PHUENTSHOLING THROMDE

REVISION OF DETAILED PROJECT REPORT FOR OM CHHU

June 2023

APECS CONSULTANCY

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The Om Chhu river that translates as ‘Duttey Khola’ translates to ‘Milky river’ and derives its name due to the heavy sedimentation from glacial flour.

The Om Chhu river that is unusually calm and silent during winter months, turns to a thunderous and violent river during summers.

It runs from the center of Phuentsholing Thromde, the gateway town for Bhutan’s trade.

The recurring measures of isolated intervention has hardly contributed to making the flood protection system resilient, whilst being highly unsustainable.

A pragmatic and robust infrastructure planning ensures green, resilient and inclusive recovery for Phuentsholing Thromde for decades to come.

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Comments and Actions

None

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Acronyms & Abbreviations

ADB	Asian Development Bank
AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ARI	Average Recurrence Interval
BHRM	Bhutan Resident Mission
BoQ	Bill of Quantities
CW	Civil Works
DEM	Digital Elevation Model
EIA	Environmental Impact Assessment
FEWS	Flood Early Warning System
FMP	Flood Management Plan
GIS	Geographic Information System
HEC-HMS	The Hydrologic Engineering Center's Hydrologic Modeling System
HEC-RAS	The Hydrologic Engineering Center's River Analysis System
ICIMOD	International Centre for Integrated Mountain Development
MASL	Meter Above Sea Level
MOWHS	Ministry of Works and Human Settlements
NCHM	National Center for Hydrology and Meteorology
Nu	Ngultrum
PIC	Project Implementation Consultant for Phuentsholing Township Development Project
PLT	Plate Load Test
PPT	Portable Penetrometer Test
PT	Phuentsholing Thromde (Municipality Corporation)
PTDP	Phuentsholing Township Development Project
RGOB	Royal Government of Bhutan
RTK GNSS	Real time kinematics - Global navigation satellite system
SPT	Standard Penetration Test
SRT	Seismic Refraction Tomography
ToR	Terms of Reference
UNFCCC	United Nations Framework Convention on Climate Change

Weights And Measures

ac	acre
ha	hectare
HP	Horsepower
km	kilometer
lpcd	liters per capita per day
lps	liter per second
m	Meter
kN	Kilonewton
MASL	metres above sea level

Conversion: 1 USD= Nu. 80

OVERALL PROJECT LOCATION

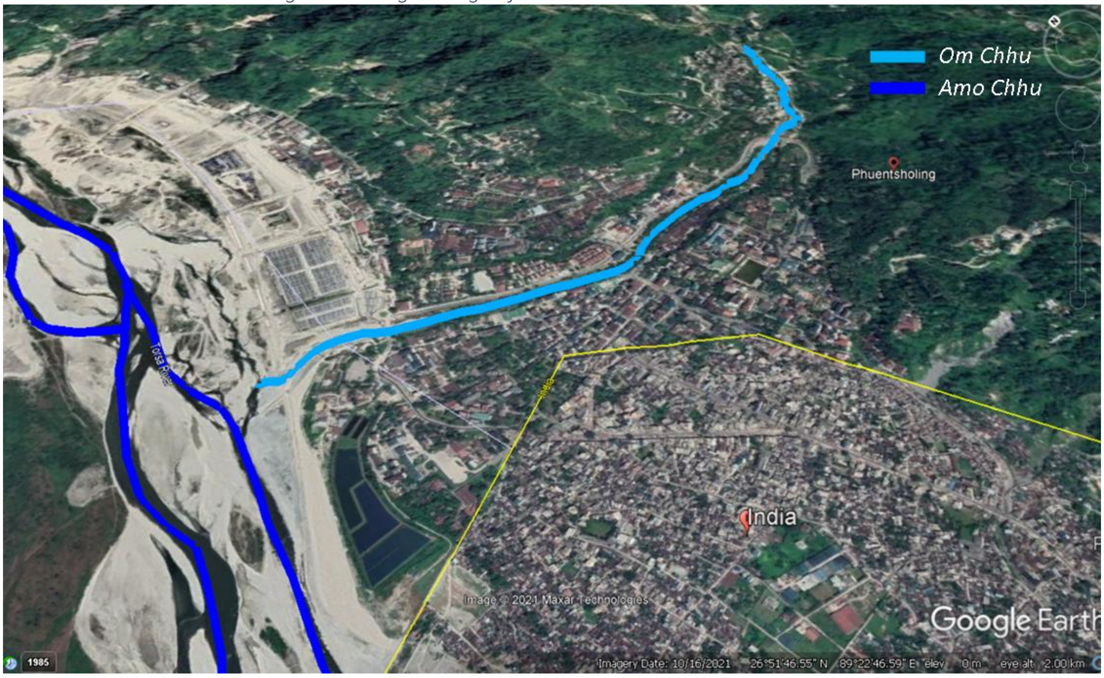
1 Executive Summary

1.1 Preamble

The Phuentsholing Thromde (PT) is located in the south western part of Bhutan, under Chhukha *Dzongkhag* (District), at latitude 26.8603° N and longitude 89.3938° E. The city is the gateway to western and central Bhutan opening the country to the Indian state of West Bengal. Phuentsholing is one of the major economic hubs of the country and hence caters to the huge influx of transient population and Indian tourists.

The Phuentsholing Thromde is featured by two river systems on the western front as shown in Figure 1; Amochhu (river) in the far west and Omchhu (river) which runs through the core city in the west. The two rivers pose significant threats to lives and properties during the summer when the river swells under the influence of tropical monsoon.

Figure 1: Google Image of the Omchhu and Amo Chhu river



While the river protection and land redevelopment work named as Phuentsholing Township Development Project (PTDP) along the Amochhu is being implemented by Construction Development Corporation Limited (CDCL) with loan from Asian Development Bank (ADB), the Omchhu protection is equally warranted to make the city resilient against flooding from both rivers.

With the increasing number of constructions works to meet the infrastructural requirements of the expanding city, there have been considerable changes in the land use pattern. The disruption to the soil morphology due to construction works combined with changing rainfall patterns have triggered landslides. The report submitted by the Department of Geology and Mines (DGM, 2010) for the area around Omchhu reported that the area is tectonically uplifted, compounded with fragile geologic conditions and geomorphologic processes. In light of the above observations and flooding caused by the river in the years 2000, 2016 and 2017, a detailed project report (DPR) of Omchhu was warranted.

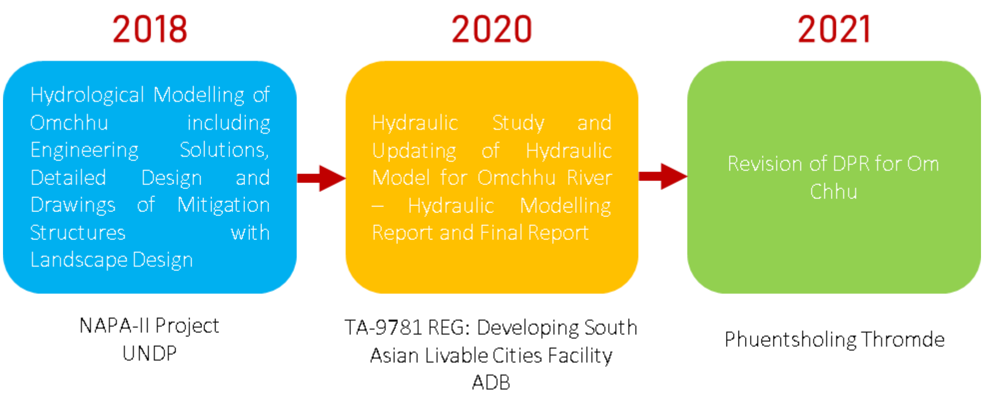
To improve the town's resilience to fluvial flooding from the Omchhu, the Phuentsholing Thromde (municipality) developed a Detailed Project Report (DPR)[1] including detailed topographical survey, geotechnical study, hydrological and hydrodynamic modelling, watershed management planning, detailed engineering designs and drawings of cost estimate, landscape designing, and cost estimates in 2018.

The report was reviewed by Mr. Chris Dunlop – Consultant, River Engineer from ADB, who sent a memo to Tshewang Norbu – Project Officer, ADB Bhutan Resident Mission (BHRM) on 8 July 2019. The review contained a critical analysis of the report and its shortcomings.

One shortage in the DPR was that two bridges constructed at downstream (near the mouth of the river- ADB Project No: L2816) and another curvilinear bridge constructed at the upstream (ADB Project No: L 3149/ G0400) were not considered in the hydrodynamic model. In addition to the presence of the new bridge, sediment deposition took place throughout the downstream river reach at the start of the 2019 monsoon season. The deposits had raised the water surface profile, reducing freeboard and hence the flood protection standard offered by the existing flood embankments. Subsequently, Egis International under the ADB TA-9781 REG: Developing South Asian Livable Cities Facility, prepared an updated hydraulic report[2] for assessment of the impact of flooding and sedimentation on the Omchhu River which addressed to several comments of Mr. Chris Dunlop's review report.

Therefore, this project was carried out to revise the DPR to address the remaining comments in the memo, conduct more extensive surveys and studies, determine a robust engineering design, and propose a pragmatic landscaping plan to make the project a sound, inclusive and sustainable investment.

Figure 2: History of the DPR Process



[1] Gyaltshen Consultancy, Hydrological Modelling of Omchhu Including Engineering Solutions, Detailed Design and Drawings of Mitigation Structures with Landscape Design, 15 January 2018.
[2] Egis International, Survey, Hydraulic Study and Updating of Hydraulic Model for Omchhu River Hydraulic Modelling Report, ADB TA-9781 REG: Developing South Asian Liveable Cities Facility, 28 September 2020

1.2 Objective

The objectives of the revision are:

a. To incorporate comments and suggestions as given by ADB (Eg.the memo from Chris Dunlop to Tshewang Norbu dated 8 July 2019).
b. To come up with a robust and long term climate resilient flood protection scheme along the specified reach of the Omchhu based on the revised hydraulic analysis and new geotechnical investigations.
c. To reclaim the land to the extent possible as per site conditions for riverfront development.
d. To come up with a pragmatic and sustainable riverfront development master plan that will benefit the community socially and economically which shall enhance community vitality and attract tourists.

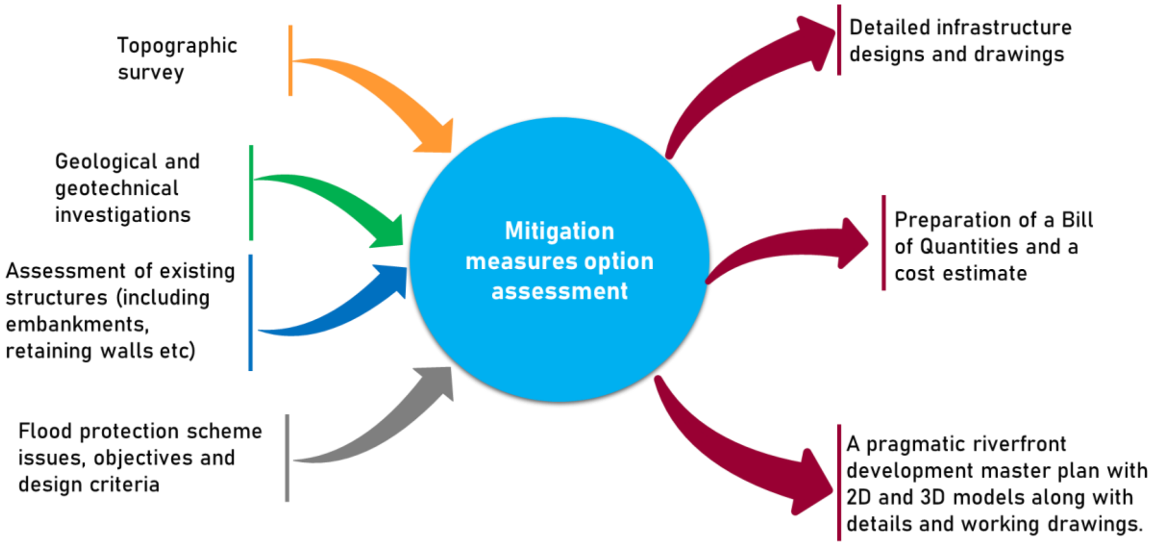
1.3 Scope of the assignment

The detailed scope of work as described well in the Terms of Reference (ToR) and comprised of the following;

i. Topographic survey.
ii. Geological and geotechnical investigations.
iii. Assessment of existing structures (including embankments, retaining walls etc)
iv. Flood protection scheme issues, objectives and design criteria
v. Mitigation measures option assessment.
vi. Detailed infrastructure designs and drawings.
vii. Preparation of a Bill of Quantities and a cost estimate.
viii. A pragmatic riverfront development master plan with 2D and 3D models along with details and working drawings. A minimum of two case studies (international best practices) relevant to the assignment must be carried out and submitted as part of the report.

As per the ToR, there were eight major scopes of work; each producing a definite deliverable in the form of reports. Understanding the scope, four scope can be considered as input, one processing, and three outputs.

Figure 3: Major scope and deliverables for Revision of DPR 2022



1.4 Detailed topographical survey

A new topographical survey was carried out for the whole areas identified along both banks of Omchhu required for the purpose of geological and geotechnical investigation, structural design, and landscape planning. The assignment included establishment of permanent ground control points and topographic survey at a scale of 1:1000.

Control Points (Horizontal and Vertical), were constructed at suitable location by means of Global Navigation Satellite System (GNSS) (coordinates computed using post processing software) while the survey was carried out using total station with angular accuracy of 3" and distance accuracy of 1.5mm+2ppm for the detailed survey. Reference System used was Druk Ref03 National Grid, while the vertical datum was Druk Geoid (EGM2008). All notable surface features (both man-made and natural) such as, existing structures, footpaths, trees, Posts, Pipelines, Streams, canal crossings, cross drainage structures, storm water drains, etc were surveyed. Contour survey was carried out in consideration to finally generate a map/data with contour interval of 1 meter. LISCAD, AutoCAD and ArcGIS software of compatible versions were used for data processing.

1.5 Detailed geotechnical study

A comprehensive geotechnical study was carried out to supplement the 2018 DPR. The 2018 DPR did not have extensive primary study on geology and geotechnical engineering, thus the present study provided valuable information.

At the Preliminary Investigation stage, Desk study and reconnaissance survey was carried out. The available information on Geology has been obtained from the available resources. The client also provided the consultant with other information relevant to this work like the previous study reports and drawings. The photo imagery of the landslide in Omchhu area was viewed in Google earth. The scope of the desk study depended on the availability and quality of maps, aerial photographs, and other published/unpublished materials/reports relevant to the area/work. The information available for the desk study saved a lot of time and labour and allowed for the identification and analysis of options for the geotechnical study, and served to focus attention on most relevant factors to be considered during site reconnaissance and detailed studies.

After the desk study, field study visit was made for a more detailed study including a check on the findings of the initial study and to collect field data wherever necessary. Data on geological, geotechnical, topography, hazard and risks aspects were gathered. The following factors were considered during this field survey.

- Geotechnical Mapping.
- The location of cliffs, gorges, ravines and other geomorphological features
- Slope steepness and limiting slope angles identified from natural and artificial slopes.
- Slope stability and the location of pre-existing landslides
- Rock types, geological structures, dip orientation, rock strength, etc.
- Soil types and occurrences
- Soil erosion and soil erodibility
- Slope drainage and ground water conditions
- Drainage features such as springs and seepages, sinks, stream channels and areas of wet ground.
- Drainage stability and the location of shifting channels and bank erosion

- Land use and its likely effect on drainage, especially through irrigation
- Likely foundation condition for the structures
- Flood levels and river training/protection requirements.
- Mitigation requirements

Geotechnical mapping was carried out by dividing the material into different geotechnical units. Soil geotechnical units were studied visually and logged using different field forms. Slope Stability Probability Classification (SSPC) form developed by Dr. Hack (ITC, the Netherlands) was used to collect the basic data on rock and to assess the rock slope stability. For weathering classification, a visual estimation was made in each geo-technical unit to arrive at the different rating parameters provided in the 'Rock Mass Weathering Classification' Form.

In terms of the field investigation, 15 number of Standard Penetration Tests (SPT) were conducted at 1.5 m depth as per the procedure in IS: 2131 – 1997. Plate Load Test (PLT) was carried out at 12 locations and roughly distributed as at 1 in every 200m but alternately on the two banks. However, due to lack of sufficient space, in three locations where PLT was supposed to be carried out, Portable Penetrometer Test (PPT) was carried out. The sub-surface investigation was carried out using Seismic Refraction Tomography (SRT) test at 20 locations, with 115-meter profile in each location.

The collected field samples were tested in laboratory as per the relevant IS and International Standards. The tests included, Grain Size analysis, Specific Gravity, Natural Moisture Content, Density Test (Dry and Bulk Density), Atterberg's Limit (Liquid and Plastic Limit), Proctor Compaction Test, Direct Shear Test, Consolidation test, Permeability test.

The soil composition in Omchhu area is gravelly sand to sandy gravels with variations in cobbles to gravels fractions. It was generally observed that the soil is not densely compacted and also contains high percent of gravels, which are rounded to sub rounded in shape, which suggests that the deposit type is mostly alluvial.

Mainly two rock units are mapped in the field. They are Phyllite and Quartzite. The rocks in the field work area belong to the Phuentsholing Formation. This Formation is represented by an alternating sequence of grey-greenish phyllite, phyllitic quartzite sequence with rare carbonates, intercalated with thick white-light grey fine to medium grained, locally gritty quartzite. The general trends of the rock are from ENE-WSW and NNW-SSE with low to moderate dips directed towards north to north-east.

Engineering geological Mapping shows four geotechnical units in the study area. They are the colluvial deposits, alluvial deposits, Quartzite bed rock and Phyllite bed rock. In the Omchhu area alluvial deposits are mainly the most extensive and is present all along on the banks and along the flow channel. The area being gentle, this flow deposit has covered a large area at the middle and lower portions of the study area.

The major discontinuities present in the study area are the beddings, foliation, joints and faults. The general regional strike of the rock is from East-West with northerly dips ranging from 15° to as high as 71°. In few cases, slight displacement of about 10 cm to 15 cm was observed in the faults. Fault gouge can be observed in few fault lines and especially near to the confluence of Namantri Chhu and Omchhu. This faulting is also considered one of the contributing factors for the failure of the slope.

Major factors determining the stability of a slope are Slope geometry, Soils and geology, Slope hydrology, slope angle, load, vegetation, surface water and ground water. Triggering mechanisms are in the landslides are mostly Rainfall though Seismic activity also contributes to slides but are not very common. Phuentsholing being located in the foothills the average annual rainfall ranges from 5000mm- 6500 mm, which indeed is high.

Literature review shows that a rainfall of 10 h or less requires a rainfall intensity in excess of 12 mm h−1 to trigger failure, but a rainfall duration of 100 h or longer with an average intensity of 2 mm h−1 can also trigger landslides in the Himalaya. Moreover, in the daily rainfall scenario, one study concluded that when daily rainfall amount exceeds 144 mm, there is always risk of landslides in Himalayan slopes.

Slope analysis using SSPC, the probability of the slope to remain stable is less than 5% for exposures RO-3, RO-12 and RO-20. The probability to be stable is found to be above 95% for the exposures RO-7, RO-11 and RO-15. The probability to be stable is found to be above 70% for the exposure RO-1. The probability to be stable is found to be above 30% for the exposure RO-2.

The detailed rock mass weathering classification gives 5 rock exposures in class C. This class has rocks which are significantly weathered. Three exposures fall in class B. This class has rocks which are slightly weathered. For the rest 2 exposures, they fall in class S1. This class has rocks which are geotechnically soil with relict discontinuities.

The Skiwedge Stereonet also shows significant slope failures due to wedge and/or planar failures. The weathering classification gives values for middle to lower rock classes, which suggests that weathering grade is high.

Sieve Analysis result shows the material in most of the area is not distributed in a wide range, so is a uniform soil with bad compaction characteristics. The coefficient of Permeability (k) values ranges from 2.20 x 10-2 cm/sec falling in medium grained sand to 2.753 cm/sec falling in fine grained gravel group.
The Proctor Compaction Test result shows Maximum Dry Density ranging from 1.66 to 1.923 g/c.c. The Optimum Moisture Content ranges from 8% to 10.50%. The specific gravity ranges from 2.61 to 2.71 g/cc. The permeability values range from 1.027 X 10-3 cm/sec to 2.991 X 10-3 cm/sec. The Consolidation analysis shows the result of the coefficient of volume change ranging from 1.5 X 10-4 cm2/kg to 2.7 X 10-4 cm2/kg with the corresponding compression index ranging from 1.1269 X 10-1 to 2.5838 X 10-1.

It is observed that the bearing capacity of the soil is low to nominal for foundations as in all the locations as the values are below 24 t/m2. The get a better bearing capacity of the soil, either the footing sizes needs to be increased or the depth of the foundation is to be increased. The best option is to increase both the footing sizes and depths of foundations. Bearing capacity of the foundations can also be improved by keeping the foundation dry which can be attained by constructing proper and deep cut off drains, increasing the footing size and also by increasing the depth of the foundation.

The Seismic Refraction Survey conducted along the project corridor at Omchhu shows four to five layers in all the seismic lines. It is expected that the first layer with compression velocities in the range of 400 m/s to 1800 m/sec could be related to unconsolidated overburden materials as Silt, Sand and boulders and the lower layer with recorded velocity in a range of 1800 m/s to 4000 m/s indicates the presence of bed rock, consisting of highly weathered and fractured phyllite or quartzite.

The major hazard for the study area is from landslide triggered by rain compounded by fragile geology, topography, landuse and geomorphology.

The instability can be minimised by carrying out mitigation measures through slope geometry change, minimizing ground water influence and strengthening or supporting slopes. Few recommended measures are stepped RCC/RMM walls, Gabions and garland drains.

The Standard Penetration Test conducted in the field area shows values from 7 to 20 'N' values. The Ultimate Bearing Capacity calculated after necessary corrections for overburden pressure ranges from 149 kN/m2 to 537 kN/m2 for 1 metre foundation width. The Ultimate Bearing Capacity calculated for 3 metres foundation width is from 201 kN/m2 to 785 kN/m2.

The Ultimate Bearing Capacity calculated from the plate load test ranges from 251 kN/m2 to 360 kN/m2. The values obtained from this PLT test are also not very high.

The three (3) Portable Penetration Test conducted in the field area shows values from 7 to 21 'N' values. The Ultimate Bearing Capacity calculated ranges from 105 kN/m2 to 190 kN/m2 for 2 metre foundation width. The Ultimate Bearing Capacity calculated for 6 metres foundation width is from 79 kN/m2 to 145 kN/m2.

The Direct Shear Box Test result shows cohesion from 0 to 0.05 kg/cm2 and the internal friction angle of 21°04' to 26°33'. Taking the obtained shear parameters, bearing capacity was calculated for 3 metres foundation depth for a square footing size of 4 X 4metres, which when calculated using local shear failure criterion, the values range from 13.64 t/m2 to 23.48 t/m2. The factor of safety considered in the calculation of safe bearing capacity is 2.

1.6 Assessment of existing structures

An assessment of the existing structures was carried out by the consultant team through physical verification by walking along the entire stretch. Apart from measurements and visual inspection, the consultants also carried out non-destructive test for compressive strength and concrete homogeneity. It was observed that the pertinent issues for the existing structures include the following;

Table 1: Critical observation of existing flood protection system

Classification	Description	Representational Photos
Not Damaged	• RCC and Gabion Wall Constructed at levels above HFL • Not constructed with the purpose of flood protection but as a retaining structure for the walkways on the bank top. • In relatively good shape and functionality.	

Classification	Description	Representational Photos
Completely Damaged	• Most gabion walls • Toe is all completely eroded, and is resting based on the self-weight • Could be eventually washed away, leading to eventual scour of the banks • In some stretches, the debris have been protecting the gabion being washed away. s	
Structurally Damaged	• Some RCC Walls which have are not toppled, but the concrete cover has been eroded, and TMT is exposed • Clear cover is observed to be less than 40mm in most cases • Toe is protected in most parts	
Scouring in Progress	• Relatively new walls • No boulder pitching and directly exposed to continual scouring • Stagnation of water on footing. • Also compressive strength is less than 20/mm2	
Safe but detrimental to river discharge	• These are RCC Walls that is protected against scouring with relatively sloped riverbed and vegetation • However, river bed level is increased by more than 2 meters resulting in less discharge capacity	

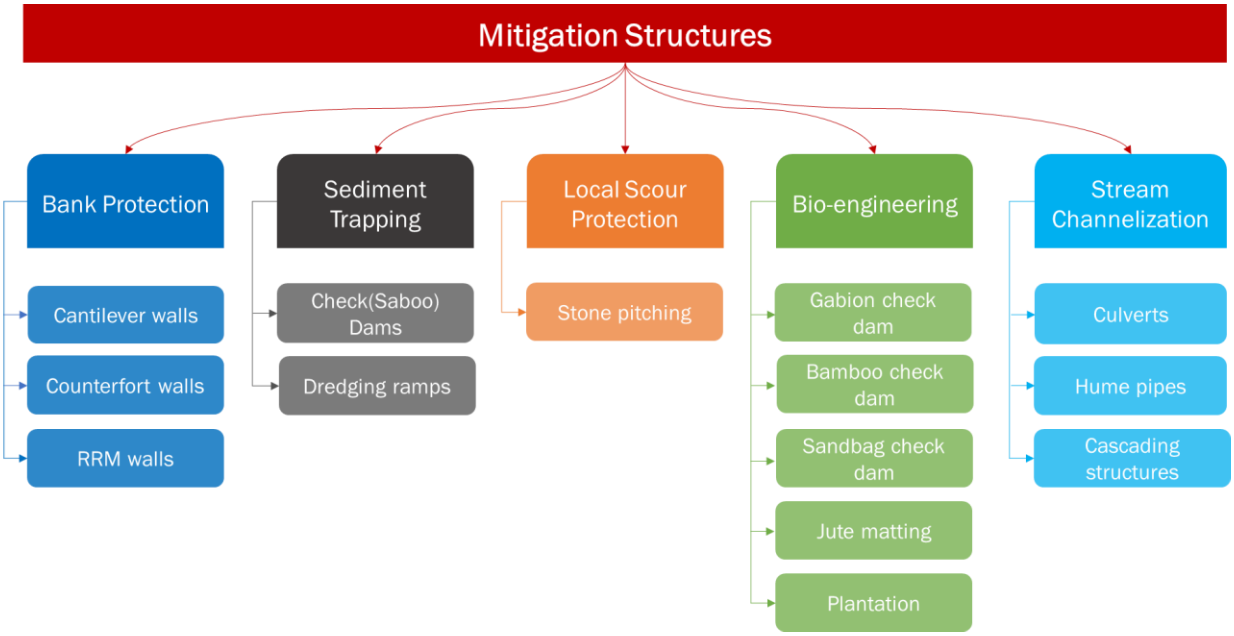
1.7 Proposed flood scheme

Principles or considerations behind the conceptualization of climate resilient measures to mitigate flood hazard in Omchhu are:

1. Structural measures are climate resilient, environment friendly and sustainable
2. Climate resilient measure has been considered for extreme discharge of 660 m^3/s (i.e., 1 in 100 ARP) including climate change scenario.
3. The entire stretch of Omchhu from Bailey bridge at Kabraytar has been considered for a continuous flood defense scheme.
4. No protection for about 20 m both upstream and downstream of the existing bridges (Bailey Bridge, Curvilinear Bridge, RSTA Bridge, Pedestrian Bridge, and Omchhu bridge) has been proposed as this could potentially undermine the bridge foundations.
5. For the severe active landslides at 3 locations, specific measures have been proposed.
6. Allowing smooth passage of maximum flow considering natural slope of the stream, morphological aspect, required width & uplift pore water pressure.
7. Availability and accessibility to potential construction materials
8. Potential usage and benefit of excavated sediment in or outside of country
9. Ease of construction
10. Potential to develop maximum land for landscaping purpose has been considered.
11. Perceptions of local engineers as well as local stakeholders
12. Budget constraints of local administration for implementation and maintenance
13. Doability and durability

On the basis of the above principles following structural measures have been outlined which is assumed to address as a strong flood defense mechanism along the entire Omchhu river.

