.6	2.4	45.8

NOTE 1. The nominal dimensions are based on the imperial values.

NOTE 2. The masses per metre run are calculated from the cross-sectional area on the basis of 0.785 kg/cm² per metre run.

List of references (see clause 2)

Normative reference

BS EN 10034 : 1993

Structural steel I and H sections — Tolerances on shape and dimensions

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