Figure 4: Summary of the proposed mitigation measures



1.8 Proposed Landscape plan

The Omchhu Riverfront proposal now host a vibrant and robust combination of many amenities, like events plaza, sports arena, parks, gardens, open spaces, footpaths and cycle trails and several other community assets and infrastructure like cafeterias, vendor stalls, public toilets, cycle sheds etc. as it is agreed that the riverfront area should be an environment for diverse activity and expanded recreational opportunities besides being flexible and can respond to market conditions and economic opportunities. A mix of open and covered spaces has been considered and proposed. These provide opportunities for year-round activities. The Omchhu riverfront has been envisaged as a well-known destination that draws people of all ages and the region to the riverfront and should offer places and opportunities for celebration throughout the year; from small informal gatherings to large festivals and educational activities. Recreational facilities include playground facilities for children and parks which can enhance learning of local Bhutanese culture, flora and fauna as well as of the nation.

Open sports facilities for people like open air gyms for yoga and exercise, volleyball, badminton, futsal grounds, spread along the Omchhu banks have been proposed. A more pedestrian oriented waterfront, with walking and cycling trails inviting intimate contact with the landscape and free from vehicular traffic. Barrier free or universal access design principles have been integrated in the design of all recreational facilities to make them accessible to all people, regardless of age, disability or other factors. Diverse trees, both ornamental and fruit bearing, evergreen and deciduous trees, plants and flowers have been intentionally chosen. Suitable evergreen species at appropriate locations for shade and aesthetics have been selected. Besides common trees like the Gulmohar, Jacaranda, Palm lesser-known tropical ornamental trees like Flame of the Forest, Himalayan ash, Hong Kong Orchid, Javanese Cassia, Mexican lilac, Purple Glory, Sandpaper Vine, Tabebuia, Yellow Mai tree, Yellow Elder etc. are being proposed to introduce a variety that is both refreshing as well as educational. Lawn grass has been proposed over the entire stretch of the Project area to reduce dust pollution and promote greenery. Grass that is flood and drought resistant need's introduction. Vetiver or khus is one such grass. Vetiver is most closely related to Sorghum but shares many morphological characteristics with other fragrant grasses such as lemongrass, citronella and palmarosa. The vetiver bunch grass in tufts and its root system is finely structured, is very strong and can grow about 3 m deep within the first year. It is thus frost, wildfire, drought and even flood resistant. Under clear water the plant can reportedly survive up to two months. Its strong deep fibrous root system can help to protect soil against heavy grazing pressure, and soil erosion.

The riverfront is designed with walkways, recreational parks, religious and cultural attributes, exercise and play areas for children and adults, fountains etc. Creating multiple destinations, connecting destinations and optimizing public access for interaction purpose. Connecting public open spaces with a continuous riverfront trail to link destinations and serve as a destination for walking, jogging and other related purposes. Facilities such as drinking water taps, public toilets and eateries have been provided as per the requirement worked out. Adequate artificial lighting with innovative ideas such as using eco-friendly lights have been proposed.

Interaction with the river has become important for planning of sustainable development. This can be tackled by selecting the heights, materials used for building, native as well as exotic plants for landscaping; reusing disturbed areas and building within the context. Public accessibility must be enhanced. People are drawn to water. Human interaction with water is innate and instinctive especially when complimented with sensitive design that is both inviting and innovative. Once introduced landscaping and water features inculcate the desire to be near it, physically or at least in its visual proximity. It therefore becomes critical that the Omchhu should have water in it especially during the non-monsoon seasons. Omchhu river front landscape development would be significantly impaired if the Omchhu didn't have water in it. To achieve this check dams have been introduced. These check dams perform the following critical functions

- Checking the flow of the river in flood
- Storing water for aesthetic and practical purposes like gardening and rainwater storage

The check dams also offer the opportunity to generate electricity through mini hydro-electric plants to provide electricity for the electrification of the riverfront and paying for its own upkeep. It must be mentioned here that the ferocity of the Omchhu during the monsoons will in all probability cause damage to these dams. Yearly maintenance to the dams themselves as well as annual dredging have to be done periodically. Nine of these check dams have been provided- mainly in the middle stretch of the Omchhu. These dams have also been strategically located to enhance the overall ambience of the landscaped gardens and parks along the Omchhu.

To achieve these objective walkways, trails and benches are provided as they give people an opportunity to be either in the river or near it. An effective or fruitful riverfront having active use can be achieved if multiple entry points to the river are available. Walkways along with bridges are important as they define as well as provide variety in movement patterns on the site. They are also physical pedestrian linkages between different parts of the site. A traditional looking Baazam constructed of RCC has been proposed to add a Bhutanese architectural element to the entire ensemble. In a similar vein a suspension bridge has been introduced. Besides being a cost-effective way to provide pedestrian, there is still the novelty of walking on a swaying suspension bridge.

1.9 Project Cost

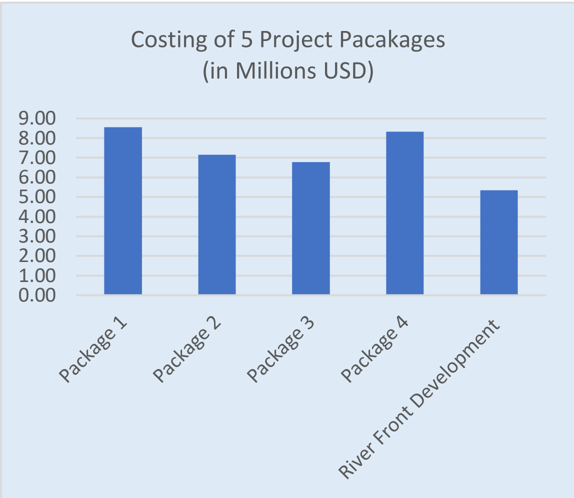
The Project Cost has been prepared based on the Bhutan Schedule of Rates 2022, and where necessary based on the analyzed rates from the market. The rate of conversion for Ngultrums to US Dollars is 1 USD= 80 Nu. based on the current rates. There are four packages of construction, and envisaged to be completed in a phased manner in the event of budget constraints. The packages are divided as follows;

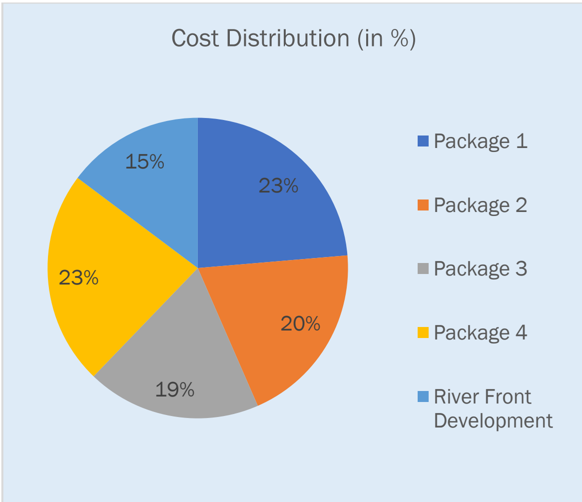
1. Package 1: Omchhu Bridge 2 To RSTA Bridge
2. Package 2: RSTA Bridge to Curvilinear Bridge
3. Package 3: Curvilinear Bridge to Upstream of Bailey Bridge (Ch-2750)
4. Package 4: Upstream of Bailey Bridge (Ch-2750) To Water Treatment Plant
5. Package 5: Riverfront Development (Landscape component)

The packages are of similar financial value and of the distance coverage. It must be realized that the execution of the entire stretch at one go may not be practical since ideal construction period would only be from October till April (6-7 months). Therefore, even the packages themselves might have to be sub-contracted and such provisions should be thought about by the client. The landscape development has been considered an independent package, and while the overall masterplan shall become the guiding principle, it is subject to change based on the final land profile and availability after construction of the flood defense mechanism.

The -costing is presented in the format of 5 components which includes the 4 packages and the Landscape component. This amounts to a total of USD 36.35 Million. The 4 packages alone constitute USD 30.85 Million. The major cost across all packages is for the reinforced wall construction which accounts to flood defense structures which accounts to USD 24.8 Million, while the earthwork component in these packages constitute another USD 2.71 Million.

Figure 5: Summary of the Project Cost





THIS REPORT, hereafter, presents a comprehensive Detailed Project Report (instead of only the Revision of the DPR). Therefore, the content of the 2018 Detailed Project Report and content of the 2020 Hydraulic Modelling Report has been used where appropriate to provide completeness to the report.

2 Introduction

2.1 Overall Approach

The Figure 6 shows overall approach adopted in this study. Both primary and secondary data collection has been done after extensive literature review on study interests.

Topographical survey was carried out for the entire project area. Permanent control points were established for future reference. Subsequent mapping and plotting of the survey data as per requirement of geotechnical, structural, and landscape studies was prepared.

In terms of the geotechnical study; desk study, field investigation, and laboratory tests were carried out. Field investigation included, Standard Penetration Tests (SPT), Plate Load Test (PLT), Portable Penetrometer Test (PPT), and Seismic Refraction Tomography (SRT) test. The final output was Geological Mapping, determination of bearing and settlement for design of structure, and Slope and Hazard Mapping. An assessment has been carried out to identify potential morphological hazards in the Omchhu catchment and add substantiation to the modelling results which may suffer from limitations due to the lack of proper field data.

Then detailed hydrological analysis has been performed through watershed analysis developing rainfall runoff model using both empirical approach as well as HEC-HMS. This was followed by flood hazard mapping through 2D hydrodynamic modeling using HEC RAS 2D using a peak flow of 660.89 m3/s from SCS CN Method using Monte Carlo Simulation as the maximum flow has been selected as a 1:100-year return period flow with climate change impact scenario. The hydrodynamic model was prepared based on several varying boundary conditions, highlighting probable scenarios.

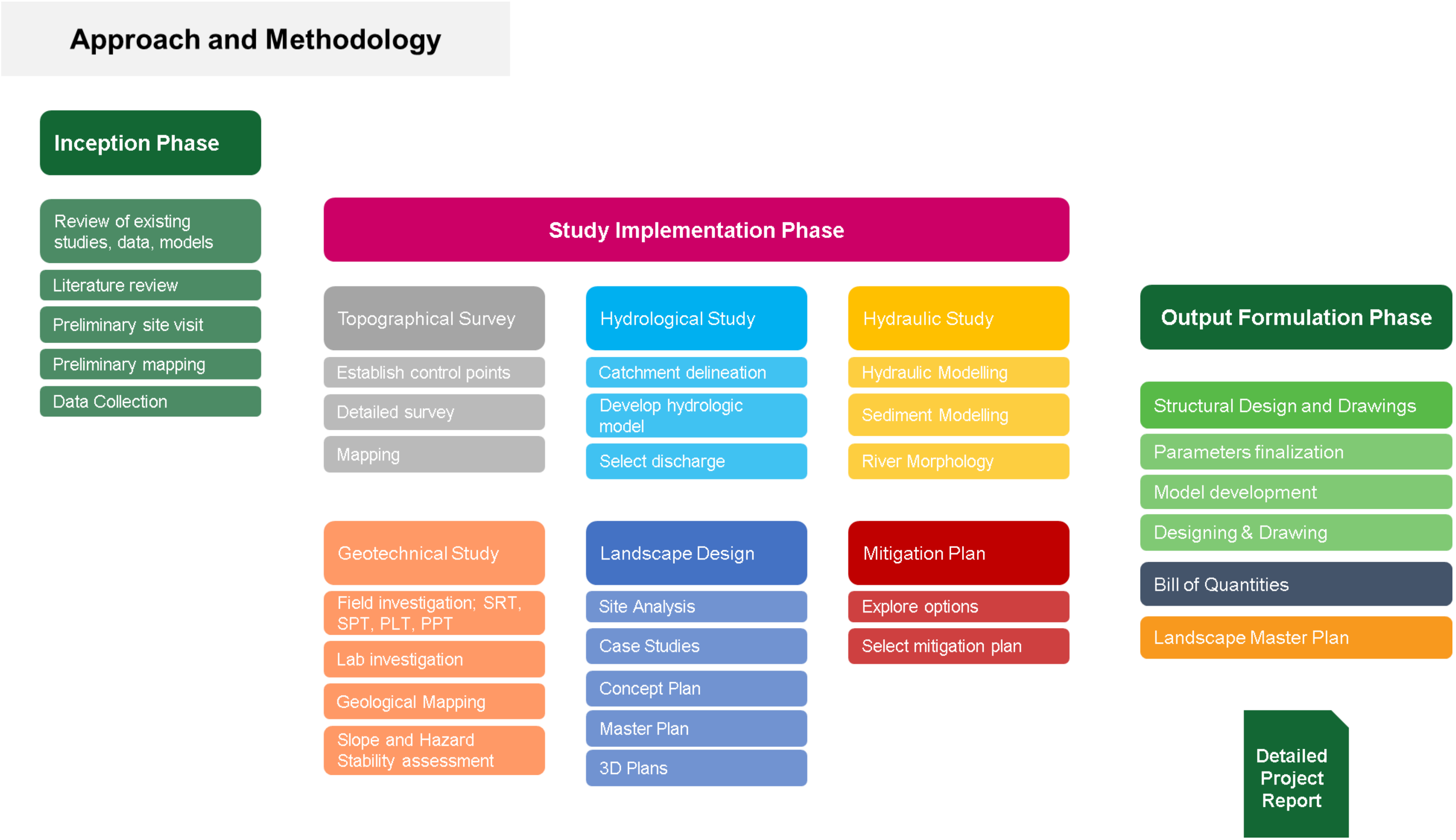
This was also followed by a Sediment modelling. Since calibration and verification of the morphological model has not been possible, a sensitivity analysis has been performed to identify which parameters may be of importance for the sediment modelling of the Omchhu.

An assessment of the existing structures was carried out by the consultant team through physical verification by walking along the entire stretch. Apart from measurements and visual inspection, the consultants also carried out non-destructive test for compressive strength and concrete homogeneity.

Based on the data obtained from hydraulic and geotechnical study, robust engineering structures have been proposed to mitigate the banks against flood and other possible hazards. Design and detailing of the structures have been carried out (Plan, Elevation, sections and reinforcement details). Detailed BoQ and estimate for every item of the infrastructure component has been prepared as per the finalized site-specific designs and drawings and material specifications.

In parallel, landscaping has been proposed considering the existing topography and geomorphology anticipated after the river training works are completed to make best use of the land after the flood defense mechanism is implemented. This was carried out after thorough site analysis study of the nearby settlements to understand its viability. Two case studies (international best practices) were carried out to adopt the concepts in the present study. Finally, a pragmatic and implementable Riverfront development master plan with 2D and 3D models have been prepared.

Figure 6: Approach of the study



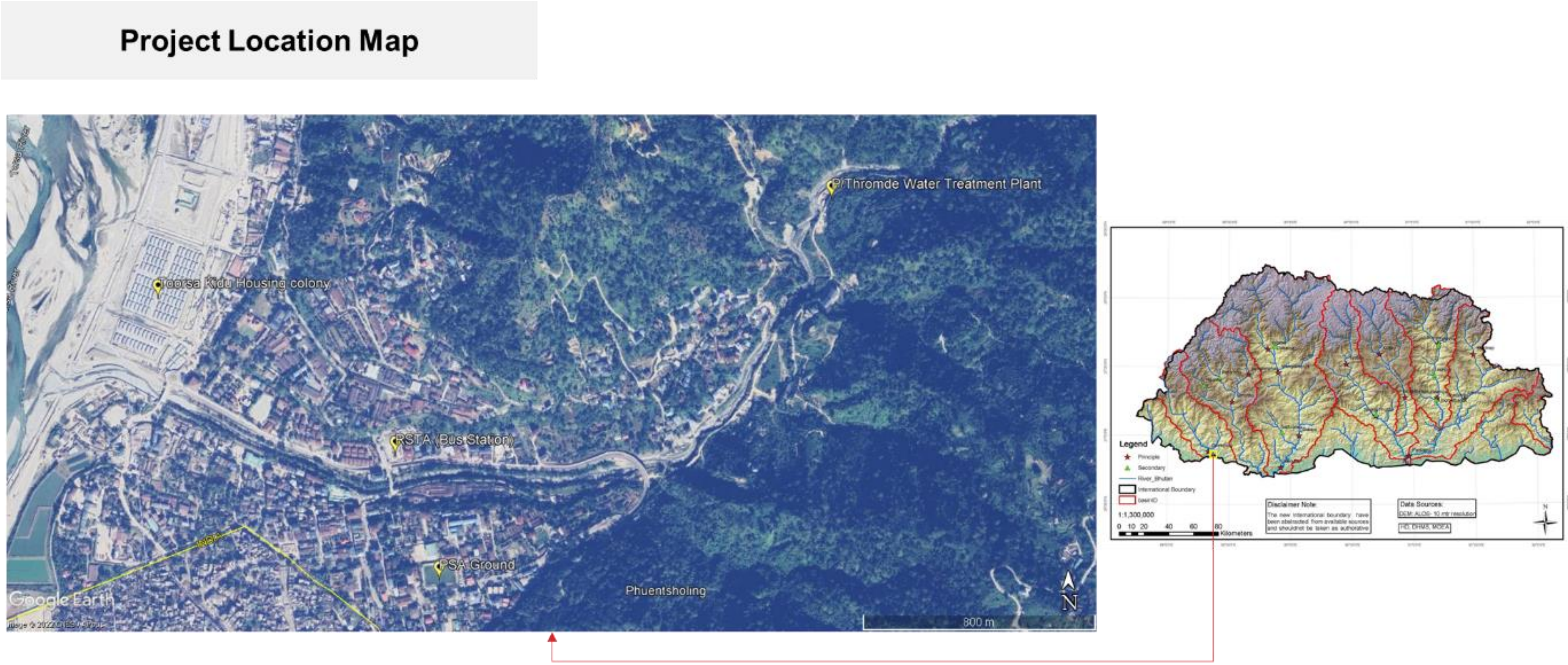
2.2 Project Location

The Omchhu is a small perennial river with an approximate catchment area of 22.50 km^2. From the hillsides of the southern Himalayan belt, it runs right in the middle of Phuentsholing Thromde which is one of the most important gateways to Bhutan and an important economic hub for Bhutan.

Phuentsholing is connected by a 171 km long highway to Thimphu, the capital of Bhutan. Phuentsholing is a Class A Thromde (Municipality) comprising of 9 Local Area Plans. The Core Area falls within the northern latitudes of 26°51'40.77"N and the eastern longitude of 89°23'2.54"E in the survey Toposheet No. 78 F/5. The study area is bounded by the latitudes of 26°52'23.80"N to 26°52'1.08"N and the longitude of 89°24'7.54"E to 89°22'28.38"E

On the right bank of the Omchhu, there are new extended areas and a vast new township under development. On the left bank, the territory is covered by the old part of Phuentsholing, which includes commercial hubs, service centers, and Government, and Private Offices. There are now three major motor bridges crossing the Omchhu within the Thromde boundary. The Omchhu flows into the Amochhu river system just upstream of Bhutan’s border with India. The Amochhu originates in China and flows through the western Bhutan districts of Haa and Samtse before finally draining via Chhukha district onto the plains of India. The upper catchment is at a high elevation with steep slopes. The Amochhu catchment area at Phuentsholing is approximately 3,785 km^2.

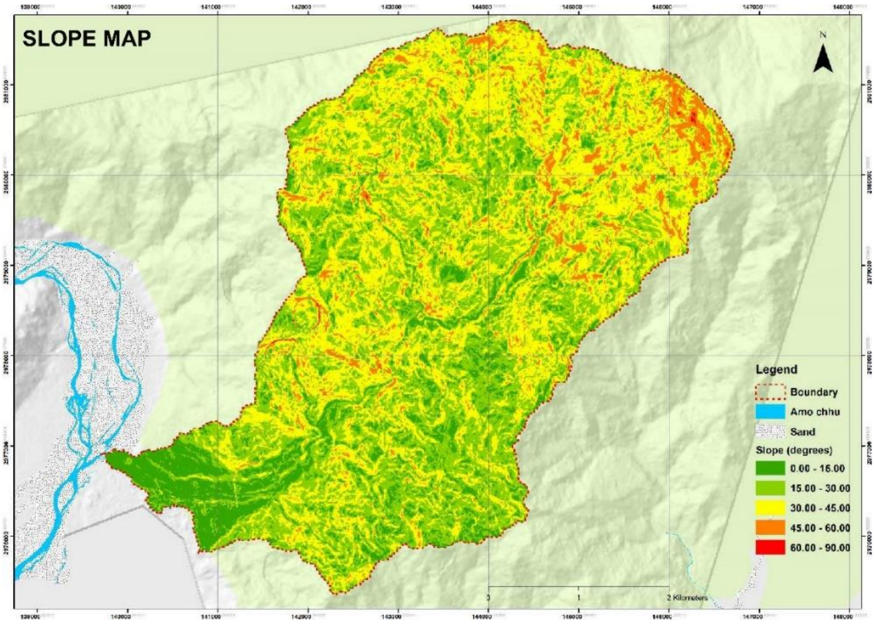
Figure 7: Project Location



2.3 Topography

The maximum slope of the Omchhu catchment is between 30-45° with 42%, followed by 15-30° in the gentler areas with 37%, while 13% are less than 15°. As we go further up on the hills the slope is higher as shown in Figure 8.

Figure 8: Slope Map of the Watershed

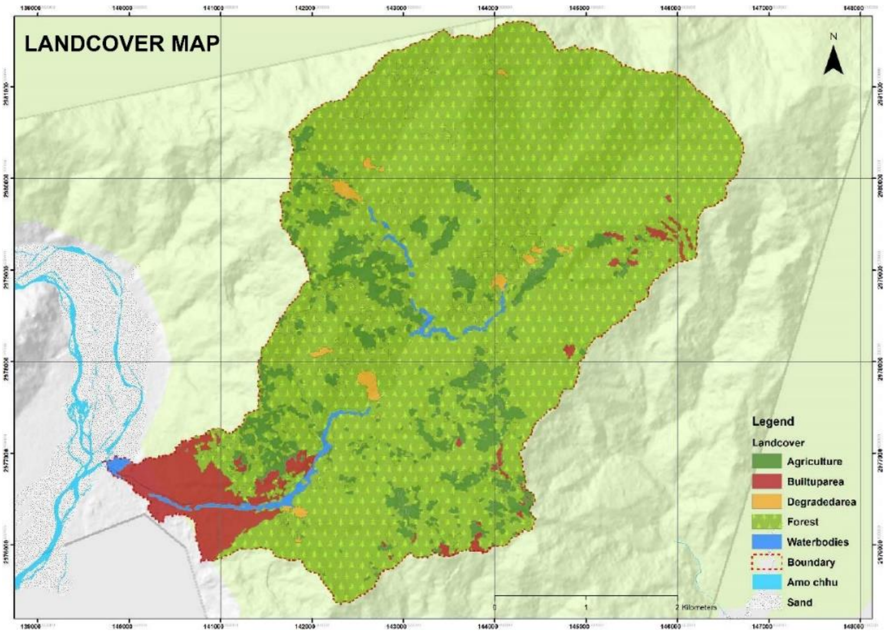


Source 1: DPR of Omchhu 2018, Gyeltshen Consultancy

2.4 Land use

Distribution of land use classes shown in Figure 9 depicts that in all of the river's watershed area, forest cover the major portion around 71%, followed by 11% agriculture land, 8% open land, and 5% of impervious built-up land[3].

Figure 9: Landcover map of Omchhu catchment



Source 2: DPR of Omchhu 2018, Gyeltshen Consultancy

[3] Tshewang et al., (2014) Hydrology Modeling Using SCS-CN Method And HEC-HMS In Omchhu Basin.

2.5 Climate and Temperature

The annual average rainfall for the data from 1996-2021 is 6,125 mm, most of which occurs in the months of June to September (46.8%). In terms of temperature, the average maximum temperature in January is 23 °C while in July it is 28.5 °C.

The maximum 24-hours rainfall during the time period was 495.3 mm which occurred on 2nd August 2000. The second largest event was on 11th July 2019 which was 318.8 mm. The other top events are between 310-250 mm.

Figure 10: Details on Rainfall and Temperature (1996-2021)

2.6 Population and Socio-Economy

In line with the Local Government Act 2009, Phuentsholing Thromde is a self-governing municipality with an elected Mayor, Thromde *Thuemis* and a Thromde Council in place since January 2011. As per the 2021 Statistical Yearbook, the population of Phuentsholing Thromde is 27,658 (15,052 Male, 12,606 Female). The Core Area LAP along which majority of the Omchhu runs has an area of 440.4 acres, existing population of 9,304, while the LAP carrying capacity is 23,706[4].

With some of the large mineral, metal and wood-based industries located in Pasakha, Phuentsholing is one of the major industrial hubs of the country. There are 1,583 business establishments (7.2%) in Phuentsholing Thromde; however, more importantly, under Production and Manufacturing (P&M) firms, there are 96 large scale, and 46 medium scale which make up 37.5% and 26.5% of the total large-scale P&M firms in the country. The recently operational Mini-Dry Port and ongoing construction of Ahlay Customs Office in Pasakha will smoothen the transport logistical issues.

It is also a major trading hub due to its close proximity to the Indian towns of Siliguri and Jaigaon. The town also has a well-established network of financial and social institutions, business enterprises, whole sale traders, training institutions and a tertiary educational institute (College of Science and Technology, Royal University of Bhutan) attracting people, goods and services.

There are two industrial estates in Phuentsholing. The Phuentsholing Industrial Estate has an area of 61 acres, including one acre near Omchhu, and houses more than 15 industrial sheds. The Pasakha Industrial Estate is located 16 km east of Phuentsholing in an area of 267 acres. The industrial estate is connected with all facilities like road network, water supply, power and transmission lines, drainage systems, telecommunication facilities and estate management office.[5]

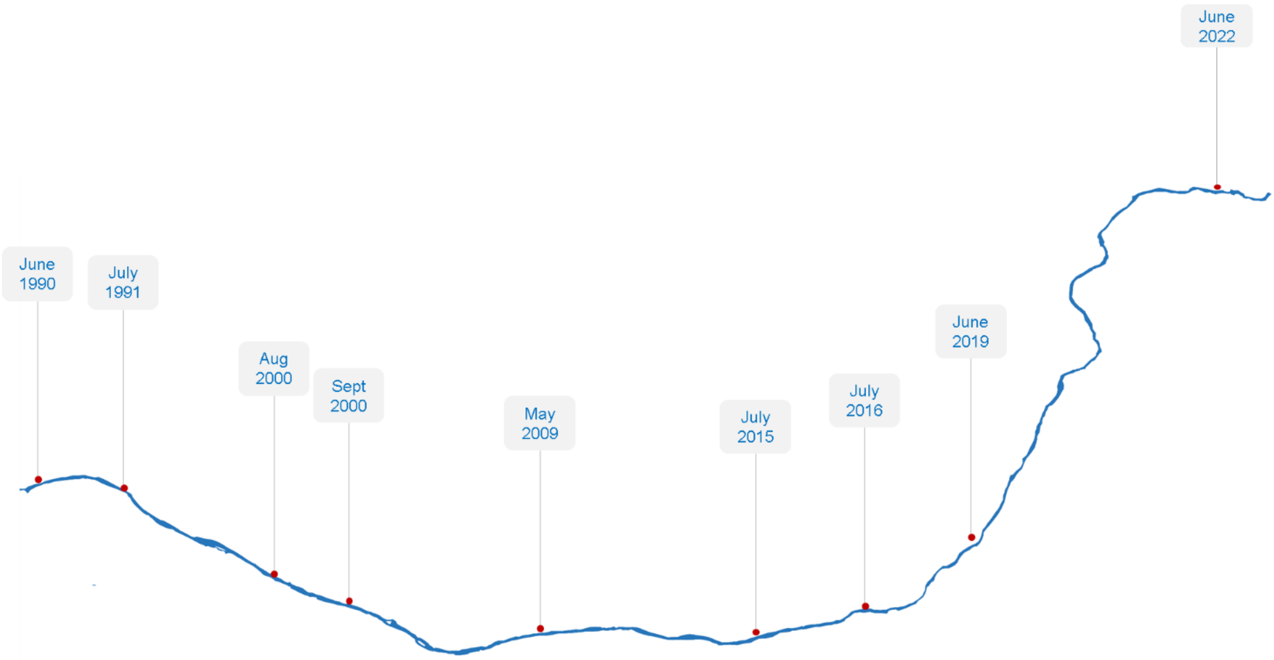
[4] Phuentsholing Thromde Core LAP Google Map (http://www.pcc.bt/divisions/urban-planning-division)
[5] Eleventh Five Year Plan (July 2013 – June 2018) Local Government Plan – Volume III

2.7 Flood and Flood Protection in Omchhu

2.7.1 List of Flood Incidents

While a comprehensive documentation of the various flood events is not available with the Phuentsholing Thromde or any other agency, the consultants have compiled the major events shown in Figure 11. However, it must be noted that, the partial failure of the flood defense mechanism of the Omchhu is an annual occurrence, as well as, immediate reconstruction of critical stretches on recurrent basis.

Figure 11: Timeline of major flood events of Omchhu



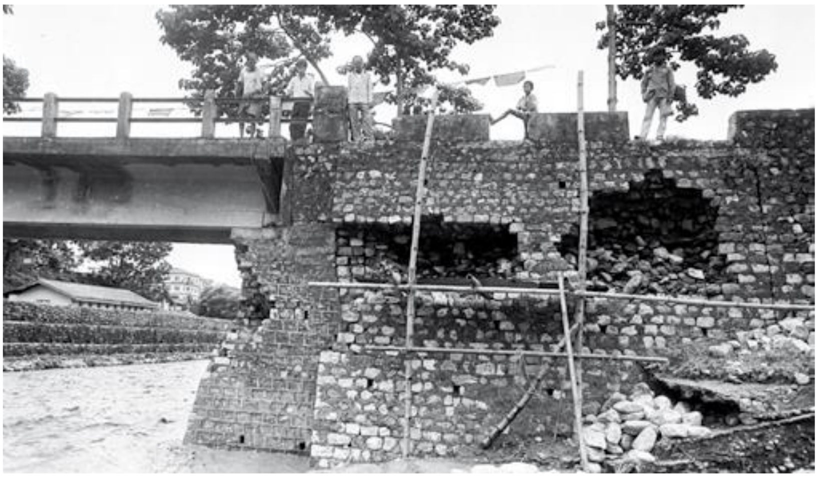
Some details (excerpts)of the flood events that could be retrieved from news articles and Compendium of Climate and Hydrological Extremes in Bhutan since 1968 by National Center for Hydrology and Meteorology (NCHM) is presented below;

Date: 2nd June 1990

The flood originated from the river's source near Sorchen, carrying down huge trees and boulders. Huge surges of water cutting two concrete units in half and carrying away two small shacks. Flooding two shacks occupied by staff of the Central Water Works Commission. The worst hit area was the Bhutan Oil Distributors, located opposite the Chukha colony, where the flood broke a portion of the wall and entered the pump, sweeping away many oil drums.

Logs, boulders and various other debris struck the Norgay Bridge with tremendous impact, damaging the bridge quite severely. Some people said that the water touched the bottom of the bridge, which is about 30 feet from the river bed. Leaving behind about two feet of silt covering the entire area over the BoD.

Some of the victims feel that, though the flood could not have been averted, the direction taken by the river was affected by protective work done near the Norgay Cinema. The Thrompon, Pema Dorji, said that the channels built by Yarkay Constructions on that side had been done with proper studies and that “the channels made no difference, when such a force of water was involved.” The City Corporation had also constructed gabion walls along some of the vital areas, which were also washed away.



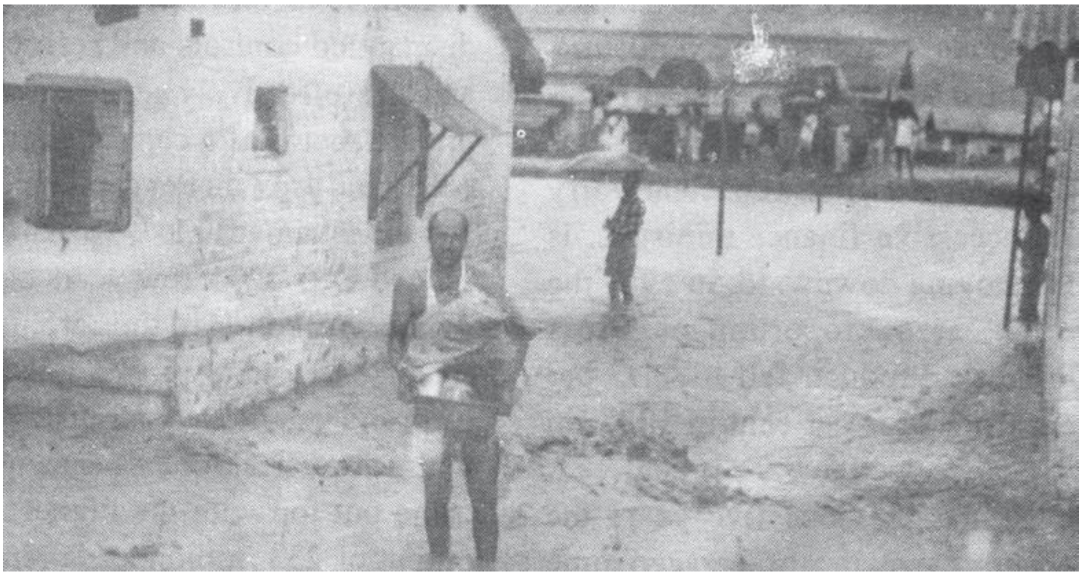
Source 3: Kuensel Corporation Limited, Bhutan

Date: 13th July 1991

The five days of rain In Phuentsholing resulted in a heavy flood. The water supply pipes at the Duti Khola source in Phuentsholing have been washed away. The small huts around the colony were all swept away and the diesel generator, the station transformer and the station power supply plant were all disrupted. The residents of the colony had to flee at midnight to save their lives and most of their property have been damaged by the water. Another flood occurred near the Phuentsholing lower BOD but no major destructions were reported.

The Duti Khola, which was in spate from the continuous rain, had washed away the barrier walls and the spurs at different intervals, thereby diverting it’s course towards the CHC colony.

Residents have appealed for measures to control the flood should be taken as this was not the first time that Phuentsholing has suffered damage from floods, reports Kuensel’s correspondent in Phuntsholing.



Source 4: Kuensel Corporation Limited, Bhutan

Date: 5th August 2000

In the worst flood disaster in the history of Phuentsholing town, peaking at about 7.00 am on Thursday, a swollen Dhuti Khola changed its course and unleashed havoc on a large part of the country's economic hub.

17 huts were washed away in the vegetable market while the BOD fuel station, a private sawmill, and the city corporation water supply office were submerged. An automobile workshop and the city corporation's staff quarters were severely damaged. Trucks, buses and other vehicles in the area were submerged, and two main water pipe bridges burst, cutting off the town's water supply.

RBA, RBP and civilian volunteers rescued more than 40 people trapped in their houses by pulling them out. In the worst flood disaster in the history of Phuentsholing town, peaking at about 7.00 am on Thursday, a swollen Dhuti Khola changed its course and unleashed havoc on a large part of the country's economic hub.

The flood came after nonstop torrential rain which began on Monday evening with the Central Water Commission office recording 416.40 millimeters of rain. The highest recorded rainfall in Phuentsholing was 394.6 millimeters in 1996.

Source 5: Kuensel Corporation Limited, Bhutan

Date: 2nd September 2000

The damages in Phuentsholing alone amounted to Nu 37.267 million. Most of Phuentsholing water supply infrastructure was destroyed while major damages were inflicted on its sewerage system, urban roads and drains, the Dhuti Kho-la river training work, office equipment, plumbing tools and vehicles.

The UDHD attributed the cause of the maximum damage to the water supply systems to the long pipe lines drawn on hill slopes from distant sources.

Date: 25th July 2016

Continuous rain triggered a flood in the town submerging the border gate area. Similarly, drains in the Kabraytar and Dhamdara areas also overflowed and roads were damaged. The Omchhu had also swollen into a massive river and the construction site for a bridge located at the confluence where it meets the Amochhu was affected. Protection walls have collapsed in several areas.

It was a better day yesterday with less rain in Phuentsholing. Mitigation works continued on both the Amochhu and Omchhu. Flood water in most of the flooded areas had receded and dried by the evening yesterday. Army and police personnel, desuups, officials of the thromde, dungkhag and government, and other volunteers were able to initiate mitigation works in other risk zones. A retention wall for the Omchhu that had collapsed was reinforced to prevent water from entering the town.

Source 6: Kuensel Corporation Limited, Bhutan

Figure 12: Omchhu embankment repairs following 2009 flooding

Source: PPTA

Source 7: ADB PTTP (2016). Disaster And Climate Risk and Vulnerability Assessment

Figure 13: Damages caused by Dhoti Khola in 2016

Source 8: FEMD, Flood Hazard Assessment for Chhukha Dzongkhag (2019)

Date: 25th June 2019

Heavy rainfall in Phuentsholing triggered several flash floods, landslides and blockages across the Thromde. People residing along the Omchhu river had to be evacuated at about 3 AM today due to swollen Omchhu river. Workshops along the Toorsa area also saw overflowing drains filling the area. Schools under Thromde also had to postpone their examinations due to the damages caused by the incessant rainfall. However, no casualties were recorded in the region.

The swollen Omchhu had also washed away a portion of the road and rose to the deck of the newly built bridge near the Youth Development Fund building. Excavators had to be deployed to clear out the debris.

Figure 14: Omchhu Flooding on 25th June 2019 (D/S of Foot bridge and at the mouth of Amouchhu confluence)

Source 9: Bhutan Broadcasting Service (June 25, 2019 Facebook Post)

Date: 27th July 2019

Nu 75 million Omchhu bridge built on 85% loan needs 4 excavators 24/7 to save it from flooding. Four machines have been deployed at Omchhu bridge in Phuentsholing for 24 by 7 dredging purpose no matter what the weather forecast is. The four machines which were deployed from June 27 will be deployed until 27 August and if required they will further extend the timing.

Ironically, the problem with the newly constructed Omchhu Bridge in Phuentsholing is that it is vulnerable to flooding given the low river bed levels due to sedimentation. The risk is high whenever there is heavy rainfall. The Thrompon said that in case of heavy rainfall they deploy 10 additional machines. Talking about the threat to settlement, he said that they have evacuated 18 families of the National Labor Force and they are temporarily being kept in the Badminton court in Phuentsholing.

Figure 15: Water level has reached to the deck of new Omchhu Bridge

Source 10: Business Bhutan (12 July 2019)

2.7.2 List of past projects

With the Omchhu known to pose serious threats to life and property during monsoon, the Phuentsholing Thromde has also made commendable funding to develop the flood defense infrastructure. The list below provides details of the maintenance projects that was carried out until 2022.

Table 2: List of investment made on Omchhu River

SL. No	Name of Project	Works carried out	Exact. Location (If available)	Year of Execution	Budget (million)	Contractor
1.	ADB-financed "Urban Infrastructure Improvement Project	Not Available	Not available	2001	77	Six national contractors -details not traceable
2.	Construction of 384 m-long reinforced cement concrete (RCC) walls on the banks	RCC wall	Not available	2010	20.0	
3.	Construction of gabion wall at Omchhu	Gabion wall	Upper & Lower of Omchu bridge	2009-2010	0.8	M/s Kamal Construction
4.	Construction of R.T works	RCC wall with Gabion wall	do	2011-2012	11.5	M/s Chapcha Construction
5.	Construction of R.T works	RCC wall with Gabion wall	do	2012-2013	5.1	M/s K.W Construction
6.	Construction of RCC wall for Omchhu	RCC wall with Gabion wall	Karbarytar	2014-2015	10.21	M/s TLK Construction
7.	Construction of RCC wall at Omchu & Amochu	RCC wall with Gabion wall	Near NWF camp	2015-2016	7.81	M/s Dhugdral Deyachen Construction
8.	Construction of R.T works at Kharaphu	RCC wall with Gabion wall	Kharaphu	2016-2017	8.16	M/s Dhugdral Deyachen Construction
9.	Construction of River protection works at Omchu	RCC wall with Gabion wall	Below Omchu Bridge	2016-2017	10.32	M/s TLK Construction
10.	Construction of Gabion wall on top of RCC wall at	Gabion wall	downstream of Omchu bridge	2016-2017	0.46	M/s Nehemia Construction

	Omchu River, Pling Thromde					
11.	Annual Maintenance Work			2018	NA	
12.	Construction of Gabion protection wall at Bokateybari	Gabion wall	at Bokateybari area	2019-2020	1.488	M/s Wangdhen Construction
13.	Construction of Gabion protection wall at Omchu River Opposite to NPPF colony	Gabion wall	Opposite to NPPF colony	2021-2022	2.1	M/s Tendrel Tshering construction
14.	Construction of Gabion wall at lower Kabreytar	Gabion wall	Near Bailey Bridge	2021-2022	0.287	M/s PD Ghishing Construction
15.	Construction of Omchu Treatment Plant, Khareyphu	Intake,Sand trap, flocculators, Sedimentation, RCC tank, Main Transmission etc.	Khareyphu LAP	2021-2022	37.875	M/s Kuenga Construction Pvt. Ltd. Paro.
16.	Construction of RCC wall below bailey bridge	RCC wall	Khareyphu LAP	2019-2020	1.01	M/s Nyinda Dradul Construction
17.	Construction of RCC wall below bailey bridge	RCC wall	Khareyphu LAP	2020-2021	0.69	M/s Wangs Pvt. Ltd

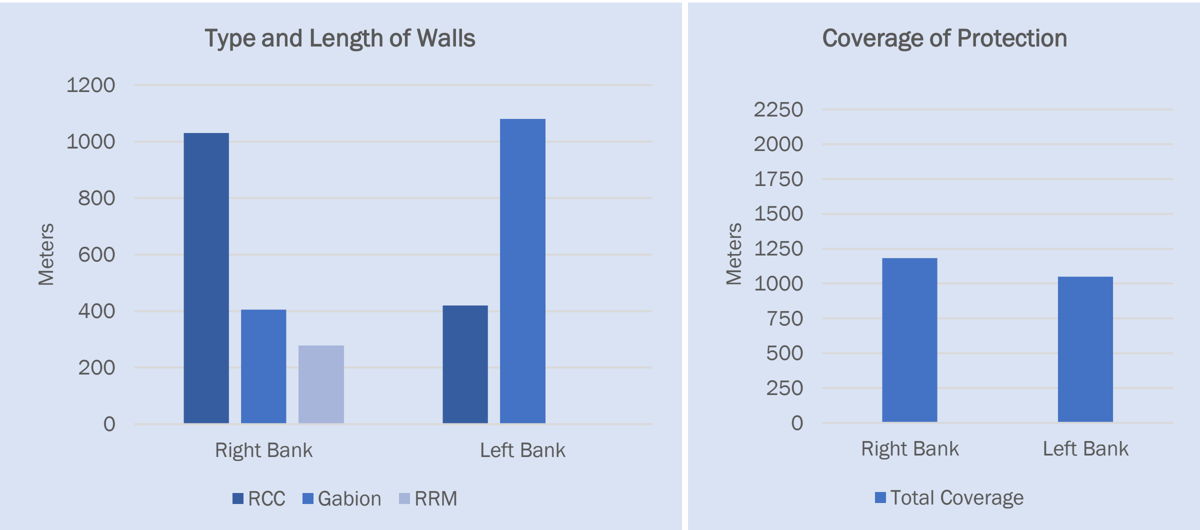
Source 11: DPR (2018) Gyaltshen Consultancy and Phuentsholing Thromde

2.7.3 Inventory as of 2022 March

As of March 2022, the inventory of structures included primarily three types of structures; Reinforced Concrete Cement (RCC) walls, Gabion walls, and Reinforced Rubble Masonry (RRM) Walls. The right bank had a total coverage of 1,183m, and 1,050m on the left bank. The right bank has more of RCC walls (1,030m), while the left has is dominated by Gabion walls (1,080m). The RRM walls is sparsely placed along the right bank.

Figure 16: Location of the flood protection structures (as of 2022 March)

Figure 17: Types and Length of the Wall Types



The general summary of the structure may be classifed as follows;

Classification	Description	Representational Photos
Not Damaged	• RCC and Gabion Wall Constructed at levels above HFL • Purpose is to protect the embankment • Not particularly affected by the floods, however, the structure itself isn't designed as flood protection	

Completely Damaged	• Gabion Wall of 3 steps • Longest stretch of gabion wall • Toe is all eroded • Could collapse anytime • Behind gabion is RCC wall acting as protection to the footpath/road • In some parts above the footbridge the debris (from failed walls) act as protection	
Structurally Damaged	• RCC Walls but all given away • 8 Panels of about 10 m each • All TMT is exposed • Toe is protected in most parts, but the concrete has been eroded and all TMT has been corroded	
Scouring in Progress	• Relatively new and constructed as a part of the Omchhu II Bridge Construction. • There are five panels. • No boulder pitching and directly exposed • Stagnation of water on footing. • Compressive Strength: 17 N/mm2	
Safe but detrimental to river discharge	• RCC Wall • Wall is protected against scouring with relatively sloped riverbed and vegetation • However, river bed level is increased by more than 2 meters resulting in less discharge capacity	

A detailed assessment of the entire stretch has been compiled as attached as a separate document.

3 Topographical Survey

3.1.1 Establishment of Control Points

Two level of control points were established. The first is of permanent nature to meet the higher order needs and to service the future needs of the project. The second was to meet the immediate needs of the survey at hand.

A control point bearing ID 12460 – a primary control point (PCS) in Feno Peg near Druk Green Power Corporation guest house was identified. The control point is in ideal proximity of the area of survey and has an all-encompassing view of the area in survey. This control point was the main and only reference used and required for this survey. This reference point has been used by the survey firm on two previous occasions for assignment pertaining to the Thromde and was found to be more stable and reliable as compared to some other points in the vicinity.

Id	Easting	Northing	Height	Remarks
12460	233313.0391	473025.7319	250.7744	Feno Peg

Static observation of at least an hour was carried out on each of the station. Four Trimble GNSS dual frequency receivers were used simultaneously. A base line processing method was adopted for computation of the new control points. The observation for the six control points was completed in two sessions. The observed data was post processed using Trimble Business Centre to meet the NLCS standard of secondary or tertiary level accuracy.

Some of the major consideration in selecting the location of control points are given below:

- The proposed site has clear visibility to the sky for standard GNSS receivers, away from obstructions such as houses, large trees, cliffs, deep valleys, etc.
- The site is located at a safe buffer from motorable roads keeping in mind the long-term security of the marks, which may be damaged or destroyed.
- Every control mark is visible to at least one other mark for orientation and performing checks during detail surveys.
- The chosen site is located on stable ground with minimum chances of deformations in the future, and preferably located on prominent spots that have commanding views of areas to be surveyed in detail.

At last, six control points of permanent nature(concrete) were constructed within the project area. The monument has a dimension of 30x30x60(cm) dimension with 10cm protruding above the ground surface. A non-corrosive metal rod was inserted in the center which shall represent the marker. The detailed location and co-ordinates are presented in Figure 18

3.1.2 Densification of control points

The survey teams carried out propagation of control points in the survey corridor. This was to basically establish a dense network of temporary control points and was used in the course of the topographical survey. The higher density of these control points minimized the need for lengthy traverses and provide for convenient avenues to close the traverse where necessary – thus reducing the probability of error propagation.

The control densification was carried out with Trimble R12i RTK the latest and highest configuration of RTK from Trimble to date. The accuracy threshold for these control points was set to a maximum 30mm in planimetry and 40mm in elevation – a standard setting for this kind of project. The actual accuracy achieve is normally much higher. The reference system was Druk Ref03 National Grid and vertical datum was Druk Geoid (EGM2008).

3.1.3 Detail survey

The detailed survey was commenced upon mutual acceptance of the accuracy of the control points by the client. All details existing in the corridor have been surveyed without fail. The feature/code table of the Thromde/MoWHS was followed. Any new features not included in the feature table was surveyed and given a provisional code. Spot elevations were surveyed with 3-5m density. Often the surveyors carried out denser data for depiction of small drainages patterns. Break lines were surveyed to represent any abrupt changes in the slope that is greater than a meter. Point density along curves and bends of features such as road and paths were picked up in higher density depending on the radius of the curvature and size of the feature.

Over head or suspended features such as bridges, major water pipes, sewer pipes extra which cross over the Omchhu were surveyed with their elevation which was set to 'non contourable'. However, telephone line, power lines do not have elevation attribute and will be shown, connected from pole to pole. All feature such as walls, and other forms of river protection infrastructure were surveyed as is indicating breaches where they exist.

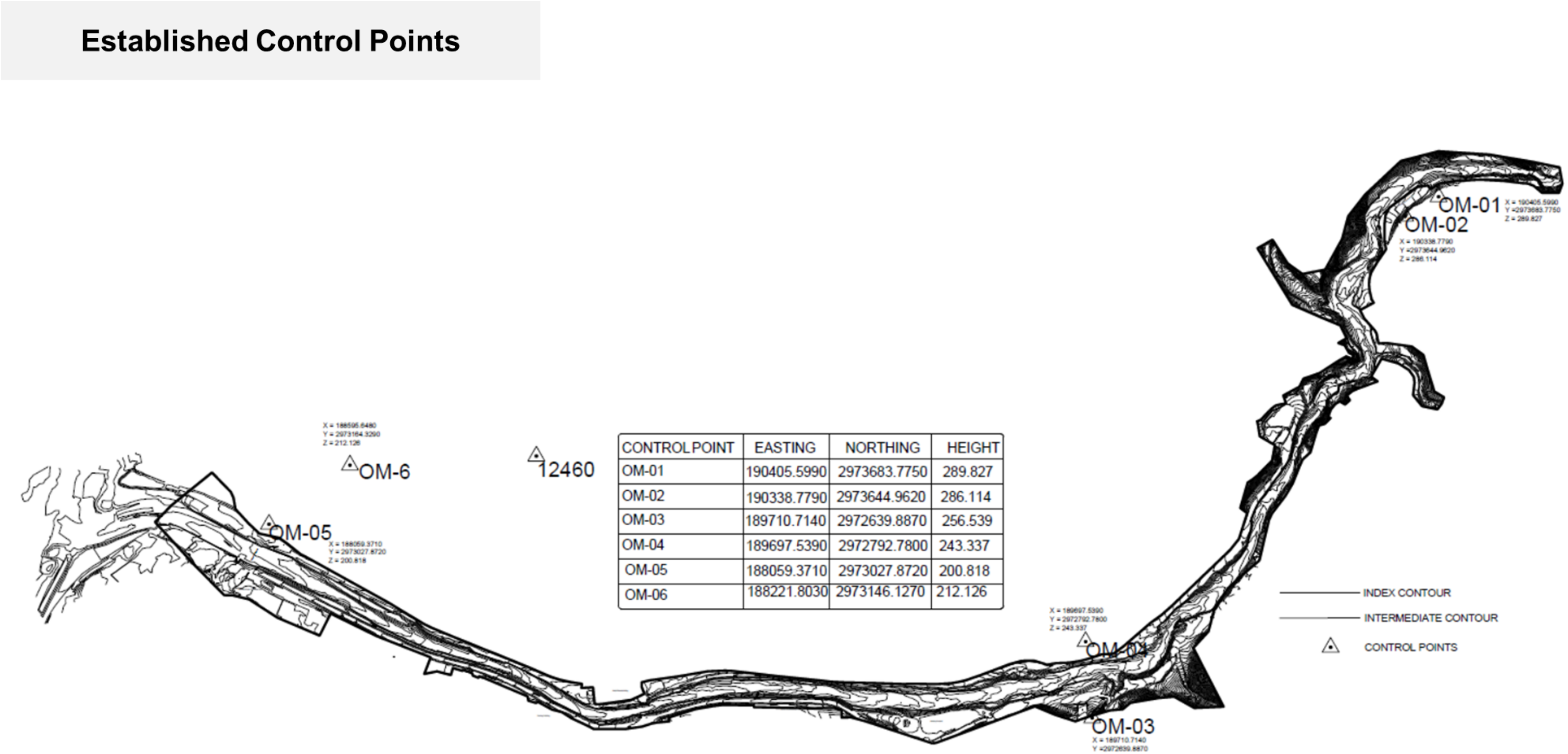
3.1.4 River surveys and survey of test location

Long profile and river cross sections along the Omchhu were carried out after consultation with the other experts and identification of the site of the transect. Survey of geotechnical test locations, unstable slopes were carried out in similar manner. Input from the domain experts (Geotechnical and Structural engineer) was sought to identify the HFL and normal water level especially in areas where the river bed has been disturbed due to human action and erased any visible signs of the levels.

3.1.5 Processing the data

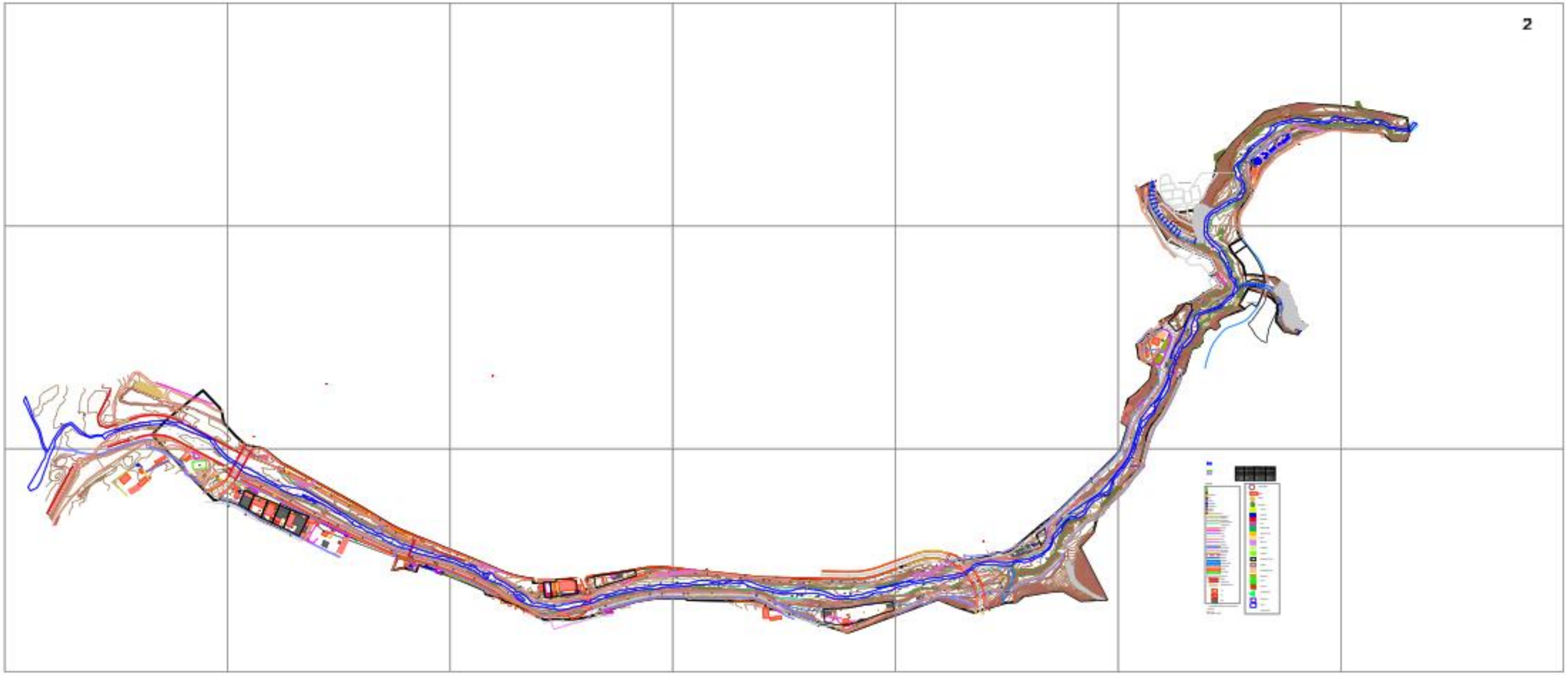
The surveyed data was downloaded on to the work station at the end of each day. The surveyor reviewed the day's work and edited and processed the data – creating map entities such as points, lines and polygons with appropriate features codes to segregate them into different classes. This data was then merged into larger base that included all the survey sections by all other surveyors. LISCAD was used for processing the survey data. The feature code ALPHA of the Phuentsholing Thromde provided by the client was adopted for the survey. The units of measure were in meters. The topographic map is presented in Figure 19.

Figure 18: Details of the Control Points established for future reference



CONTROL POINT	EASTING	NORTHING	HEIGHT
OM-01	190405.5990	2973683.7750	289.827
OM-02	190338.7790	2973644.9620	286.114
OM-03	189710.7140	2972639.8870	256.539
OM-04	189697.5390	2972792.7800	243.337
OM-05	188059.3710	2973027.8720	200.818
OM-06	188221.8030	2973146.1270	212.126

Figure 19: Topographical Map

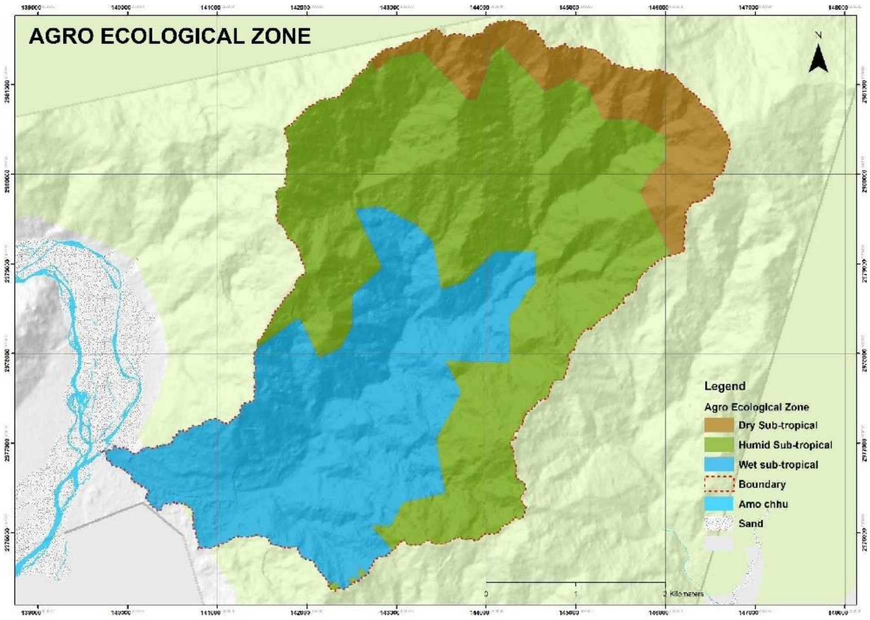


4 Hydrological Analysis

4.1 Climatic Features

In general, the climate in Bhutan varies according to latitude and altitude, the latter being a predominant governing factor. The climate and soils that determine the natural vegetation are generally classified as follows: the alpine tundra (above 3800 m), the cold temperate forest (3000-3800 m), the warm temperate forest (2000-3000m), the semi- humid subtropical forest (700-2000 m) and the humid subtropical forest (200-2000m) to the south where the effect of summer monsoon is most pronounced[6]. Majority (52%) of the Omchhu watershed of Phuentsholing Thromde/Dungkhag falls under the humid sub- tropical forest zone with pleasant but hot summer and warm winters. The others are Wet Sub-tropical (37%) and Dry Sub-Tropical (11%). The details are shown in the on Argo- Ecological Map of Omchhu basin.

Figure 20: Agroecological Map of the Watershed



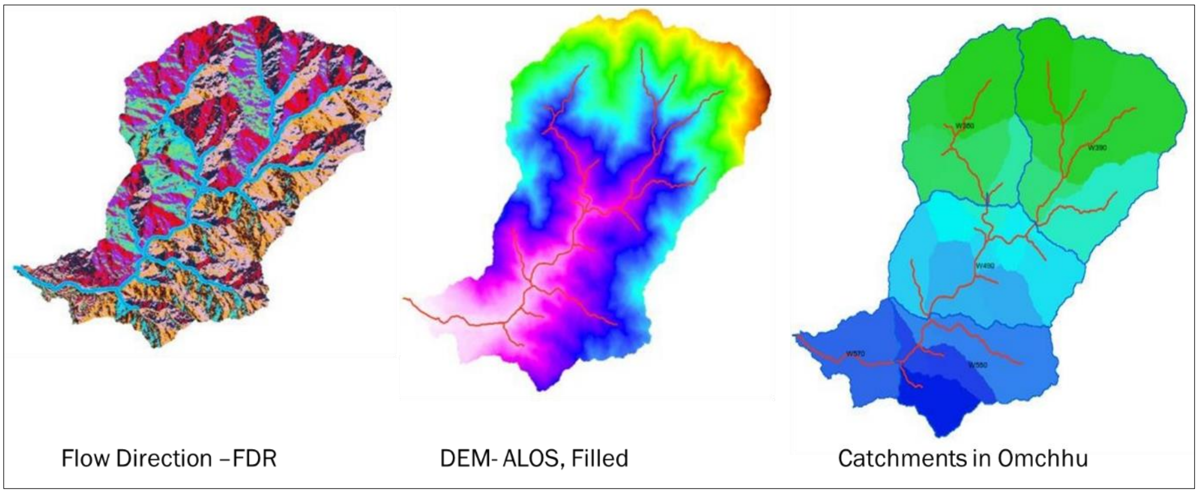
4.2 Watershed

An ALOS DEM of 10 m resolution was used for the processing of basin. It was received from Ministry of Economic Affairs[7]. A Digital Elevation Model (DEM) which is a representation of continuous elevation values over a topographic surface which was used for watershed delineation, terrain processing and Hydrological modelling in Hydrologic Modeling System (HEC-HMS) platform. The elevation of the Omchhu watershed ranges from 146 m in the south to 1725 m on the ridges.

[6] Technical Report- NAPA-2 Project Preparatory Grant

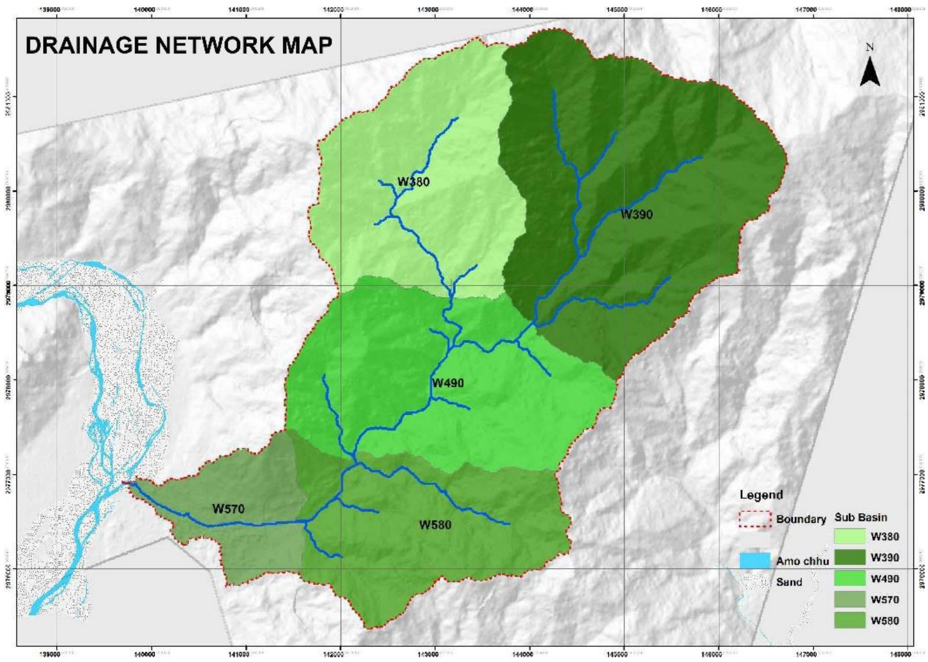
[7] ALOS DEM purchased by DGM through the SATREPS Project funded by JICA.

Figure 21: Sequential Processing of DEM



The total watershed area of Omchhu is 22.54 km^2 with total drainage length of 17,002 m and basin slope of 0.724 as shown in Figure 22. The basin is divided into 5 separate sub-basins for the hydrological analysis for better calibration and assessments as they may have different characteristics. The areas and their drainage lengths with slopes in each sub-basin are shown in Table 4

Figure 22: Drainage network map of the catchment



Source 12: DPR of Omchhu 2018, Gyeltshen Consultancy

Table 3: Length of main channel and tributaries

Sl.No.	Name	Length (m)
1.	Omchhu Main Channel	3,330
2.	Ramitey Khola	200
3.	Namantari Khola	180
4.	Kharaley Khola	84

Table 4: Details of Area, Slope and Lengths in the Omchhu watershed

Parameter	Basin 1	Basin 2	Basin 3	Basin 4	Basin 5	TOTAL
Watershed ID	W380	W390	W490	W580	W570	
Drainage Area, A (km2)	3.940	7.100	5.750	3.960	1.790	22.540
Drainage Length, L(m)	3,035	3,658	3,875	3,707	2,726	17,002
Basin Slope. (m/m)	0.877	0.941	1.152	0.586	0.064	0.724

4.3 Choice of Average Recurrence Interval (ARI)/ Return Period

Return Period which is based on the past occurrence of random events is used to predict the probability of an event in the future. It is also called the Average Recurrence Interval (ARI) by some groups for clarity. ARI is the probability or percent chance of an event occurring in any given year. For example, a rainfall magnitude corresponding to 100-year ARI should be interpreted as having a 1 in 100 chance or 1% chance of occurring in any given year. Since ARI is an average, a similar or even larger rainfall (or discharge) could occur this year, next year or any other year without any gap. As such, in Table 5 shows three common ways of expressing the frequency of an event.

Table 5: Three Common Ways of Expressing Frequency of an Event

ARI (years)	Probability of Occurrence in any given year	Percent Chance of Occurrence in any given year (AEP), Annual Exceedance Probability	Described As: (Event may be flood, extreme rainfall,..)
100	1 in 100	1%	100-year event
	1 in 50	2%	50-year event
	1 in 20	5%	and so on
	1 in 10	10%	
	1 in 1	100%	

Most Urban standards are generally designed to pass 25-years and 50-years ARI flood. In rural areas where settlement and traffic densities are low, reduced standards are also adopted. In river protection, major roads and flood embankments, it is necessary to have 100 years ARI flood. Table 6 contains the suggested design ARI that could be adopted as a standard.

Table 6: Recommended Design ARI

Source	Details	ARI Adopted
MOWHS	National Highway Design ARI Minor System Design ARI Urban Development Guidelines	Suggested best international practices
Based on Best Practices	Major Road Design - Urban Major Road Design - Rural Based on Development Category Major Road Kerb and Channel Flow Cross Drainage (Culverts) Minor Road Kerb and Channel Flow Cross Drainage (Culvert)	100 years 50 years 10 years 20 years 10 years
CDCL Project	ALDTP Project for Amochhu River	100 years with climate change

MOWHS	Phuentsholing Chamkuna Project- for side outfalls	50 years with climate change. Climate Change is used as 20% additional.

For the current project on the Omchhu, a 100 years ARI flood was selected with climate change to ensure that the best safety measures are provided since the risk due to flooding to lives and property is quite huge.

4.4 Climate Change Impact

All climate models for Bhutan predict that there is consistent warming pattern across the country, with greater changes projected for the winter season according to the ADB TA 8623 on Adaptation to Climate Change through IWRM (NECS 2016). For precipitation, most of the models predict increase in annual rainfall, although one model (ECHAM5) shows a decrease in the National Environment Commission's Second National Communication (NEC's SNC) for United Nations Framework Convention on Climate Change (UNFCCC). Practically all models agree on a projected rise in temperature. Three quarters (75th percentile) of the more than 40 GCMs used by the IPCC in its Fifth Assessment Report agree that the average temperature over Bhutan during winter (December to February) is likely to increase up to 1.5°C in 2016-2035, and by up to 3.0°C in 2046-2065 under RCP 4.5. During summer (June to August), the likely increase in temperature projected by three quarters of the climate models is by up to 1.0°C in 2016-2035, and by up to 3°C in 2046-2065 under RCP 4.5. The median of the projections suggests winter warming of 1.5 to 2.0°C, and summer warming of 1.0 to 2.0°C.

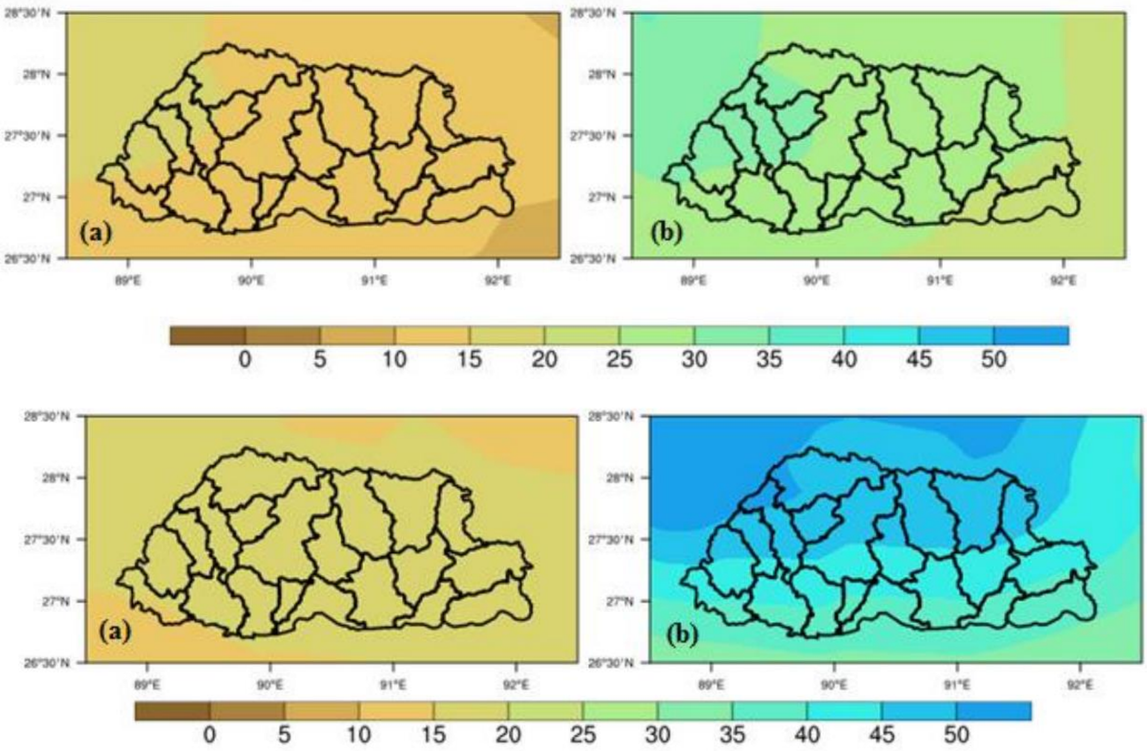
Regarding precipitation, the agreement among models is not as strong as for temperature but a general pattern of increasing rainfall is projected by most models as well. The Table 7 below summarizes model findings on precipitation which is derived from previous modelling studies in the region, and from the results of the climate modelling carried out under ADB TA 8623 on Adaptation to Climate Change through IWRM.

Table 7: Comparison of Modelling Findings on Climate Change Trend in Precipitation

Modelling study	Precipitation projection
2007 Asia-Pacific Network for Global Change, based on 11 GCMs not downscaled	Precipitation to increase in most parts of South Asia, with larger increases projected over the northeast region which covers Bhutan (up to 300 mm increase per year in the 2080s)
2010 Indian Network for Climate Change Assessment using PRECIS	Seasons were projected to become warmer by around 2°C toward the 2030s. All four Indian subcontinent regions examined were projected to experience an increase in precipitation in the 2030s, with the highest increase expected in the Himalayan region.
2011 Norwegian Water Resources and Energy Directorate, using ECHAM5	Projected a reduction in mean annual precipitation sums over future decades in the range from below 300 mm to zero.
2011 NECS regional climate modeling applied to Bhutan, using PRECIS	Total annual rainfall is projected to increase, with much wetter monsoons and slightly drier winters. SNC – Second National Communication to UNFCCC by NECS.
2012 ADB R-PATA 7423 using ECHAM5 downscaled using RegCM4	Precipitation changes in the 2030s is projected to be in the range of -3% to 4%; -1% to 6% by mid-century; and 0 to 1% in the 2080s; monsoon months predicted to become wetter and warmer

World Bank Climate Change Knowledge Portal, based on 9 statistically downscaled GCMs	Model ensemble shows an increase in rainfall during the monsoon/summer months (June to October) and a slight decrease during the winter months (November to March)
2015 ADB TA-8623 based on statistical downscaling of 2 GCMs based on APHRODITE historical rainfall data	Projected increase in annual rainfall across the country
2015 ADB TA-8623 based on statistical downscaling of 3 GCMs based on HYDROMET historical rainfall data	Except for the area around Samtse which shows a decrease in future annual rainfall, the annual amount of rainfall for the rest of the country is projected to increase
2019 NCHM Analysis of Historical Climate and Climate Projection for Bhutan based on ensemble of five global climate models of CMIP5	Under the RCP 4.5 scenario, the mean annual rainfall over Bhutan indicates an increase of about 10%-30% on the mean annual scale, with summer (JJAS) rainfalls between 5% - 15 %. Under the RCP 8.5 scenario, the mean annual rainfall indicates an increase of about 10% - 20% during 2021-2050 and with more than 30% increase all over Bhutan towards the end of the century (2070-2100).
2020 NECS Assessment of climate risks on water resources for the National Adaptation Plan (NAP) in Bhutan using down-scaled projections from a set of 5 GCMs from the NASA GEX-NDDP dataset were taken	For total precipitation an increasing trend was calculated for almost all Dzongkhags, however statistically not significant in most places. For the dry season most of the calculated trends show a decrease in precipitation over the past 40 years, though for most regions statistically not significant (for more information see Deltares 2021). The annual increase in precipitation over the climate projections has a mean of +23% (with 5th and 95th percentiles of +6% and 54%). All studied climate models project an increase in mean summer precipitation, with a mean of +25%. However, both an increase and a decrease in mean winter precipitation is projected, ranging from -38% to +57%.

Figure 23: (a) RCP4.5 (b) RCP 8.5 Change (%) in annual mean precipitation between future and present-day climates:



Source 13: NCHM (2019)- Analysis of Historical Climate and Climate Projection for Bhutan

With the exception of ECHAM5, the general pattern in climate change trends indicated by multiple models surveyed above including the use of two historical data sets (APHRODITE and HYDROMET) for the GCM downscaling suggests that rainfall over Bhutan is projected to increase, but with large spatial and temporal variations.

The projected increase in rainfall is spread evenly throughout the year. The increased rainfall is projected to occur during the wet season (monsoon), with mostly adverse consequences, whereas rainfall during the rest of the year would not increase, and may even decrease as suggested in some models. This would magnify river flows, causing floods and transporting more sediment and debris. The change in annual precipitation is taken into account for the hydrological analysis of the Omchhu with an increase in monsoon flow by 20%.

4.5 Hydrological Calculations

Hydrological calculations of the Omchhu basin were carried out with following methods to get the most suitable flood magnitude for the site.

4.5.1 SCS runoff curve number method

The SCS-CN model, developed by the United States Soil Conservation Service (SCS, 1972), estimates runoff volume (Qr) and runoff depth (Q) of individual rainfall events. The model is based on direct estimation of runoff, soil characteristics and land use, vegetation cover and antecedent moisture conditions (AMC). For the design of the hydraulic structures, the maximum design discharge of each of the watersheds has to be calculated. The SCS unit hydrograph method reflects how a catchment converts a hyetograph (a graph of the distribution of rainfall over time) into a hydrograph (a graph showing changes in river flow over time), while the SCS peak discharge method empirically relates peak flood flow to rainfall using land-cover-related parameters. The SCS unit hydrograph method can be used for most design applications including storage facilities. The SCS Unit Hydrograph is a useful tool to be applied. However, the watersheds are ungauged, so for some of the inputs it is uncertain which values are to be applied. To address these uncertainties, the Monte Carlo simulation method can be used. The Monte Carlo simulation is a technique that uses random or pseudorandom numbers in order to find a solution to a problem. It achieves an approximate solution of a mathematical or physical problem by simulating random quantities. The Monte Carlo algorithm, in general, consists of a process for generating a random event of some kind, then repeating this process an arbitrarily large number of times and averaging the results. The use of a method for solving models with a random element is a powerful tool in hydrology, as many components of a hydrologic system have inherent randomness.

In our case, a Monte Carlo simulation can be useful to assess the uncertainty of the output variable, being the maximum design flow, based on a random variation of the value of the input variables. For the calculation of the design flow from the watersheds, which are discharging to the Omchhu, we use the SCS Unit Hydrograph Method as is applied in the 'Enabling Climate Change Responses in Asia and the Pacific-Building Resilience to Disaster and Climate Change Impacts (Subproject 2), Climate Change Vulnerability Assessment and Adaptations' project, dated June 2013. For the latter project, the consulting hydrologist developed a spreadsheet comprising the modelling of the mentioned SCS method. This model uses one set of input parameters from which the values of a number of inputs may be uncertain. The input parameters of the SCS Unit Hydrograph model are listed in the next paragraphs. Thereafter we discuss the implementation of the Monte Carlo (MC) approach using the SCS UH method. Finally, we discuss the results of the MC simulation. The following properties are used from the watershed characteristics:

- Drainage Area, A (km^2)
- Main Drainage Length, L (m)
- Uppermost Drainage Elevation(m)
- Drainage Elevation at POI (m)

From these basin properties the Time of concentration, Tc, is computed, as well the time base of the rainfall event. The Time of concentration is the period after which the entire catchment area will start contributing to the runoff. The basin properties as mentioned above have been derived from the 10 m Digital Elevation Model (DEM) and have been assumed as fixed and have therefore not been used as random input variables. Of course, it can be discussed that using a different DEM would give different values for the basin properties.

4.5.1.1 Hydrological inputs

The hydrological inputs which are needed for the SCS UH method concern the following:

- Hydrological Soil Group
- Curve Number, CN

The Hydrological Soil Group defines the type of soils which can be found in the basin area. Depending on the soil type, the infiltration capacity as well as the retention may vary spatially. The Curve Number (CN), is in fact a container variable. Besides the Hydrological Soil Group, it is related to:

- Land Use or Cover, like: Fallow, Crops, Woods, Pasture, etc.;
- Treatment or Practice, like: Straight row, Contoured, Contoured and terraced, etc.;
- Hydrological Condition, like: poor, fair, good.

The Curve Number is presented as a number ranging 0-100. A value of 0 means that the total rainfall depth is stored in the soil and no run off will occur. A value of 100 represents a fully saturated and or impermeable soil which leads to the direct runoff of the total rainfall. Both the Hydrological Soil group and Curve Number are represented by the actual selected value of the Curve Number. Table 3-5 shows the hydrological soil groups while Table 3-6 shows the range of curve numbers which we have been using.

Table 8: Hydrological Soil Groups

HYDROLOGICAL SOIL GROUP	
A	Deep sand, aggregated loess aggregated soils, high infiltration (7.6 to 11.4 mm/hr)
B	Shallow loess, mixed with sandy loam, medium infiltration (3.8 to 7.6 mm/hr)
C	Clay loams, soils low in organic matter, usually high in clay, low infiltration (1.3 to 3.8 mm/hr)
D	Heavy plastic clays, certain saline soils, soils that swell when significantly wet, very low infiltration (0 to 1.3 mm/hr)

Table 9: Range of Hydrological Soil Groups and Curve Numbers

Curve Numbers (CN) for Hydrological Soil-Cover Complexes for Antecedent Moisture Condition Class II and Ia=0.2S (after SCS, 1972)						
Land Use or Cover	Treatment or Practice	Hydrological Condition	Hydrological Soil Group			
			A	B	C	D
Meadow	Permanent	Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads, dirt			72	82	87	89
Roads, hard surface			74	84	90	92
			Min→	Max→	55	89

We assumed that Soil Groups C and D may form the majority, but Soil Group B may also be present. This leads to a range of Curve Numbers between 55 and 89. This range is used as input to the Monte Carlo simulation. It should be noted that the maximum potential retention (S) and therefore the excess rainfall and runoff are very sensitive for the value of the Curve Number (CN). This can be seen from the relationship between the S and CN:

$$S = \frac{25400}{CN} - 254$$

4.5.1.2 Meteorological inputs

The driving force for the SCS Unit Hydrograph model is the rainfall event under consideration. Since we want to make an assessment of the design flows for the outfall channels/culverts, the rainfall event will be the design rainfall event for the area under consideration. The meteorological properties which are needed for the SCS UH method are the following:

- Design Return Period or ARI (years)
- Design Storm Duration (hours)

The hydrological assessment of the CC Vulnerability Assessment project[8] comprises a statistical analysis of the hydrology for the Southern Belt of Bhutan. This analysis yields the following rainfall depth values at different durations and return periods as shown.

Table 10: Design rainfall intensity at different durations and return periods under Climate Change conditions in 2050

DESIGN RAINFALL INTENSITY for 2050		under CC conditions				
ARI, T years ▶	2-year	5-year	10-year	20-year	50-year	100-year
Design Rainfall, by Gumbel EV I, mm/day ▶	240	274	299	321	351	373
Duration, D hours ▼	Rainfall for T-year return period, mm ▼					
1-hour	66.0	75.6	82.8	88.8	97.2	103.2
2-hour	91.2	104.4	114.0	122.4	134.4	142.8
3-hour	110.4	126.0	138.0	148.8	162.0	171.6
6-hour	152.4	174.0	189.6	204.0	223.2	236.4
12-hour	204.0	232.8	253.2	272.4	297.6	316.8
24-hour	240.0	273.6	298.8	321.6	350.4	373.2

The International Centre for Integrated Mountain Development (ICIMOD) has made an assessment on the Climate Change impacts in Bhutan. This assessment expects for 2050 an increase in temperature being 2.5 ° ± 0.4° C. The hydrological study of the earlier mentioned project also mentions that the extreme rainfall should be adjusted accordingly with the temperature rise. A percentage adjustment of 8 % increase per 1°C should be applied in case of a 50-year return period. As we have seen there is an uncertainty in the expected temperature rise. For the Monte Carlo simulation we vary between:

- 2.1- 2.9° C which means, using 8% per degree, a percentage range:
- 16.8- 23.2 % rainfall increase

We also do not know which storm duration will generate the maximum design flow. We already know that the maximum runoff will occur at the Time of concentration. This narrows down our range for storm durations. We have set the duration range at 0.5 to 2 hours.

Given the ranges for storm duration and the rainfall increase we can derive the rainfall depth using the formula[9]

$$\frac{P_t}{P_{24}} = sin\left(\frac{\pi t}{48}\right)^{0.4727}$$

Where:

t = specified time (in hours) for which the rainfall amount needs to be estimated

Pt = the rainfall in t hours

P24 = the total rainfall depth in 24 hours

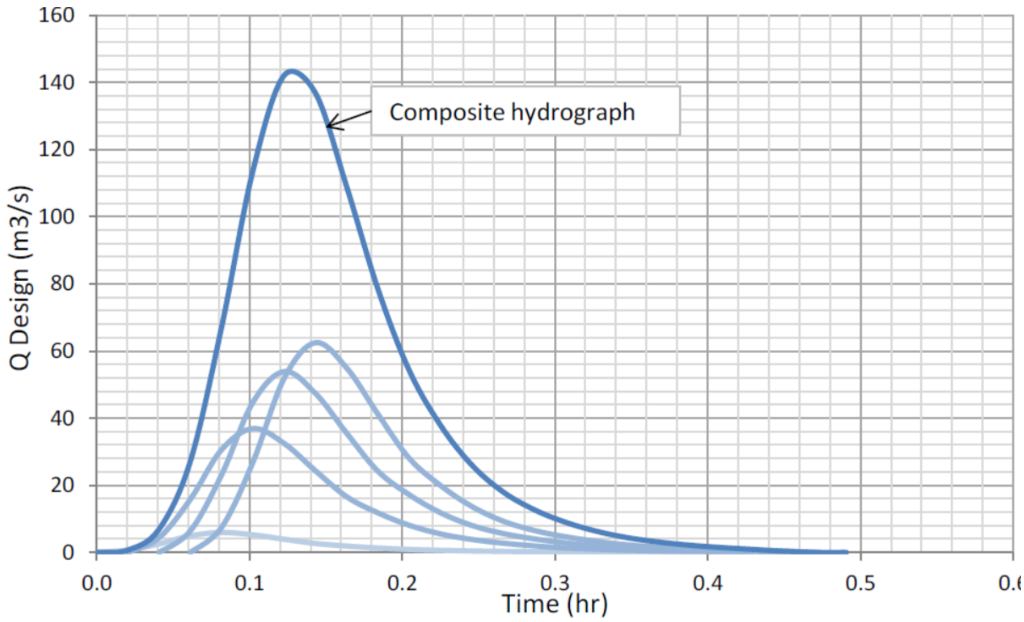
[8] Enabling Climate Change Responses in Asia and the Pacific-Building Resilience to Disaster and Climate Change Impacts (Subproject 2), Climate Change Vulnerability Assessment and Adaptations', June 2013

[9] Representative Rainfall Thresholds for Landslides in the Nepal Himalayas, Ranjan Kumar Dahal and Shuichi Hasegawa, Department of Safety Systems Construction Engineering, Kagawa University, Japan, Science Digest, 2 February 2008

The Monte Carlo simulation uses the P24 rainfall with variation in increase due to temperature rise and the variation in duration as inputs to formula 2. The result is a rainfall depth for the estimated duration, including the randomized climate change effect.

The result of the SCS UH method is an estimate of the rainfall excess translated into the direct runoff. This direct runoff can be multiplied with the dimensionless SCS unit hydrograph to generate the actual unit hydrograph. This will give a somewhat overestimation of the maximum discharge. A more realistic approach is to use a composite hydrograph which simulates the spatial distribution of the rainfall and the resulting runoff in the basin. This is done by splitting the excess rainfall for the selected duration into separate smaller storm events of 0.5 hours each and applying a time shift for each of the smaller events of 5% of the total time base. An example of such a composite hydrograph is shown in Figure 24.

Figure 24: Example of a composite hydrograph.



The time shift between consecutive sub-hydrographs is relatively small, which is due to the short time of concentration in the basins which are contributing to the Omchhu project area.

4.5.1.3 MC Simulation

To perform a Monte Carlo (MC) simulation, the SCS UH Excel spreadsheet was extended with macros comprising the actual calculation code, which enabled the looping process of the SCS UH simulation. To perform a sound statistical analysis, at least a population of 1000 output datasets is required. The MC simulation set up was executed by looping 10000 times, while varying the CN-number, rainfall duration and temperature increase between the adopted ranges using the Excel random generator. This generator assumes a normal distribution of the uncertainty as an input, so it is likely that the uncertainty of the output will also be normally distributed as displayed in Table 11. Given the fact that at each step in the looping process the uncertainties of each of the input variables are varied separately, we might get a good insight in the sensitivity of the uncertainty of the output variables for the input uncertainty. To assess this, we will be looking at different statistical characteristics of the output variables.

Table 11: Statistical variables used the outputs of the MC analysis.

Statistical variable	Remarks
Minimum	The minimum value of 10000 runs
Average	The average value of 10000 runs
Maximum	The maximum value of 10000 runs
10 % percentile (p10)	10 % of the observations lie below this value
50 % percentile (p50)	50 % of the observations lie below this value
90 % percentile (p90)	90 % of the observations lie below this value
Standard deviation (StDev)	Standard deviation

As output variables we are interested not only in the maximum design discharge but also at the values of the actual selected input parameters as well as the resulting runoff coefficient. The latter enables us to check the validity of the computed run off.

4.5.1.4 Results

For the analysis and derivation of the final design discharge of the Omchhu sub basins, we look at the P50 value of the statistical analysis of the 10000 simulation runs. To get an idea of the statistical bandwidth we also show the P10 and P90 values. From this method we find out that the design flows from the 22.5 km^2 watershed is 660.89 m^3/s. The results of the analysis are shown.

Table 12: Results of Peak Runoff Determination by SCS Unit Hydrograph (Monte Carlo Version)

Basin	Percentile	Curve Number CN	Max potential retention (mm)	Duration (hours)	% CC effect Rainfall depth	Rainfall depth (mm)	Q Design (m^3/s)	Run off factor (-)	Area (km^2))	Q/Area ($m^3/s/km^2$)
1.00	P10	36.00	48.38	0.50	17.44	69.40	12.90	0.01		
1.00	P50	60.00	169.33	1.50	20.05	113.89	139.06	0.20	3.94	35.29
1.00	P90	84.00	451.56	2.00	22.59	135.30	447.11	0.60		
2.00	P10	36.00	48.38	0.50	17.43	69.16	21.43	0.01		
2.00	P50	60.00	169.33	1.00	20.01	98.96	217.60	0.19	7.10	30.65
2.00	P90	84.00	451.56	2.00	22.55	134.57	736.66	0.59		
3.00	P10	36.00	52.02	0.50	17.47	69.25	18.07	0.01		
3.00	P50	59.00	176.51	1.00	20.07	98.97	180.97	0.18	5.75	31.47
3.00	P90	83.00	451.56	2.00	22.56	134.52	614.38	0.59		
4.00	P10	35.00	51.66	0.50	17.44	69.17	9.93	0.01		
4.00	P50	59.00	176.51	1.00	20.01	99.04	99.83	0.19	3.96	25.21
4.00	P90	83.10	471.71	2.00	22.54	134.50	337.37	0.59		
5.00	P10	35.00	52.02	0.50	17.42	69.09	2.39	0.01		
5.00	P50	59.00	176.51	1.00	19.95	99.06	23.43	0.18	1.79	13.09
5.00	P90	83.00	471.71	2.00	22.54	134.44	82.01	0.59		
TOTAL	P50	59.40	868.19	1.10	100.09	509.91	660.89	0.19	22.54	

4.5.2 SCS Peak Flow Method

The SCS Peak Flow Method is a simplified SCS procedure taken from the SCS Technical Release 55 (TR-55) which presents simplified procedures to estimate storm runoff volume. The selection of design rainfall duration and subsequent breakups into unit storms are very prejudiced in the absence of storm rainfall-runoff data. This simplified method is applicable to small drainages in urban drainages characterized by impervious soils due to buildings, parks, streets and roads, built-up impervious structures, etc. The determination of percentages of each of the land use zones is essential to determine the weighted CN. The following assumptions apply to the use of the simplified SCS procedure.

- The watershed is hydrologically homogeneous
- The watershed may have only one main stream, if more than one, the individual branches must have nearly equal time of concentrations
- The weighted CN must be greater than or equal to 40 and less than or equal to 98
- The watershed time of concentration must be between 0.1 and 10 hours.

The input requirements for this simplified SCS method are as follows:

Table 13: SCS Peak Flow Method

Parameters	Formula
Time of concentration (hr), Tc	$T_c = 0.0195L^{0.77}S^{-0.385}$, use 15 minutes if Tc is less than 15 min.
Time to Peak, (hr), Tp	Tp ≈ (2/3)Tc
Peak Discharge (m3/s per cm of excess rain), Qp	$q_p = \frac{2.08.A}{T_{peak}}$
Potential Maximum Retention, (mm), S	$S = \frac{25400}{CN} - 254$
Excess Runoff (mm), Ro	$RO = \frac{(P-0.2S)^2}{P+0.8S}$
Initial Abstraction	Ia = 0.2S
Rain Fraction	Ia/P
Select Coefficients for Type IA Rainfall for Ia/P ratio above	C_0, C_1,C_2 from Table G,
Unit Peak Flow (m3/s/km2/mm)	$q_u = 0.000431x10^{C_0+C_1 Log\, T_c + C_2 (Log\, T_c)2}$
Pond and Swamp Adjustment Factor	Fp, Select from Table H
Design Peak Discharge by SCS Peak Method (m3/s), Qpeak	$Q_{peak} = q_u A.RO.F_p$

Based on the analysis carried out according to the SCS Peak method, the Peak flow from the watershed is 249 m^3/s. the details are in Table 14.

Table 14: Results of Peak Runoff Determination By SCS Peak Flow Method

Sl.	Parameter	Abbre v.	TOTAL	Basin 1	Basin 2	Basin 3	Basin 4	Basin 5	UNIT
1	Drainage Area, A	A	22.54	3.94	7.1	5.75	3.96	1.79	km2
2	Main Drainage Length	L	17002	3035	3658	3875	3707	2726	m
5	Basin Slope	Slope	0.72	0.88	0.94	1.15	0.59	0.06	m/m
6	Drainage Time of Concentration, Tc	Tc	0.67	0.16	0.18	0.18	0.22	0.41	hr
7	Time to Peak, Tp ≈ (2/3)Tc	Tp	0.44	0.11	0.12	0.12	0.15	0.28	hr
8	Unit Hydrograph Peak Discharge, qp	qp	10.56	7.49	12.02	10.06	5.53	1.35	m3/s / cm of excess rain
9	Select Curve Number	CN	59.4	60	60	59	59	59	-
10	Potential Maximum Retention, S	S	173.61	169.33	169.33	176.51	176.51	176.51	mm
11	Assign Design Return Period, ARI	ARI	100	100	100	100	100	100	Years
12	24-hr Design Rainfall Depth for ARI specified in (7) above,	P	373	373	373	373	373	373	mm
13	Excess Runoff	RO	223.5	226.1	226.2	221.8	221.8	221.8	mm
14	Initial Abstraction	a=0.2 S	34.72	33.87	33.87	35.3	35.3	35.3	mm
15	Unit Peak Flow	-	0.05	0.07	0.07	0.07	0.07	0.06	m3/s/km2 /mm
16	Pond and Swamp Adjustment Factor	Fp	0.72	0.72	0.72	0.72	0.72	0.72	-
17	Design Peak Discharge by SCS Peak Method	-	191.2	45.74	81.11	64.71	43.04	17.15	m3/s
18	Climate Change Adjustment to Precipitation		8	8	8	8	8	8	%
19	Estimated Warming Projection based on Design ARI		3.8	3.8	3.8	3.8	3.8	3.8	0C
20	Estimated Rainfall Intensity Increase under Climate Change		30.4	30.4	30.4	30.4	30.4	30.4	%
21	Design Peak Discharge, under Climate Change - Q		249.4	59.64	105.77	84.38	56.12	22.37	m3/s

4.5.3 Rational Method

The most widely used un-calibrated equation is the Rational Method. This empirical method relates the peak discharge or runoff to the drainage area, the rainfall intensity and a runoff coefficient. The equation is:

$$Q = \frac{CIA}{3.6}$$

Where,

- Q = design peak runoff, m3/s
- C = runoff coefficient, (0 < C < 1)
- I = rainfall intensity in mm/hr for the design return period and for a duration equal to the "time of concentration" of the watershed
- A = Area of watershed, km2

The runoff coefficient, "C" represents the fraction of rainfall on a watershed that becomes surface runoff and this depends on integrated effects of several factors such as: (a) soil type, (b) vegetation, and (c) catchment size and slope.

The rainfall intensity, "I" in mm/hr. relates to the design return period and for duration equal to the "time of concentration" of the watershed. In order to design the design storm intensity for known duration and return period, some type of intensity-duration- frequency (IDF) data for the location of interest is necessary. The Rational Method is applicable to rural catchments up to 25 km2 but limited to 1 km2 in case of urban catchments due to complexity of land use.

In the Rational Method equation, Tc = time of concentration – time required for water to flow from the most remote point of the area once the soil has become saturated and minor depressions filled. It is assumed that when the duration of the rain storm equals the time of concentration, all parts of the watershed are contributing simultaneously to the discharge at the outlet.

Out of many methods used to estimate Tc, one of the simplest and most widely used is the Kirpich equation. The equation is:

$$T_c = 0.0195L^{0.77}S^{-0.385}$$

Where,

- L = maximum length of flow in m
- S = the watershed gradient in m/m, or the difference in elevation between the outlet and the remote part.

Based on the rational method of analysis, we the total peak flow generated from the Omchhu watershed is 292.62 m3/s. The details of their calculation are in Table 15.

Table 15: Results of Peak flow by Rational Method

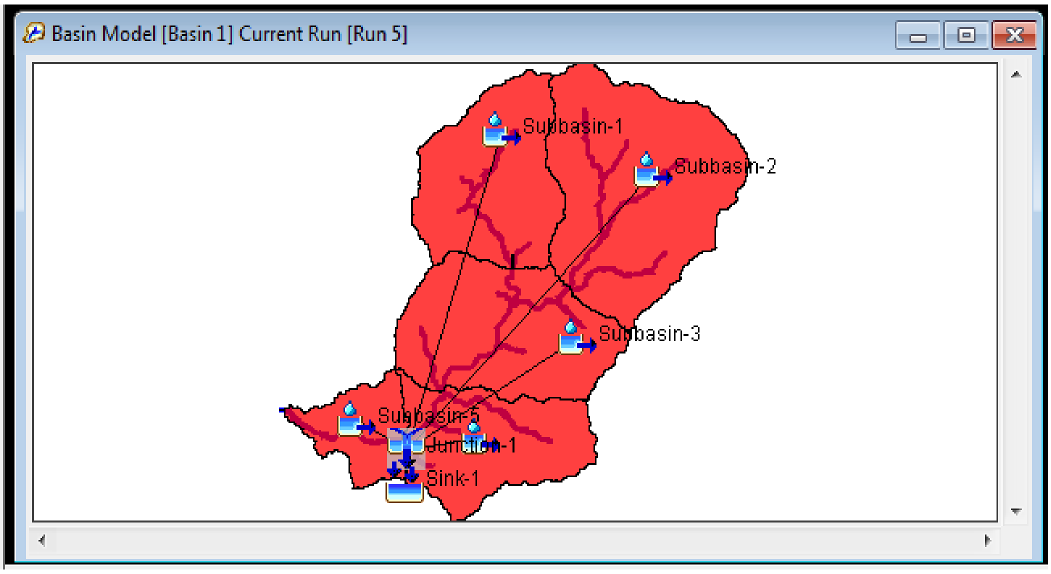
Parameter	TOTAL	Basin 1	Basin 2	Basin 3	Basin 4	Basin 5	UNIT
Drainage Area, A	22.54	3.94	7.1	5.75	3.96	1.79	km2
Main Drainage Length, L	17002.44	3035.25	3658.16	3875.25	3707.68	2726.1	m
Soil Group	B	B	B	B	B	B	0

Runoff Coefficent, C	0.28	0.21	0.21	0.89	0.35	0.48	0
Drainage Time of Concentration, Tc	39.	9.8	11.0	10.6	13.4	24.7	Min
Design Storm Duration, D	39.9	15	15	15	15	24.7	min
Design Return Period (ARI)	100	100	100	100	100	100	years
Design Rainfall Intensity,	128	128	128	128	128	128	mm/hr
Design Peak Discharge, Q	224.40	29.42	53.01	181.95	49.28	30.54	m3/s

4.5.4 Hydrological Modelling Method

A hydrological (rainfall-Runoff) model of the basin has been developed using the HEC- HMS Software. Detailed results of the hydrological model are presented in Appendix 2 Results from HEC-RAS Simulation. The basin schematic is shown in Figure 22.

Figure 25: Basin Schematic of the HEC-HMS Rainfall-runoff model



Model simulation runs have been carried out for three meteorological events:

- Hourly rainfall data of Phuentsholing real time station (2016)
- Triangular Hyetograph prepared for the maximum 24-hour recorded rainfall of 500 mm
- Daily Rainfall at Phuentsholing met station: data available from 1993 to 2016. But the simulation is carried out from June to September 2000 to capture the daily maximum of 495 mm observed on June 8, 2000.

4.5.4.1 Results

Table 16 shows the summary of simulated peak runoff at the basin outlet for the above three-time rainfall time series.

Table 16: Summary of simulated peak runoff at the basin outlet

Case	Description	Peak runoff (m3/s)
1	Hourly rainfall data of Phuentsholing real time station (2016)	251.20
2	Triangular Hyetograph prepared for a 100-year return period rainfall of 500 mm	206.50
3	Daily Rainfall at Phuentsholing met station	128.90

4.5.5 Flood Frequency Method

When peak daily streamflow or maximum 24-h total precipitation are arranged in the descending order of magnitude they constitute a statistical array whose distribution can be expressed in terms of frequency of occurrence. There are two methods of compiling flood peak data—the annual floods and the partial duration series. In the annual floods, only the highest flood in each year is used thus ignoring the next highest in any year, which sometimes may exceed many of the annual maximum. In the partial duration series, all floods above a selected minimum are taken for analysis, regardless of the time-interval, so that in some years there may be a number of floods above the basic stage, while in some other years there may not any such flood at all. The disadvantage of the partial duration series is that the data do not furnish a proper frequency (true distribution) series and so a reasonable statistical analysis cannot be made. These annual maxima for successive years can generally be considered to be independent and identically distributed, making the required frequency analyses straightforward.

The probability of occurrence "P" of a flood (having a recurrence interval T-yr) in any year, i.e., the probability of exceedance is: P=1/T, And the probability that it will not occur in a given year, i.e., the probability of non-exceedance, P' is: P' = 1-P. Table 3-16 below shows the maximum rainfall in Phuentsholing over the period of 1994- 2013. The maximum daily precipitation was in the year 2000 with 495.3 mm of one day rainfall. We believe that this is an outlier and has taken the next highest rainfall of the year which is 254.2. All the data has then been ranked as and analyzed.

The descriptive statistics of the maximum daily rainfall are shown in below. It clearly indicates that 495.3 is an outlier.

Table 17: Maximum Rainfall

Descriptive Statistics			
Statistic	Value	Percentile	Value
Sample Size	18	Min	97
Range	398.3	5%	97
Mean	186.19	10%	118.42
Variance	7165.3	25% (Q1)	131.97
Std. Deviation	84.648	50% (Median)	179.9
Coef. of Variation	0.45463	75% (Q3)	197.3
Std. Error	19.952	90%	254.46
Skewness	3.0568	95%	495.3
Excess Kurtosis	11.486	Max	495.3

Table 18: Daily Maximum Rainfall at Phuentsholing Ranked

Date	Precipitation (mm)
1/1/2000	495.3
1/1/1998	227.7
1/1/2007	212
1/1/2003	200
1/1/2002	197.2
1/1/1996	195.3
1/1/1997	194.5
1/1/2005	191
1/1/1995	190.9
1/1/1999	185
1/1/2004	174.8
1/1/2012	170
1/1/2013	167
1/1/2010	155
1/1/2001	132.6
1/1/2011	130.1
1/1/1994	125.5
1/1/2008	125
1/1/2009	120.8
1/1/2006	97

When the complete data set is analyzed, we find that Gumbel Extreme Value Type I is the best fit with a distribution shown as below in

Table 19. We find that that 100-year return period flood in this case, with probability of 1% is 450 mm on daily rainfall.

Figure 26: GEV Distribution

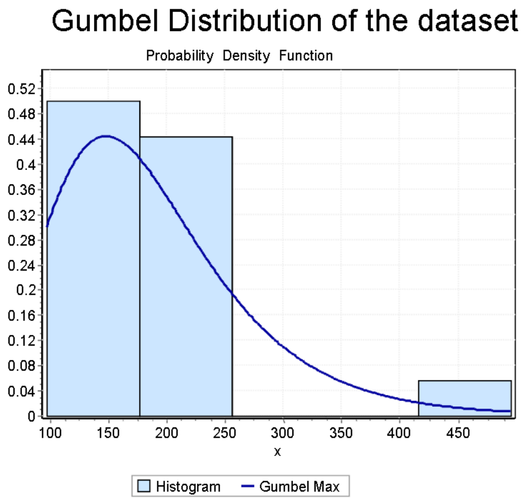


Table 19: Result of fitting Analysis with complete data

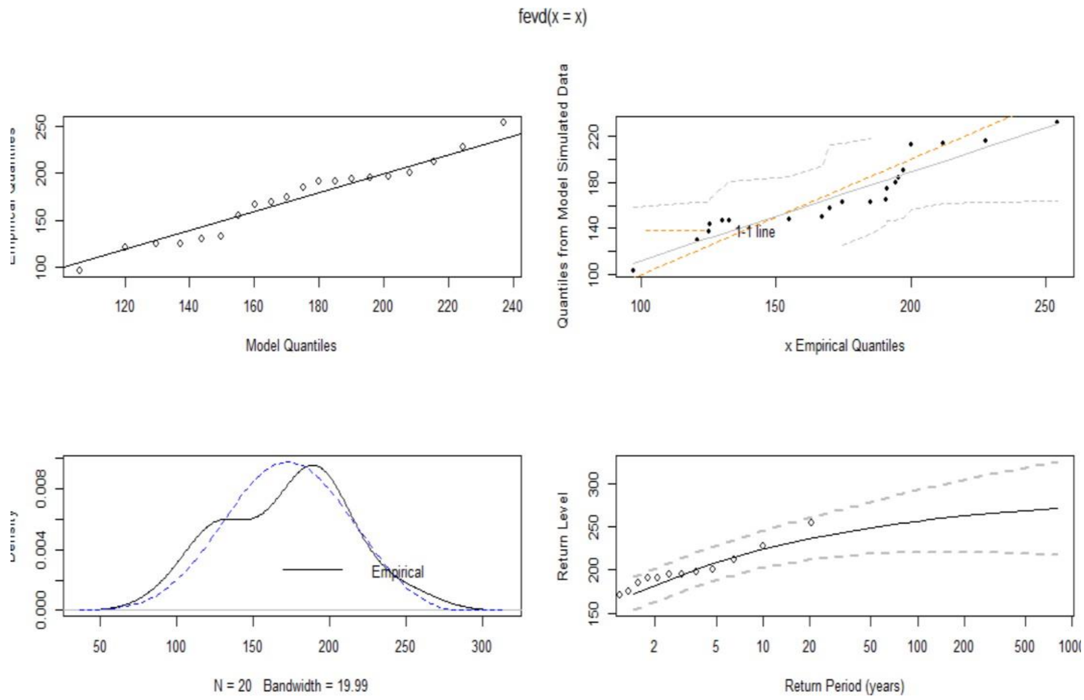
Rank	Distribution	Parameters
22	Gumbel Max	s=66.0 m=148.09
23	Gumbel Min	s=66.0 m=224.28
33	Log- Pearson 3	a=2.2161 b=0.23138 g =4.6487
53	Weibull (3P)	a=1.2183 b=96.791 g= 95.562

When the outlier of 495.3 is taken out we find that the most fitting distribution is GEV- General Extreme Value Distribution, followed by Log Pearson Type III. The 100-year return period rainfall from GEV distribution is 256.64 mm estimated, while from Log- Pearson Type III, the same is 252.807 mm as shown in Table 20. The distribution and statistics for GEV distribution are also shown in Figure 26.

Table 20: Distribution of Rainfall for Phuentsholing

GEV distribution	95% LCI	Estimate	95% UCI	Log Pearson Type III	Estimate
2-year return level	153.06	172.61	192.16	2-year return level	175.328
20-year return level	212.58	236.18	259.79	20-year return level	233.184
50-year return level	219.16	249.11	279.06	50-year return level	245.247
100-year return level	220.72	256.64	292.56	100-year return level	252.807
200-year return level	220.50	262.70	304.89	200-year return level	259.399

Table 21: Distribution diagrams of Rainfall for Phuentsholing[10]



Based on the above analysis we can choose a daily total rainfall of 256.64 mm as the 100-year return period rainfall for Phuentsholing for further analysis of discharge.

[10] Used R-studio Analysis Program and associated scripts

4.5.6 Results from Past Projects

As part of literature review and comparison, we also looked into various past report of the Omchhu watershed by other projects and studies. Table 27 shows the key assumptions, methods and their results.

Table 22: Results from past Projects

Sl. No	Project	Omchhu Discharge – m3/s	Remarks
1	Hydrology Modeling Using SCS-CN Method And HEC-HMS In Omchhu Basin. 2014.[11]	23.1	CN selected are 36, 35, 39, 63 and 64 with average of 47.5
2	Study Of Flood Embankment Deterioration of Omchhu, 2015.[12]	221.6	With Rational Method and Return Period
3	Northern Bypass Road Project[13] 2016.	784.79	Slope Are Method- Manning's Method. N-0.055, slope 6.566. Mean Scour Depth – 3.68m.

4.6 Flood Magnitude at other Probabilities

4.6.1 General equation of Hydrologic Frequency Analysis

The General equation of Hydrologic Frequency Analysis after Chow (1951) was used to ascertain the values of distribution of the flow in Omchhu. Considering that there are 10 peak flow observations, with a mean of 200 m^3/s, standard deviation of 106.5 m^3/s, the peak flow estimated for Omchhu is most appropriate as 660.4 m^3/s for 1:100 AEP; 582.0 m^3/s for 1:50 AEP and 503.2 m^3/s for 1:25 AEP as shown below;

Table 23: Results from Hydrologic Frequency Analysis

N	10		
yn	0.4952		
Sn	0.9496		
	for Q25	for Q50	for Q100
yt	3.198	3.901	4.600
K	2.846	3.587	4.322

Mean	Standard Deviation	Q25	Q50	Q100
200	107	503.2	582.0	660.4
230	100	514.7	588.8	662.3
300	85	542.0	605.0	667.4

[11] Hydrology Modeling Using SCS-CN Method And HEC-HMS In Omchhu Basin. Mr.Tshewang, Mr. Arun Gurung, Mr. Vasker Sharma , Ms. Ugyen Lhachey. Guide Mr. Leki Dorji, Department Of Civil Engineering. College Of Science And Technology, Phuentsholing, Bhutan, June 2014.
[12] Study Of Flood Embankment Deterioration Of Om Chhu. Ganja Singh Ghallay, Karma Yuden, Mahendra Subedi, Guide: Mrs. Monika Thapa And Mr. Tadashi Takahashi, Department Of Civil Engineering And Architecture College Of Science And Technology Rinchending: Phuentsholing, Bhutan, June, 2015.
[13] Detailed Design, Main Report. Procurement Assistance & Construction Supervision Consultancy Services For Construction Of Northern Bypass, Mini Dry Port & Allay Land Custom Station Under SACEC Road Connectivity Project (ADB Funded Grant-BHU-0400). Final Detailed Project Report, Northern Bypass Road, Phuentsholing, , October 2016. Gyeltshen Consultancy and SMEC International.

4.6.2 Gumbel Distribution from Rainfall Data

Gumbel distribution is found to be the most appropriate distribution for Bhutanese rivers[14]. The Gumbel distribution analysis was carried out in MS excel for the maximum rainfall data over the last 23 years[15] (1996-2018) and plotted. Based on the distribution analysis, the 1:100 AEP rainfall is 460 mm for which we know the flood estimated is 660.4 m^3/s. Using the same ratio, the 1:50 AEP is estimated as 599.74 m^3/s and 1:25 AEP flood is calculated as 545.2 m^3/s for as shown in table below.

Table 24: Result from Gumbel Distribution and ratio

AEP	Q (m^3/s)	Rainfall from Gumbel Distribution
1:100	660	460
1:50	599.74	418
1:25	545.22	380

The results of the Gumbel distribution are plotted in figure below, while the subsequent table has the actual details of the maximum annual rainfall and Gumbel distribution parameters.

Figure 27: Probability Analysis by Gumbel Method

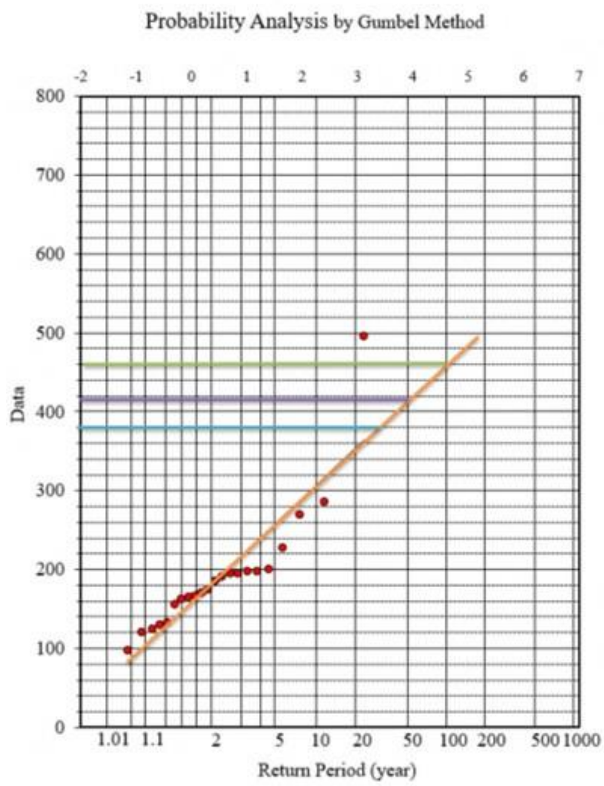


Table 25: Phuentsholing Maximum Daily Rainfall

Annual max. data		Ordinal data			
Year	Data	No.	Year	Data	Plotting P.
1996	195.3	1	2006	97.0	-1.142787
1997	194.5	2	2009	120.8	-0.892959
1998	227.7	3	2008	125.0	-0.711420

[14] ADB CDTA 8623: Adaptation to Climate Change from Integrated Water Resources Management. ADB and NECS, 2016. River Basin Modelling Report.
[15] Missing data for 2014, so in fact only 22 years data were analyzed.

1999	185.0	4	2011	130.1	-0.559158
2000	495.3	5	2001	132.6	-0.422687
2001	132.6	6	2010	155.0	-0.295453
2002	197.2	7	2013	162.5	-0.173604
2003	200.0	8	2016	165.4	-0.054538
2004	174.8	9	2015	167.0	0.063718
2005	191.0	10	2012	170.0	0.182831
2006	97.0	11	2004	174.8	0.304355
2007	197.6	12	1999	185.0	0.429879
2008	125.0	13	2005	191.0	0.561163
2009	120.8	14	1997	194.5	0.700299
2010	155.0	15	1996	195.3	0.849932
2011	130.1	16	2002	197.2	1.013613
2012	170.0	17	2007	197.6	1.196399
2013	162.5	18	2003	200.0	1.405997
2014		19	1998	227.7	1.655193
2015	167.0	20	2018	270.0	1.967815
2016	165.4	21	2017	285.4	2.397206
2017	285.4	22	2000	495.3	3.113351
2018	270.0				
N =	22	18			
X_{mean} =	192.6909	39.0944			
S_x =	79.35737	17.2858			
C_v =	41.18%	44.22%			
a =	0.040358	0.060632	Sy(N)/Sx		
X_o =	94.74573	30.52143	X mean-1/a*Y mean(N)		

Flood Magnitudes on Thimphu Chhu based on observed flows

There are no stations in the south of Bhutan with reliable hydrological data and flood frequency analysis. A recent analysis done for the Thimphu Chhu had been used to compare the ratio of 1:100 AEP flood to that of 1:50 AEP and 1:25 AEP flood and determine the corresponding flows for Omchhu using the same ratio. Based on the same ratio using the Gumbel distribution flood for 1:100 AEP of 660 m^3/s, the 1:50 AEP is estimated as 575.2 m^3/s and the 1:25 AEP flood is calculated as 489.7 m^3/s, as shown in table below. It may be noted that the catchment of the Thimphu Chhu at Lungtenphu is about 663.0 sq.km in the northern Bhutan at altitude of above 2000 MASL, while the Omchhu catchment in the southern foothills of 100-200 MASL. is about 22.5 sq.km.

Table 26: Flood Magnitudes on Thimphu Chhu based on observed flows (m3/s)

Station	Gumbel Distribution		
	25 yr	50 yr	100 yr
Lungtenphu	379.95	446.24	512.04
Ratio	0.742032	0.871494	1
Estimate for Omchhu	489.741	575.1863	660

4.7 Selection of Discharge for the Omchhu

Based on the above hydrological analysis we got various flow figures. Choosing a 100-year return period flow with climate change impact the flows for the Omchhu ranges from 224.4 m^3/s using Rational Method to 784.79 m^3/s from the Northern Bypass Road Project as shown in Table 27.

Table 27: Results from different methods.

Sl. No	Project or Method	Discharge – m^3/s
1	100-year return period flood with SCS CN Method using Monte Carlo Simulation	660.89
2	Hydrological Modelling in HEC HMS	251.0
3	SCS Peak Flow Method	249.0
4	Rational Method	224.4
5	Northern Bypass Road Project 2016.	784.79

We have selected 660.89 m^3/s of flow from SCS CN Method using Monte Carlo Simulation as the ultimate maximum flow for 1:100 AEP. We believe that this flow will also take into consideration the sediment loads, fluctuations in the flows and errors in any estimation considering it as a worst possible scenario. The Northern Bypass Road Project flow analysis was only done for the bridge and not suitable for our case.

The other flood magnitudes chosen for this study area are 582.0 m^3/s for 1:50 AEP and 503.2 m^3/s for 1:25 AEP. These values have been adopted since the SCS runoff curve number method is already based on extremely high estimates and the one from Thimphu Chhu is on lower side. For the modelling purpose, these flows are just taken as PF1, PF2 and PF3 since the determination of the return period is not reliable due to lack of data.

5 Hydraulic Study[16]

In our case we have used HEC-RAS (Hydrologic Engineering Center's River Analysis System) for the assessment of flood risks in the Omchhu basin. The results from the hydrological analysis from the DPR study were used as the flood magnitudes. A peak flow of 660.89 m3/s from SCS CN Method using Monte Carlo Simulation as the maximum flow has been selected as a 1:100-year return period flow with climate change impact scenario. Both 1D and 2D models were prepared and analyzed for the hydraulic study.

5.1 Hydraulic Modelling Plan

The components to set up the model run plan for the assignment are as below;

5.1.1 Model geometry

i. Current conditions (With new Omchhu bridge, Northern-by pass Bridge, and new topographic and bathymetric survey of March 2020), and
ii. Future conditions with PTDP proposed infrastructures and riverbed levels of the new Omchhu bridge.

Figure 28: Hydraulic model extent (Source: Google Earth image)

5.1.2 Boundary conditions

i. Inlet discharge: three flood events; represented in models as PF1, PF2, and PF3 respectively.
ii. The downstream boundary drawn from the PTDP study from which the maximum water surface level was used for the 1 in 10-year AEP on the Amochhu as a fixed water level. This is assumed to be the Frequent Flood Level on the Amochhu (PTDP, 2020).
iii. A future profile condition with a detailed proposed bed level of the new Omchhu bridge and proposed levels of structure from the Omchhu bridge to the confluence of the Amochhu/ PTDP were also modelled.

Knowledge of the upstream sediment flux is poor. It is assumed that the bed level at the upstream sediment boundary in the model is fixed, i.e., the bed is in equilibrium and there is no net erosion or deposition.

[16] This document contains only the relevant aspects of the hydraulic study from the report on ADB TA-9781 REG: Developing South Asian Liveable Cities Facility 17/81- Survey, Hydraulic Study and Updating of Hydraulic Model for Omchhu River. Readers are advised to refer the main document for in-depth details.

5.1.3 Simulation Settings

This includes running the model with different Manning's roughness[17]. It will help test the sensitivity of the channel roughness assumptions. The Manning's to be checked are 0.035 and 0.05 for winding natural streams with weeds and mountain streams with rocky beds respectively[18].

The model runs covering 2 model geometries, 3 flow scenarios, and 2 roughness values, Manning's n=0.035 and n=0.05. The runs, which are necessary for the sensitivity analysis are defined separately and based on the outcomes of the first set of runs as listed. Both the sensitivity analysis as per sediment transport capacity and the variations in model geometry and boundary conditions gives us a better understanding of the geomorphology in the downstream Omchhu reach.

Table 28: Basic Model Run Plan[19]

Model Run Plan for Omchhu	A			B			C		
Run ID	1	2	3	1	2	3	1	2	3
Model Geometry									
Current conditions	X	X	X						
Future conditions	X	X	X						
Boundary conditions									
1:10 FFL level at Amochhu				X	X	X			
1:100 HFL level at Amochhu				X	X	X			
Simulation Settings									
Manning's n=0.035							X	X	X
Manning's n=0.050							X	X	X

5.2 Hydraulic Modelling in HEC RAS

Hydraulic modelling was set up using the 10 m resolution DEM of the river in the HEC-RAS software in addition to the cross-section survey details.

5.2.1 Model Set up

For the present study, HEC- GeoRAS extension in GIS was used to create the geometry file using the 10 m resolution DEM. A key plan for the area along with indicative chainage is in Figure 29. A 1D steady flow model was set up in HEC-RAS assuming that there is a single flow direction, that flow varies gradually and that there are no significant storage areas or lakes within the Omchhu riverbed, which may introduce 'attenuation' effects thus requiring unsteady flow simulation. Hydrological data from the hydrological analysis was used to model the river flow. The flows selected are as below;

Table 29: Flows used for Hydraulic Modelling

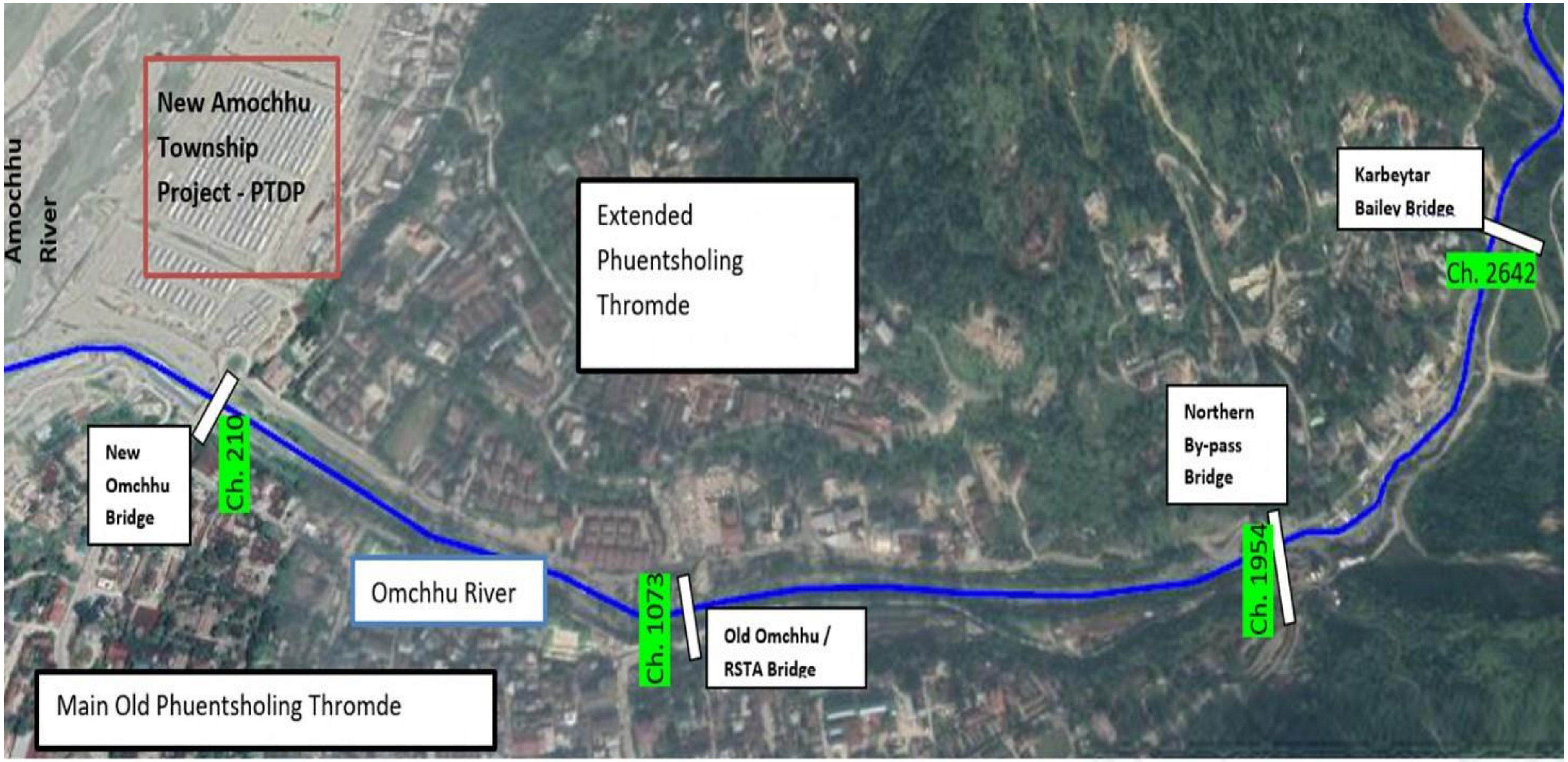
PF	Q (m3/s)
PF1	503
PF2	582
PF3	660

[17] Note that roughness in 2D models is typically less than that used for 1D models because the 2D model accounts for much more detail, in particular locally higher velocities, which have to be lumped into the resistance number of the 1D model

[18] http://www.fsl.orst.edu/geowater/FX3/help/8_Hydraulic_Reference/Mannings_n_Tables.htm

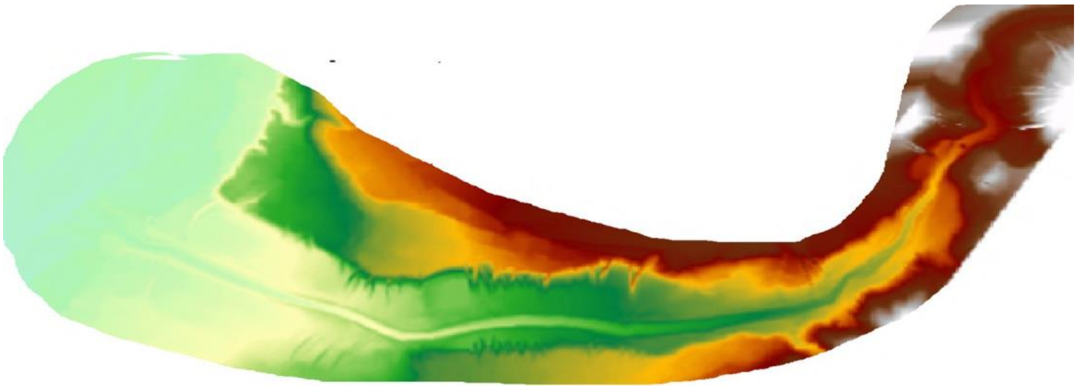
[19] For instance, A1- is old geometry without bridge for PF1; A2 is old geometry without bridge for PF2. 1,2,3 are for different flows.

Figure 29: Overall key plan of the Omchhu



The survey data along with the ALOS DEM was used to generate the TIN file for processing in GIS as shown in Figure 30. From an earlier project by Phuentsholing Thromde, the ALOS[20] DEM has been applied to set up the model geometry and derive cross sections where additional model information is needed. Further correction and adjustments had to be made due to DEM leaking at some sections. It was overcome by creating levees at the lower sections.

Figure 30: DEM 10m resolution



5.3 Hydraulic Model Results

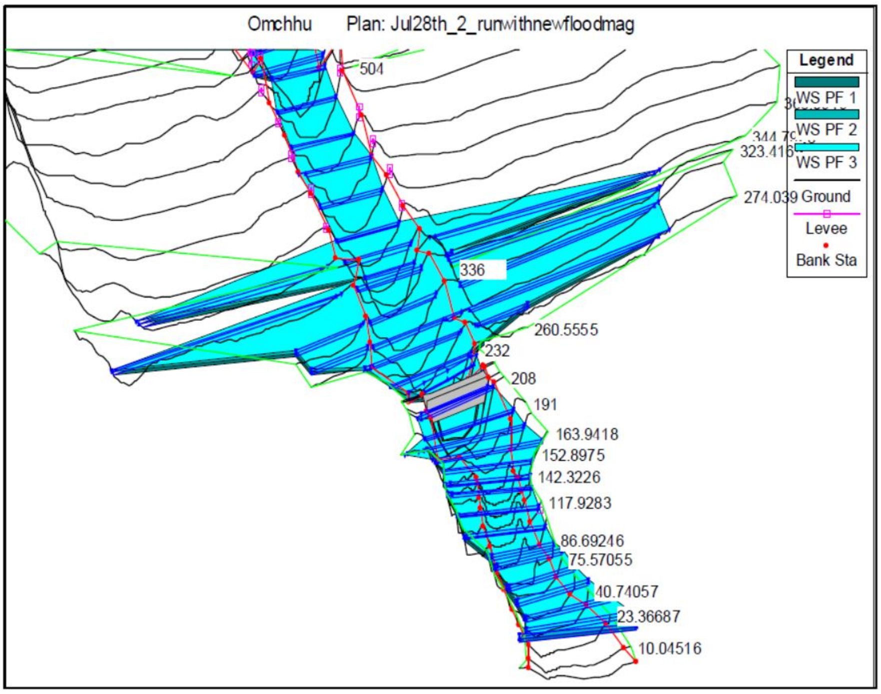
5.3.1 Current Condition

The flooding extent and depth for scenarios PF1, PF2, and PF3 are shown in the figures below. Embankment flooding can be observed under all the flood conditions mentioned above around the new Omchhu Bridge between chainage 260- 340.

Figure 31: Flooding depth (PF1- 503.2 m³/s)

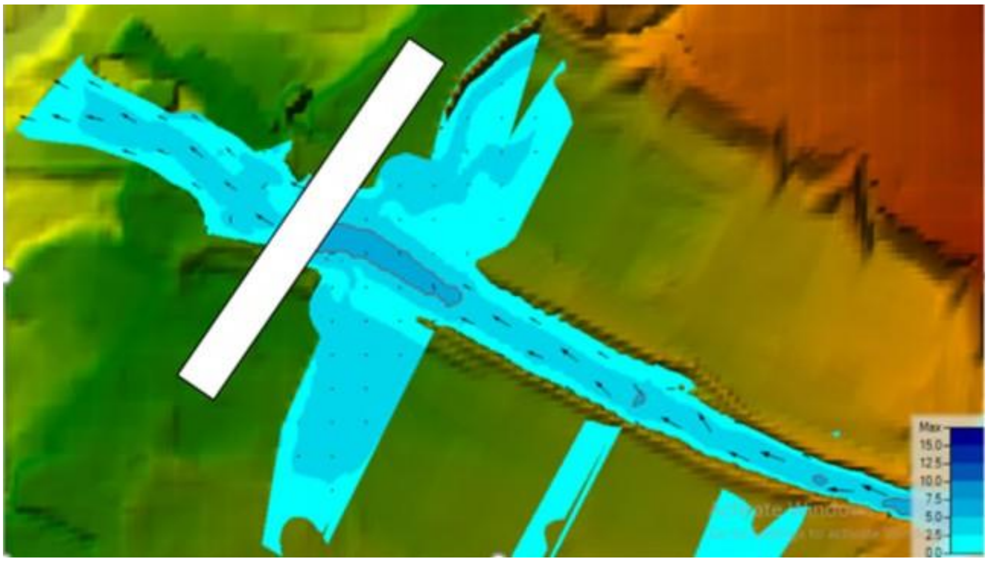
[20] Advanced Land Observing Satellite (ALOS), also called Daichi (a Japanese word meaning "land"). Data provided from DGM-SATREPS Project for Government of Bhutan by JAXA.

Figure 32: Flooding under different PFs from 1D-model



The zoomed-in velocity and extent of flooding at the new Omchhu bridge is shown in Figure 33 below.

Figure 33: Flow extent and Velocity under PF3 at New Omchhu bridge area (m/s) from 1D model.



5.3.1.1 Model Sensitivity

To demonstrate model sensitivity to selected model inputs/parameters including the downstream boundary condition, model runs were performed with different Manning's n value to check the river flow sensitivity to the Manning's roughness coefficient. Although the effect of the bed friction is assumed to be negligible, while increasing the river bed friction to 0.05 and flood plain value to 0.07, there is a slight increase in the flow through the channel and an increase in the backwater at the new bridge. This is shown in Figure 34 and Figure 35.

Figure 34: PF 3 _ Mannings 0.035 river bed and 0.05 flood plain

Figure 35: PF 3 _ Mannings 0.05 river bed and 0.07 flood plain

5.3.2 Future conditions with PTDP

The model geometry was edited to simulate for future conditions with the PTDP proposed infrastructures at the downstream end of the Omchhu reach. This level is based on the information provided by PTDP as per the design level of the proposed diaphragm walls from Amochhu-Omchhu confluence up to the new Omchhu bridge. The riverbed level of the new Omchhu bridge near the YDF is proposed to be lowered by around 1.75 m from the existing 194.25 m to 192.5 m. The proposed bed slope from the confluence up to the Omchhu bridge is 1:74. The avoid a sudden drop at the bridge foundation, the river bed further upstream of the Omchhu bridge up to the pedestrian bridge was also lowered in the model by the slope of 1:74. The model geometry and profile are as shown in Figure 36 and Figure 37.

Figure 36: Plan of the new geometric section with the PTDP walls and new river bed level

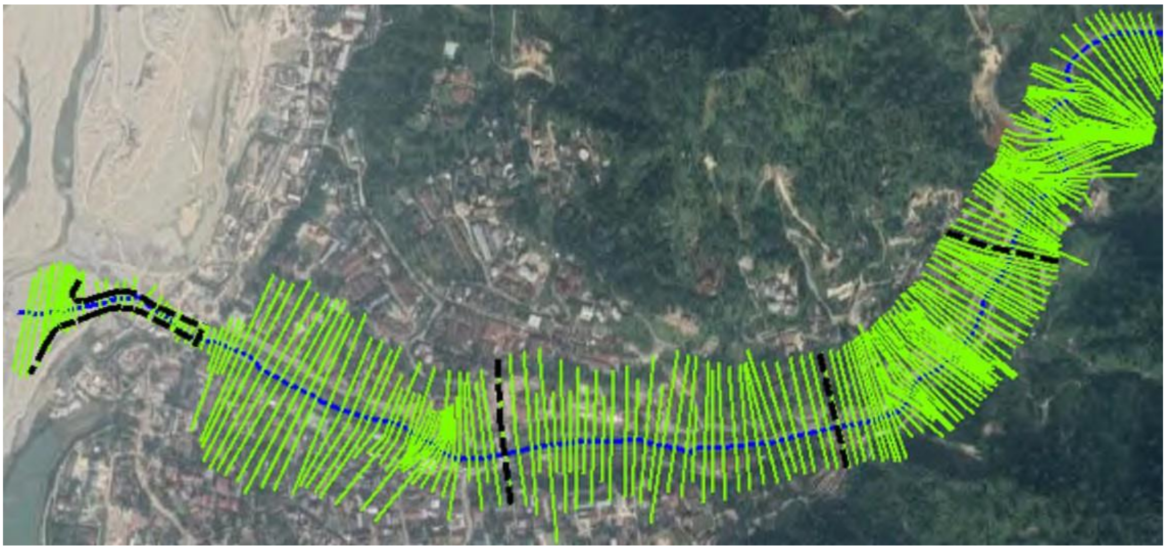
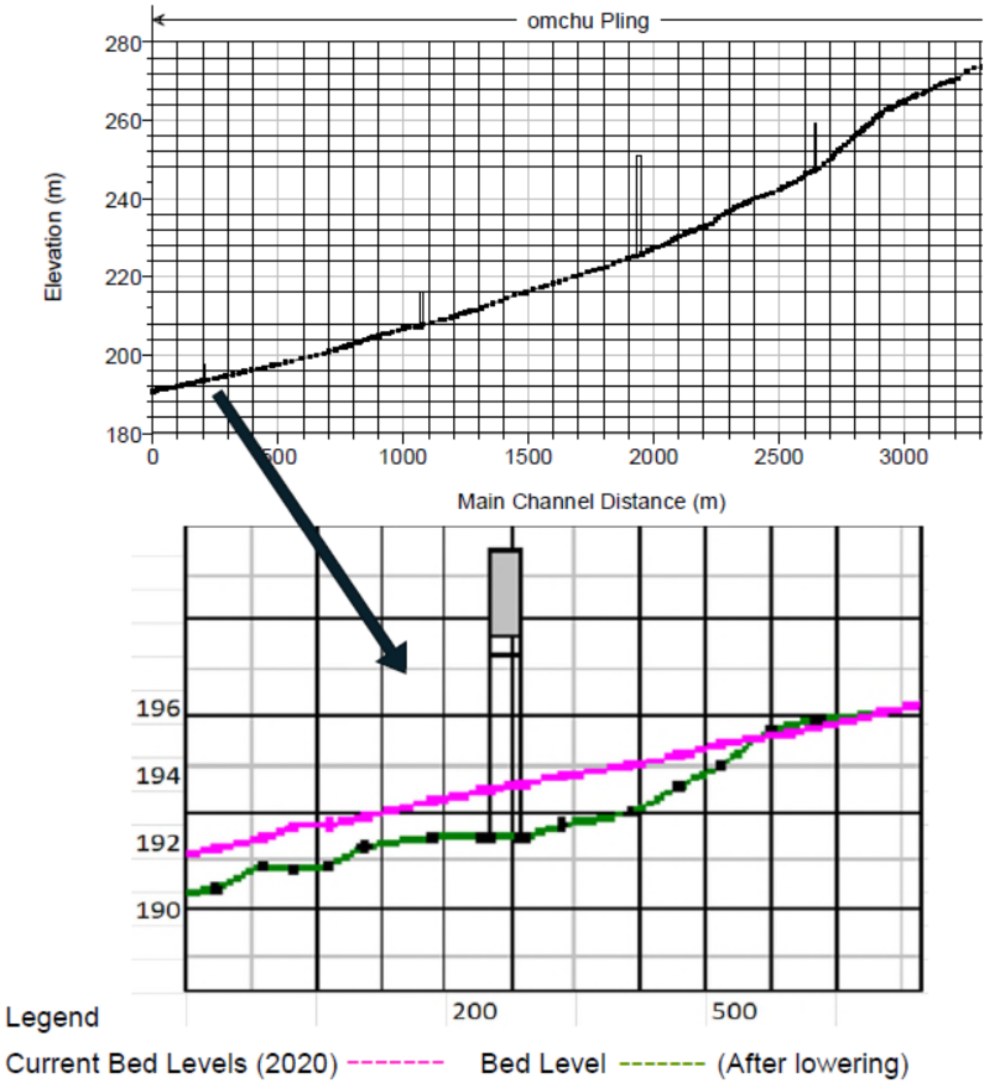


Figure 37: Profile of the new geometric section with the PTDP walls and new river bed level

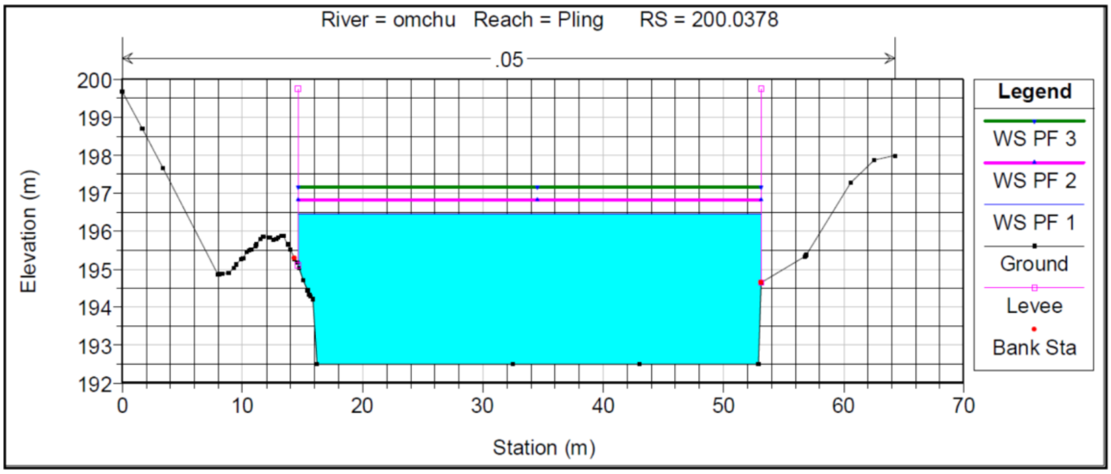


The maximum flow depth and extent under PF3 flood and future conditions with the proposed PTDP infrastructure are shown in Figure 38. According to this bed lowering, there is no backflow or overflowing under any circumstance once the new Omchhu bridge is lowered and the bed sloped maintained.

There is no noticeable overbank flooding under any of the flood situations. Sample cross-sections are shown for chainage 200 (Figure 39), downstream of Omchhu bridge, and chainage 592. There is a hydraulic jump at the new Omchhu bridge under PF 2 and PF3 as shown in Figure 42. The details of the water level under different flood scenarios are in Appendix 1.

Figure 38: Flood Depth and extent under PF 3 with PTDP walls and lowered river bed level

Figure 39: Cross-section of flooding at Ch. 200 just below the new Omchhu bridge with future sections and levels[21]



[21] Note that the channel cross section at the bridge has been edited to the 'design' level.

Figure 40: Cross-section of flooding at Ch. 592.3 with future sections and levels

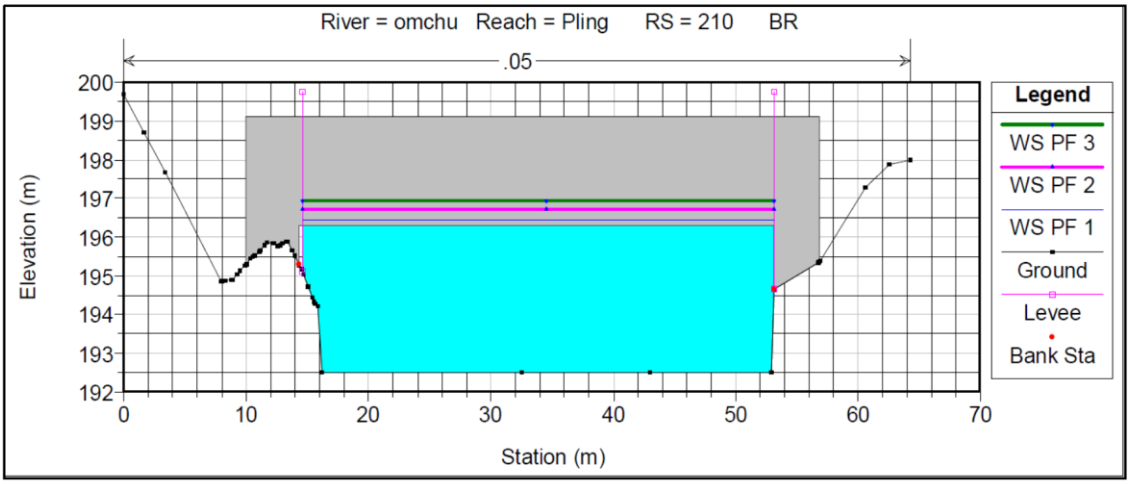
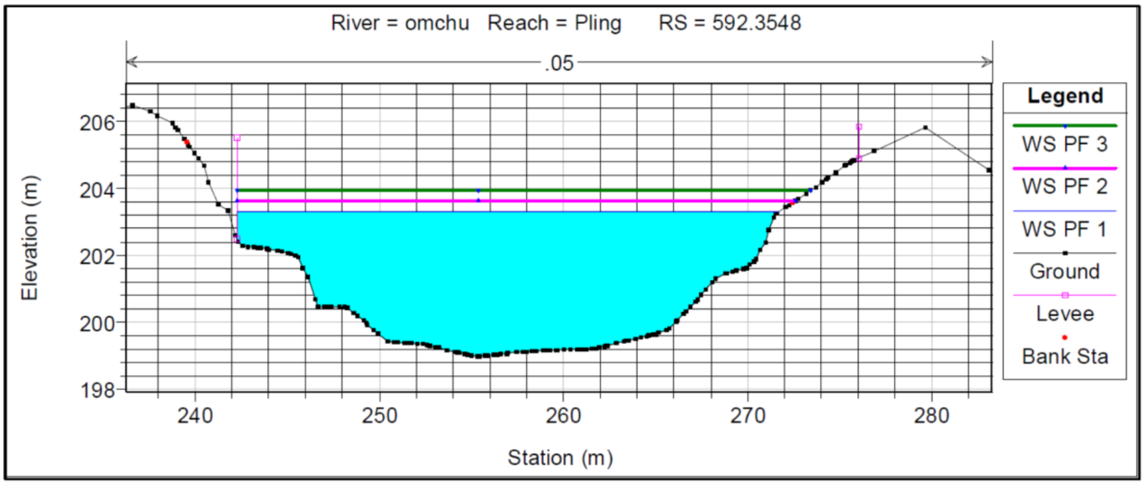


Figure 41: Cross-section of flooding at Ch. 210 just above the new Omchhu bridge with future sections and levels



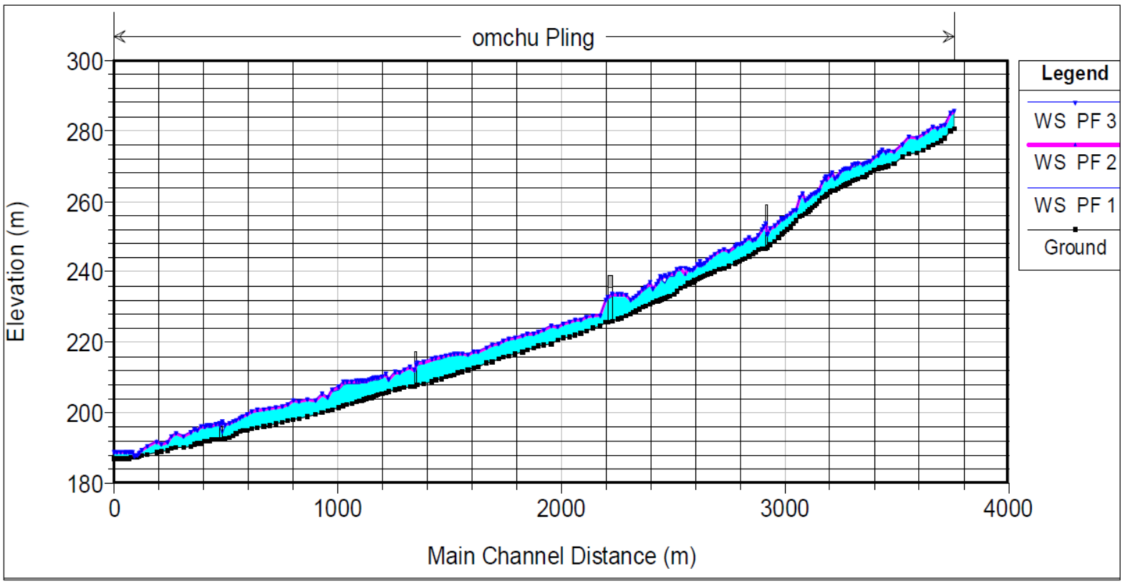
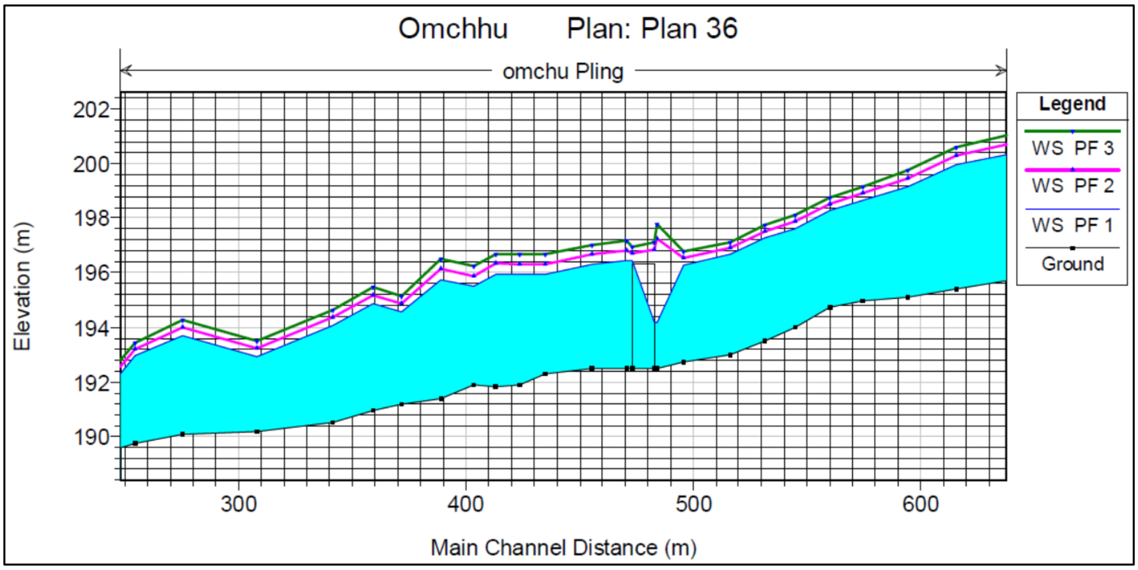


Figure 42: Longitudinal profile of the Omchhu under PF1, PF2 and PF3



5.3.3 Changing the downstream boundary condition

Model runs were performed by changing the downstream boundary condition to known water surface elevations of 191 m and 193 m at the outlet of Omchhu. An overbank flow over the right levee was observed when the water surface elevation was set to 193 m and 191 m at the outlet of Omchhu.

Figure 43: Flooding depth under PF3, KWS El. 191, (m)

Figure 44: Flooding depth under PF3, KWS El. 193, (m)

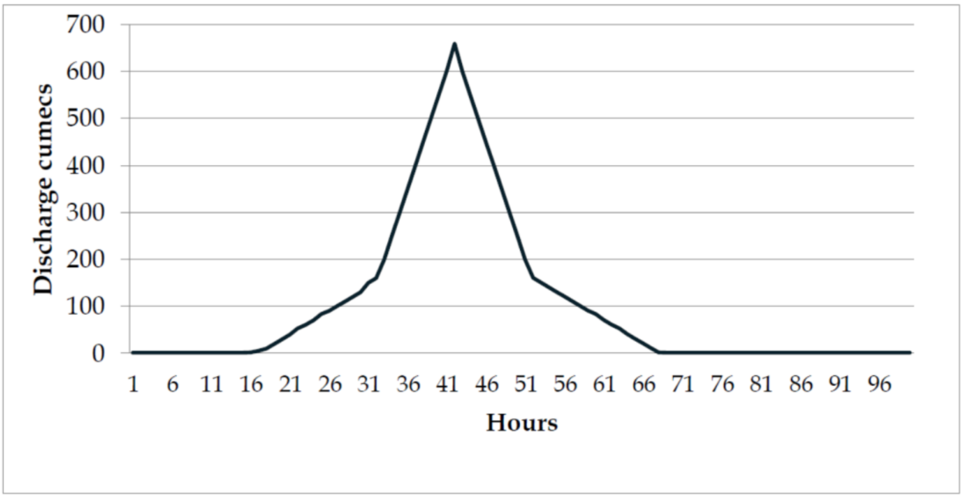
5.4 2D model

A 2D model was set up since an overflow result in a change in the preferred single flow axis. The diffusion wave equation was used to model the 2D flow. A mesh size of 10 m was used for generating the 2D flow area cells (Figure 45) in addition to the survey data.

Figure 45: 2D mesh area

The boundary condition for the upstream river reach is a hydrograph with a maximum flow of 660.89 m^3/s. The flow hydrograph is shown in Figure 46.

Figure 46: Flow Hydrograph



Normal depth condition was used as the downstream boundary condition for the 2D flow area with an estimated friction slope of 0.001 at the downstream area of the 2D. The normal depth for each given flow is calculated with the Manning's equation using the value of the friction slope.

Figure 49 shows the 2D model results at maximum flow depth for the old scenario of 2017 and 2020. The velocity profile of the model is shown in Figure 50. The following figures Figure 47 and Figure 48 _bookmark47show zoomed in details of velocity and depth of flooding under PF3 around the YDF centre area. We can see that the results are not particularly clear due to limitation in the data[22]. From the velocity spatial distribution, it can be observed that the flow velocities in the flow channel exceed 6 m/s almost for the entire channel except for the downstream portion.

Figure 47: 2D model (Maximum water depth (m) for PF3

[22] The flow towards the lower end to the traffic circle (YDF side) has also been limited with artificial flowpath restriction.

Figure 48: Maximum Velocity (m/s) for PF3

Figure 49: Flood depth at 660 m³/s (m)

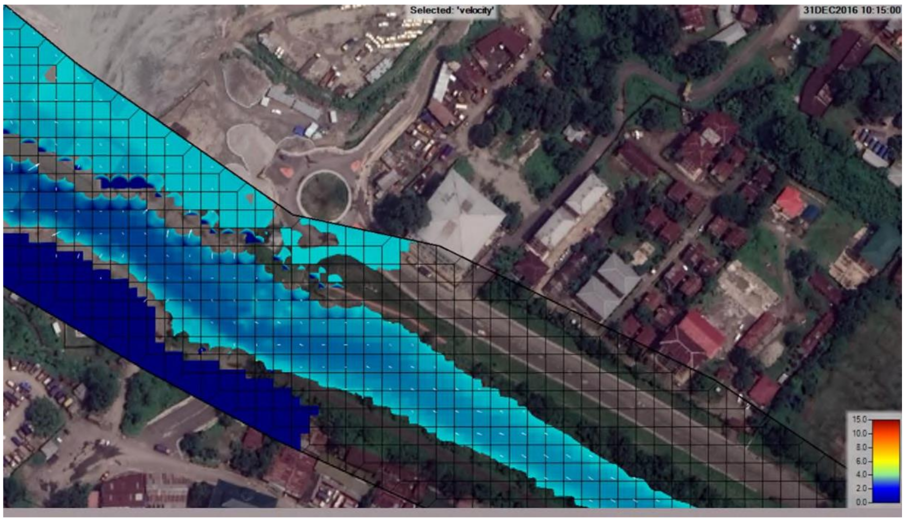
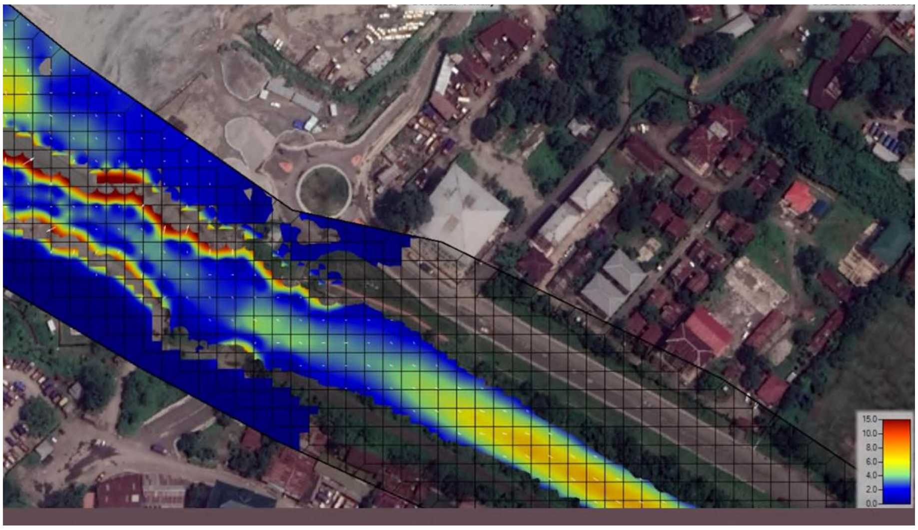


Figure 50: Flow velocity at 660 m³/s (m/s)



The 2D model developed shows similar results to the 1D model with a backwater at the new bridge under the PF3 scenario.

5.5 Scour Depth Analysis

The Scour depth for the Omchhu was determined using Lacey's and Blench methods. The main reason behind using these methods were due to their wide use and reliability in the Indian subcontinent. The Blench method used is as per the technical supplement 14B of the National Engineering Handbook[23] and the formula is described below. Coefficients considered from TS14B-23 as below;

K	0.530	b	-0.667
a	0.667	c	-0.1092
D50;	4.80 mm -5.0 mm – based on field data		

$$z_t = K Q_d^a W_f^b D_{50}^c \qquad \text{(eq. TS14B–23)}$$

where:

z_t = maximum scour depth at the cross section or reach in question, ft (m)
K = coefficient (table TS14B–8)
Q_d = design discharge, ft^3/s (m^3/s)
W_f = flow width at design discharge, ft (m)
D_{50} = median size of bed material (mm)
a, b, c = exponents (table TS14B–8)

Based on the Blench method considering medium bends, the depth of scouring ranges from 1.2 m to 2.75 m. The Lacey's regime formula is as below; R = 0.47 $(Q/f)^{1/3}$, With Ds= XR-h

Where,

- Ds (m) – Scour Depth at design discharge Q (m3/s)- Design Discharge
- H (m) – Depth of flow, (HFL-LWL)
- F (-)- Lacey's silt factor = 1.76(d50)1/2
- D50 (mm)- Median Diameter of sediment particle X (-) – Multiplying factor for design scour depth
- X ranges from 1.25 for straight rivers to 1.75 for bends to 2.00 for right-angle turns.

The scour depth of the Omchhu at all sections been found using the factor of 1.5 in Lacey's formula. It was found that the average simulated scour depth is 2.3 m while values from 0.50 m to a maximum of 6.83 m at certain locations, where the flow channel is narrow, were simulated. The average depth for the whole 3.5 km stretch is 2.3 m.

Based on the recommendation of the National Engineering Handbook, the higher figure from Lacey's was used to ascertain the scour depth. The details of the scour depth are shown in Appendix 3

[23] Shields, F. D., Jr. 2007. Scour calculations. Technical Supplement 14B in Stream Restoration Design, National Engineering Handbook Part 654, USDA-NRCS Washington, D. C.

5.6 Conclusion from the HEC-RAS Modelling

The flow simulations with the new bridge at Omchhu near YDF, Chainage 210, there is flooding observed under PF1, PF2 and PF3. This indicates that the new bridge indeed is the one that is going to be a problem under a flow equivalent to PF1 and above, if adequate measures are not taken. The channel capacity of the new Omchhu bridge is below 503.2 m³/s. This is plausible given the fact that the bridge deck depth is 2.8 m which drastically reduces the section available above the bed and below the bridge deck.

Overbank flows were observed from all PF1, PF2, and PF3 scenarios resulting in a backwater from the bridge. The height of the overflow on the upstream of the bridge is around 200-198.5 = 1.5m. There is a slight change in backwater simulated due to different bed friction coefficients. But it is negligible. The Manning's n used for the model are thus 0.035 for the main channel and 0.05 for the side channel. For more sensitivity analysis of sediment load, it is covered in the Hydraulic Modelling Report, which is a separate report of this study.

As informed by the PTDP PIC team, the river bed level of the new Omchhu bridge is planned to be lowered with the addition of the new PTDP structure at the Omchhu outlet with a gradual slope of 1:74. When the lowering of the river bed level has been implemented, the river bed at the new Omchhu bridge level will be decrease with almost 1.75 m. With such a change, even the PF3 flood could then be accommodated by the new Omchhu bridge without any significant problems. Therefore, it is imperative that the lowering of the Omchhu- Amochhu confluence and the new Omchhu bridge at Ch 210 will be implemented. Besides that, annual dredging or even dredging after each flood event at these sections should be applied without fail.

The bed level of the Omchhu to be maintained at the Omchhu bridge is 192.50. If we only look at the Amochhu water level of 191m (FFL) or 193 m (HFL) with current geometry, there is possibility of flooding in the Omchhu specifically around the new Omchhu bridge under all PF1, PF2, and PF3 scenarios. The 1D and 2D models show similar results of flood depth with the new cross-sections without the bridge.

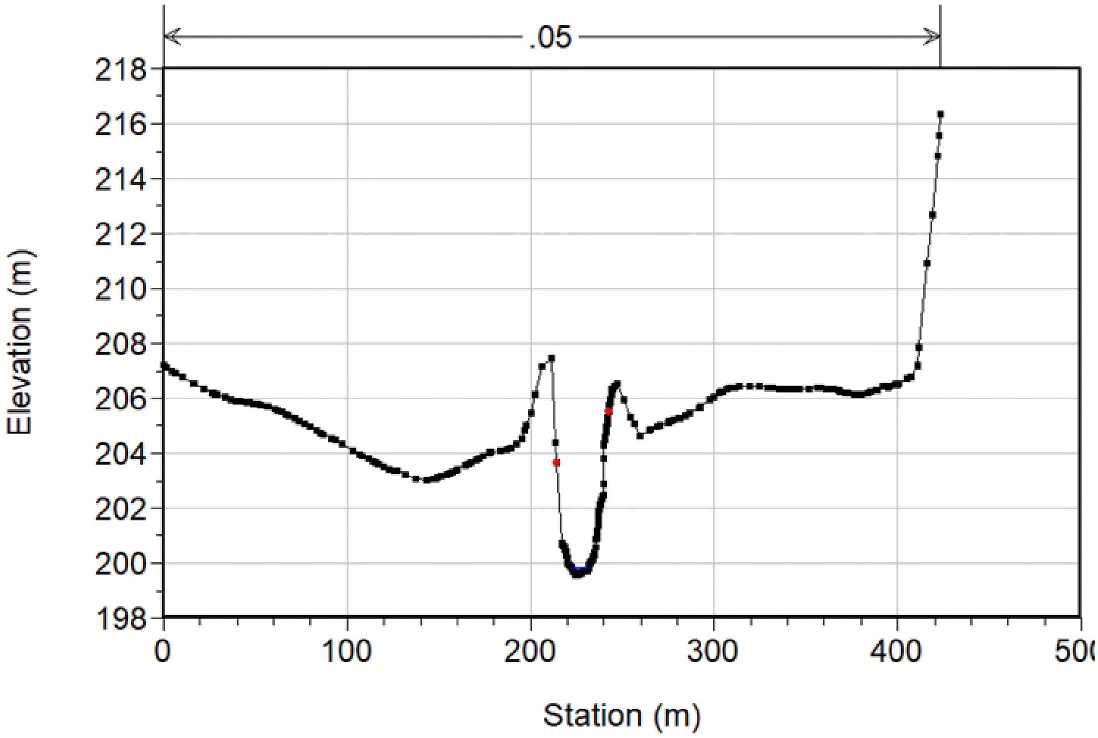
Both Blench and Lacey's method of scour depth assessment were used to determine the scour depth. The average scour depth is estimated at 2.3 m and a maximum of 6.83 m using Lacey's method which calculates higher values than the Blench method.

6 Sediment Modelling in HEC-RAS

6.1 Introduction

The riverbed of the Omchhu is steep and is mainly composed of gravel with different grain sizes combined with small, medium and large boulders. The main channel cross-section is relatively small, about 20 m-30 m wide at the top level of the levees, v-shaped and sharp curved. An example of the cross-section at the footbridge is given below.

Figure 51: Example of the cross-section at the footbridge



The embankments and flood plains outside the main flow channel, when not serving as residential area, are filled mainly with bushes and rocky areas. The embankments of the main flow channel are steep and vulnerable for sliding because of toe instabilities induced by scouring. The scouring is often found during high flood events when stream velocities show very high values, 5 – 6 m/s. The setup of the hydraulic model of the Omchhu has been addressed in Chapter 5 of this report. One of the objectives of this model study was to perform a river morphology assessment as well, for which we apply the hydraulic model as discussed and add sediment simulations for various scenarios. It is to be noted that a sediment and morphology analysis of the Omchhu is challenging, dictated by but not limited to:

- No flow and water level observations during floods available;
- No sediment load observations available;
- The slope of the river bed is steep;
- The main flow channel cross-sections are small and sharp curved; and,
- The rising limb of storm events is steep: the transition from low flows to high flows is relatively fast (within hours).

The results of the current sediment modelling have to be looked at against the data limitations and specific river/terrain properties as mentioned. To cope with these shortcomings, we have conducted a sediment modelling sensitivity analysis, which is included in the Hydraulic Modelling Report of this study.

6.2 Sediment Model Set up for the Omchhu

Sediment modelling in HEC-RAS requires additional input data for the sediment model to be applied, such as:

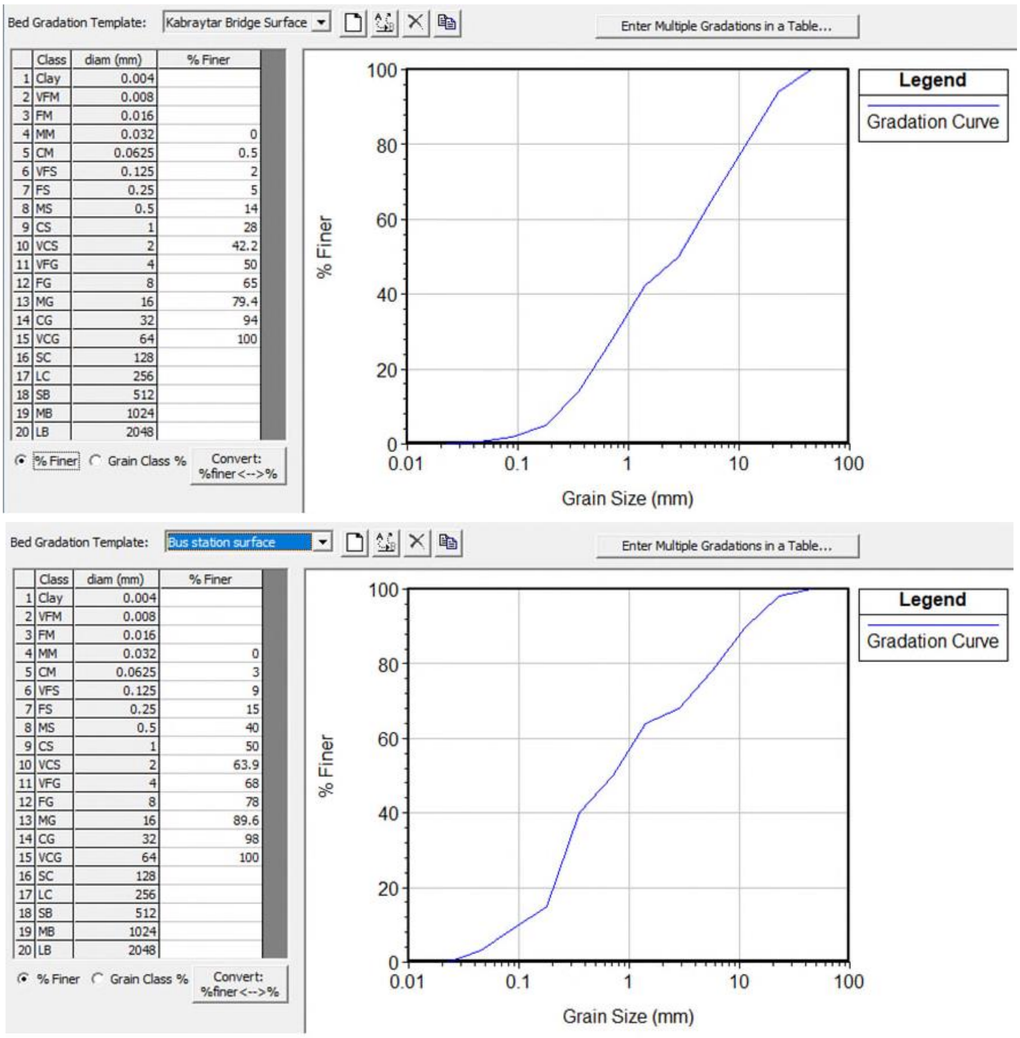
- River bed gradation;
- Section of the cross-section vulnerable for erosion;
- Sediment transport function (ST-function);
- Sorting method;
- Fall velocity method; and,
- Boundary conditions.

Within each of these items, further detail can be applied if required. We will discuss the items hereafter.

6.2.1 River bed gradation

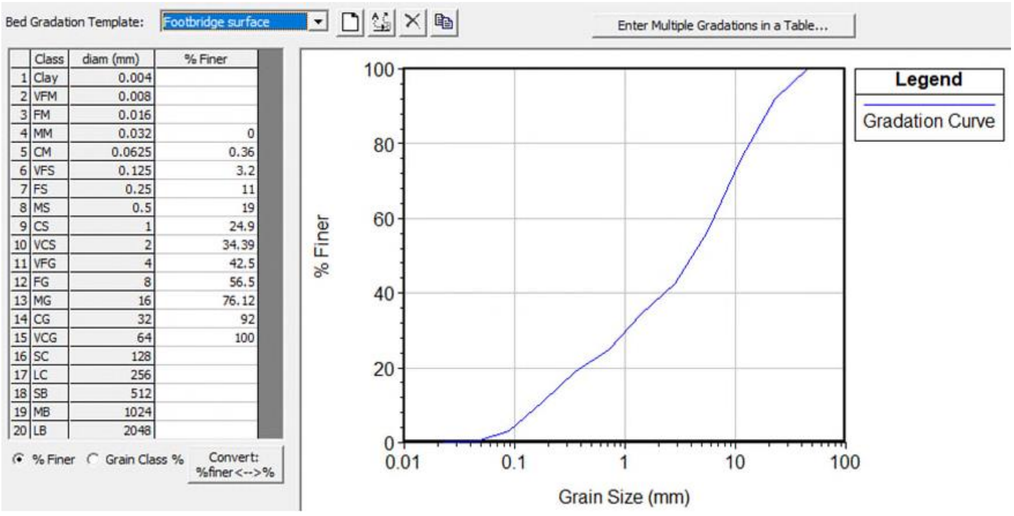
For the definition of the river bed gradation, the river bed samples taken at three locations: Kabraytar Bridge, Bridge at the bus station and near the confluence have been used.

Figure 52: Bed samples at three locations



	Class	diam (mm)	% Finer
1	Clay	0.004	
2	VFM	0.008	
3	FM	0.016	
4	MM	0.032	0
5	CM	0.0625	0.5
6	VFS	0.125	2
7	FS	0.25	5
8	MS	0.5	14
9	CS	1	28
10	VCS	2	42.2
11	VFG	4	50
12	FG	8	65
13	MG	16	79.4
14	CG	32	94
15	VCG	64	100
16	SC	128	
17	LC	256	
18	SB	512	
19	MB	1024	
20	LB	2048	

	Class	diam (mm)	% Finer
1	Clay	0.004	
2	VFM	0.008	
3	FM	0.016	
4	MM	0.032	0
5	CM	0.0625	3
6	VFS	0.125	9
7	FS	0.25	15
8	MS	0.5	40
9	CS	1	50
10	VCS	2	63.9
11	VFG	4	68
12	FG	8	78
13	MG	16	89.6
14	CG	32	98
15	VCG	64	100
16	SC	128	
17	LC	256	
18	SB	512	
19	MB	1024	
20	LB	2048	



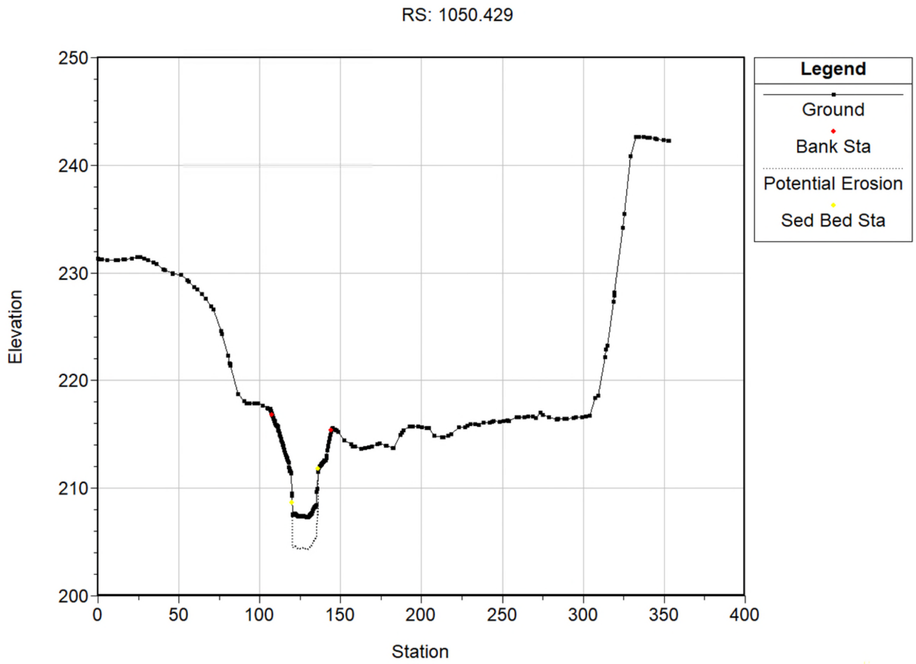
From the bed samples we used the surface samples and added them to the model as gradation templates. Next it was possible to assign each of the gradation templates to each cross-section. Since we have only three templates these have to be assigned to three sections of the modelled channel. So, we assigned the Kabraytar Bridge gradation template to the section RS=3480.295 (Inflow boundary) to RS=2340.688. The bus station bridge gradation template has been assigned to the model section RS=2337.262 to RS=1073 (bus station bridge). For the downstream section to the confluence, we applied the third bed gradation template and named it the Footbridge gradation template.

For the extent of the sediment model, we selected the entire reach of the Omchhu as has been modelled in the hydraulic model. To prevent immediate and uncontrollable erosion at the upstream boundary, we gradually decreased this erodible layer thickness from 3 m to 0.5 m, starting from RS=2720.412 m to RS=3480.295 m, the latter being the upstream inflow boundary.

6.2.2 Section of the cross-section vulnerable for erosion

For every cross-section so-called ‘Movable Bed Limits’ have to be specified which constrain erosion and deposition to the cross-section nodes between the selected limits. The model will only deposit or erode wet cross-section points between these lateral limits by default. We have selected the position of the movable bed limits mainly at the channel bank positions, but where required, we adjusted the positions. Below figure shows the cross-section at the bridge at the bus station with the bed limit points (yellow shaded).

Figure 53: Cross-section at the bridge at the bus station with potential erosion between movable limits



The potential erosion is given by the dashed line. For the ST-calculations it is required to assign a certain erodible depth in the riverbed. This would be the vertical measure until a solid layer or rock bottom is reached. For the Omchhu we have set the thickness of the erodible layer at 3 m.

6.2.3 Sediment transport function (ST-function)

Haddadchi et. al.[24] stated that at a given water discharge, two important input parameters that can affect the bedload discharges are river bed slope and representative grain size of sediments. Their analysis of bed load equations showed that higher overall accuracy of a formula does not guarantee that the formula is superior to the others under all flow and sediment conditions. The accuracy rating of a formula may vary depending on bed slope, grain size diameter, and other hydraulic and sedimentological data. The study states that this indicates that one cannot predict bed load transport discharge with any degree of reliability without an adequate number of observations. The Haddadchi study was conducted to a mild river bed slope being 0.03 % (0.0003 m/m). In our case the Omchhu riverbed slope varies from values around 0.05 m/m in the upstream section of the model through 0.025 m/m in the mid-section to 0.01 m/m in the lower section of the model. The steepness of the Omchhu river bed slope may have a considerable impact on the accuracy of the sediment calculation results, which seems independent of the selected ST-formula. We have selected the Meyer-Peter and Müller (MPM) formulation as most suitable for application of bed load transport simulation in mountainous gravel bed rivers, as the Omchhu. In the Sensitivity analysis chapter of the Hydraulic Modelling report, the uncertainty in the sediment simulation results by applying other ST-formulas and parameter settings is discussed.

6.2.4 Sorting method

Sediment transport functions compute transport potential without accounting for availability. The bed sorting method (sometimes called the mixing or armouring method) keeps track of the bed gradation

[24] Haddadchi, et. al. Bedload equation analysis using bed load-material grain size, J. Hydrol. Hydromech., 61, 2013, 3, 241–249 DOI: 10.2478/johh-2013-0031

which HEC-RAS uses to compute grain-class specific transport capacities and can also simulate armouring processes which regulate supply results. We applied the simple two-layer active layer method. The active layer thickness is set equal to the d90 by default which assumption is only appropriate for gravel beds.

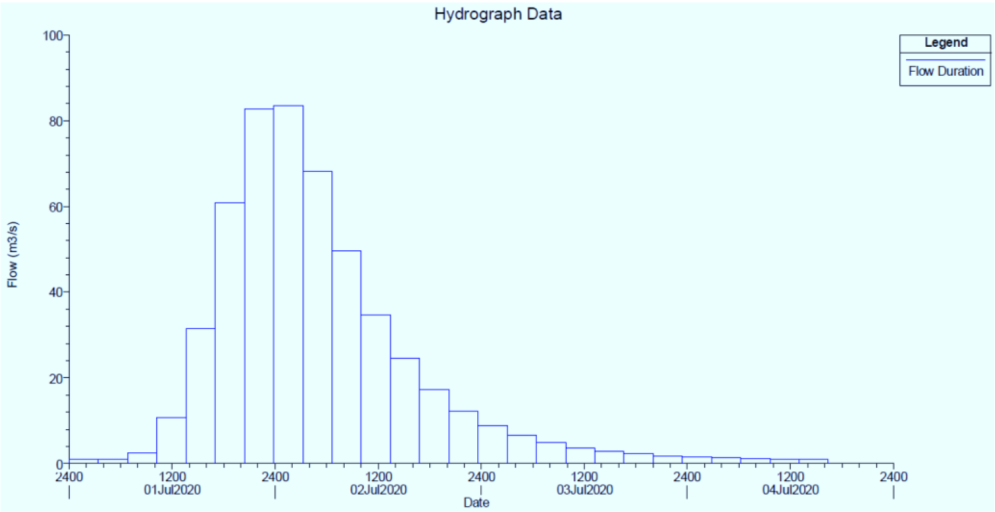
6.2.5 Fall velocity method

The fall velocity controls how fast particles can drop out of the water column and deposit. By comparing the vertical distance, a particle has to travel to reach the bed surface and the vertical distance a particle can travel in a time step (fall velocity * time). We have selected the default method in which case the fall velocity as used in the selected sediment transport function is applied. The default fall velocity method is used to remain consistent with the development of the sediment transport function.

6.2.6 Boundary conditions

For sediment simulation in HEC-RAS the hydraulic regime is the driving force. Instead of steady flow calculations as applied in the hydraulic assessment of this report, we will apply the quasi-unsteady hydraulic simulation mode. This enables us to simulate a hydrograph as would occur also in reality. For the selected probable floods, a set of hydrographs has been set up with the maximum flow of the probable flood as the peak flow. The figure below shows an example of the probable flood event PF3 (Qmax = 83.5 m3/s).

Figure 54: Quasi-unsteady flow hydrograph for probable flood PF3



The dates along the X-axis are not real historical dates but has been applied, as time series in HEC-RAS use real dates as property for the time step data. We composed hydrographs with a 3:25 hour time steps. The flow time step has been sub divided into morphology time steps of 15 min to prevent instabilities in the ST-calculations.

Observations of upstream bed load transport are not available to configure an upstream sediment boundary condition for our model of the Omchhu. Therefore, for the sediment load upstream boundary condition, we applied the equilibrium load boundary condition. This method computes the boundary sediment load from the bed gradation and the transport capacity. HEC-RAS computes the equilibrium sediment transport capacity- for each time step and grain class- at the upstream cross-section and introduces these capacities as load time series into the next cross-section. Since load is set to capacity at this boundary, equilibrium load cross-sections are essentially pass-through nodes. They will not aggrade or degrade. The equilibrium load boundary condition is to be assigned to a cross-section well upstream of the area of interest, which is the case of our application at the Omchhu model.

6.3 Morphological Analysis[25]

6.3.1 Introduction

For the morphological analysis we have made a set of simulations covering the 3 system states and 3 inflow hydrographs:

System states

1. Topographic situation 2020 with existing bridges: Bridge at the bus station and the New Bridge;
2. Topographic situation 2020 with existing bridges and future Northern bypass bridge; and,
3. Topographic situation 2020 with existing bridges, future Northern bypass bridge and finalized construction of the PTDP.

Inflow hydrographs

1. Probable flood PF4, 251 m3/;
2. Probable flood PF3, 83.5 m3/s; and,
3. Probable flood PF5, 660 m3/s.

The inflow hydrographs have been applied similar to the inflow hydrographs as applied at the sensitivity analysis. These inflow hydrographs differ from the ones as addressed in section 5.2 where flows at low probabilities were analyzed. The combination of system states and inflow hydrographs gives us a set of 9 model simulations, as shown in Table 30.

Table 30: Morphology simulations and their mutual comparison

Simulation	Case01 Reference	Case02	Case03	Case04	Case05	Case06	Case07	Case08	Case09
Topography	Situation 2020	Situation 2020	Situation 2020	Situation 2020	Situation 2020	Situation 2020	Situation 2020	Situation 2020	Situation 2020
Bridges / PTDP	Bridge Situation 2020	All bridges constructed	Bridge Situation 2020	All bridges constructed	Bridge Situation 2020	All bridges constructed	All bridges constructed, PTDP constructed	All bridges constructed, PTDP constructed	All bridges constrcted, PTDP constructed
Qmax inflow	251 m3/s	251 m3/s	83.5m3/s	83.5m3/s	660 m^3/s	660 m^3/s	251 m^3/s	251 m^3/s	660 m^3/s
Compare1	X	X							
Compare2	X		X		X				
Compare3		X					X		
Compare4				X				X	
Compare5						X			X

For the comparison of the model simulation results we first have defined the reference situation, shown as Case01 in the morphology simulations table. When we compare the simulation results, we look at the graphs of longitudinal profiles along the modelled Omchhu river reach for the next outputs:

- Bed level at the start, at the top and the end of the flood event for the reference situation;
- Bed level difference after the flood event for simulation result comparisons;
- Maximum water surface level difference for simulation result comparisons; and,
- Maximum stream velocity difference for simulation result comparisons.

[25] The details in this section have not been altered since the comparison is made based on a reference

The bed level change at top of the flood gives us an indication for the scour along the modelled reach of the Omchhu. The scour values will be listed in separate appendix. Besides graphs of longitudinal profiles, we want to know the total simulated bed level change volumes as these may indicate the required efforts for maintenance activities to be performed after the flood event. Therefore, we calculate the total mass bed level change in tonnes for each simulation for 2 sections of the modelled river reach:

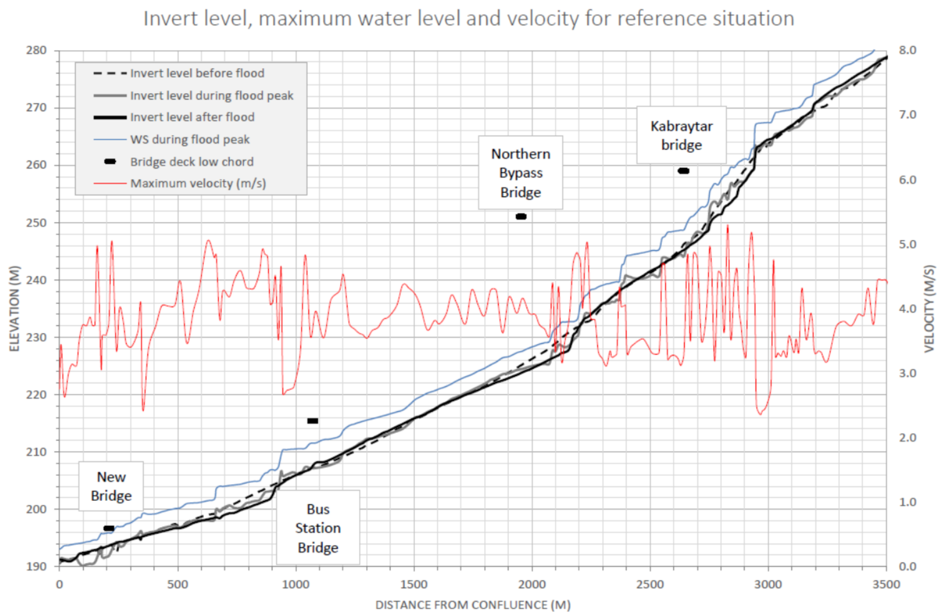
- Section 1: starting at the Kabraytar bridge and ending at the confluence with the Amochhu; and,
- Section 2: starting at the footbridge and ending at the confluence with the Amochhu.

Section 2 has been addressed separately to check on the 2019 sedimentation which occurred after the July 2019 flood event and affected the lower part of the Omchhu in the vicinity of the new bridge. We will look at simulated results starting just downstream of the Kabraytar bridge location, RS=2600, to the confluence with the Amochhu in the reference situation[26], RS=0. Because the upstream part is too close to the inflow boundary, simulation results may be unreliable there. In the next paragraphs the results of the simulation comparisons will be discussed.

6.3.2 Reference situation

Regarding the reference situation, for which the properties as given in morphology simulations Table 30, the simulated bed levels and maximum water surface level are shown in Figure 55.

Figure 55: Invert level, maximum water level and stream velocity for the reference situation



In Figure 55, simulated invert levels are shown together with the maximum simulated water surface level and stream velocity. The simulated resulting invert level profiles indicate locations with significant erosion from 900 m to around 650 m, from 2200 m to 1800 m and from 2950 m to 2700 m. These locations with significant erosion occur just downstream of relatively steeper sections of the riverbed. The maximum scour of 3 m below the initial bed level is simulated just upstream of the Northern Bypass

[26] Comparison of all simulation results is done assuming the location of the confluence (RS=0) as before implementation of the PTDP. So, also for simulations including the PTDP outlet, the original location of the confluence is used.

bridge location, around RS 2150 m. Note that the distortion of the horizontal and vertical scales regarding the distance and elevation in Figure 55 results in exaggeration of the river bed slope. This also results in a 'bumpy' rendering of the water surface level. Downstream, from around 300 m to the confluence, the river bed seems to return to the original bed level after the flood event. The maximum stream velocity indicates acceleration after the drops in bed level at the locations as mentioned above. Just before the bed level drops, the velocity shows lower values. Just upstream of Kabraytar bridge a maximum velocity of 5 m/s is simulated. The maximum simulated sediment concentration during the flood peak is 3420 mg/l while the average simulated sediment concentration for the flood event is 1829 mg/l.

For Section 1 of the river reach, from Kabraytar bridge to the confluence a total mass of 10114 tonnes (=6361 m3) bed level change is calculated. For Section 2 of the river reach, from the footbridge bridge to the confluence a total mass of 15909 tonnes (=10006 m3) bed level change is calculated. In the next paragraphs the results of the model simulations will be mutually compared showing graphs with signals of the difference between the simulation outputs of:

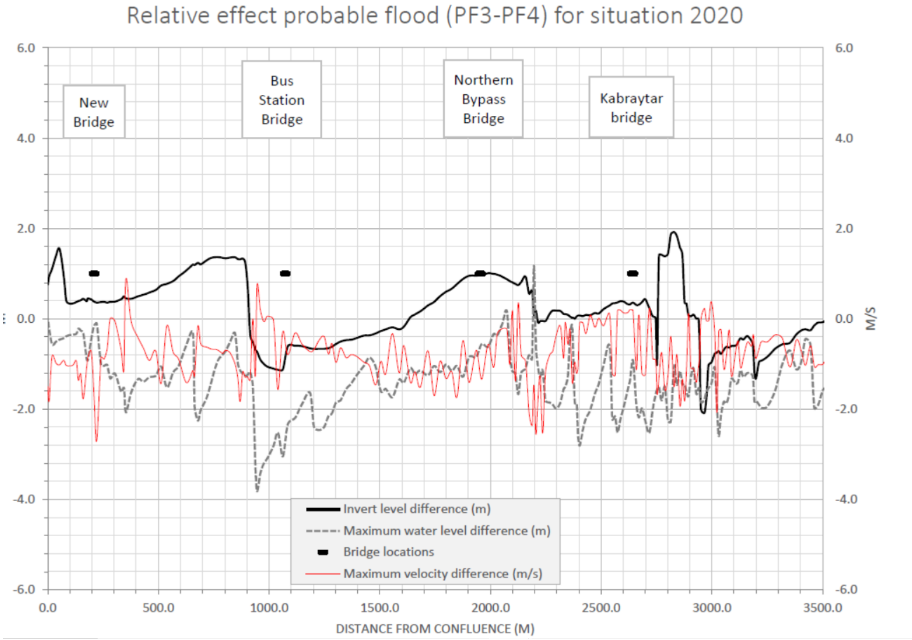
- Invert level difference at the end of the flood event;
- Maximum water surface level difference; and,
- Maximum stream velocity difference.

The average sediment concentration at the top of the flood event and at the end of the flood event will be summarized in Table 31 in the last paragraph of the chapter. Table 31 also contains the total mass bed level change in tonnes and the total volume bed level change in m^3 for each simulation for 2 sections of the modelled river reach as mentioned above.

6.3.3 Impact of lower probable flood (PF3-PF4)

The effect of a lower probable flood. PF3 with with Qmax = 83.5 m3/s, compared to the probable flood PF4 with Qmax = 251 m3/s is shown in Figure 56.

Figure 56: Impact of a lower probable flood PF3 versus the reference flood PF4

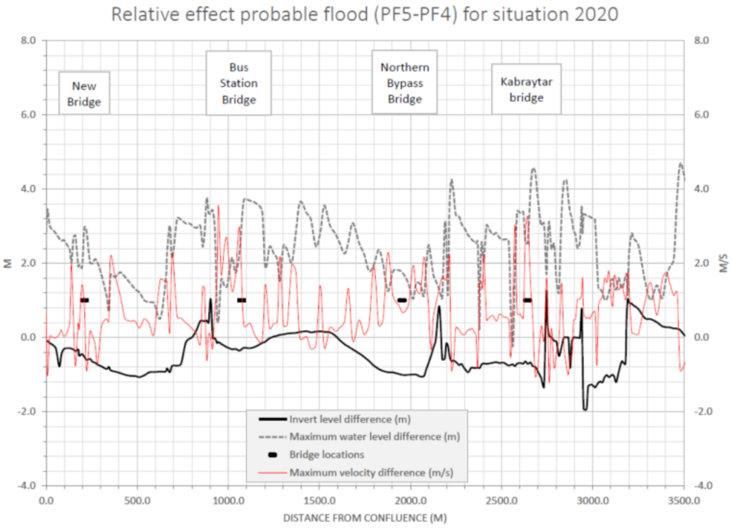


As can be expected, the maximum stream velocities and maximum water surface levels are simulated lower with the PF3 probable flood than with the PF4 probable flood (reference case). From Figure 56 it can also be noted that the bed level change in the section from 900 m to the confluence with the Amochhu compared to the reference situation is on average 1 m higher. This seems to indicate that sedimentation in the downstream section is sensitive for the flood discharge rate: at lower maximum discharge rates, higher sedimentation is simulated.

6.3.4 Impact of extreme probable flood (PF5-PF4)

The effect of an extreme probable flood. PF5 with with Qmax = 660 m3/s, compared to the probable flood PF4 with Qmax = 251 m3/s is shown in Figure 57.

Figure 57: Impact of an extreme probable flood PF5 versus the reference flood PF4

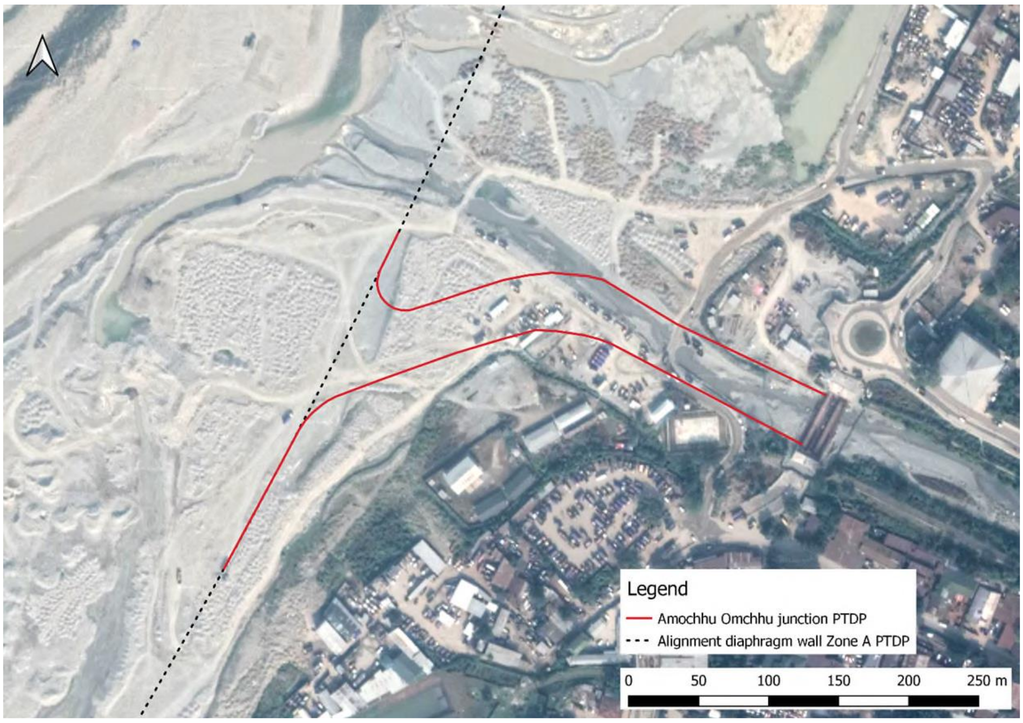


As can be expected, the maximum stream velocities and maximum water surface levels are simulated higher with the PF5 probable flood than with the PF4 probable flood (reference case). Water level differences over 4 m are simulated in the upstream part of the model and near the bridge at the bus station 3.7 m water level increase during the PF5 flood peak is simulated compared to the simulated maximum water level at the reference situation, PF4. From Figure 57 it can be noted that the overall bed level change in the modelled reach of the Omchhu is about-0.8 m, indicated an increase of erosion at times of extreme floods. At a few locations some accretion is simulated: around 900 m from the confluence (downstream of the bridge at the bus station) and at around 1400 m from the confluence (upstream of the bridge at the bus station). At several locations 2 m/s increase of stream velocity is simulated compared to the reference situation. The maximum absolute simulated stream velocity at the PF5 simulation is 6.5 m/s.

6.3.5 Impact of the PTDP construction at 3 probable floods

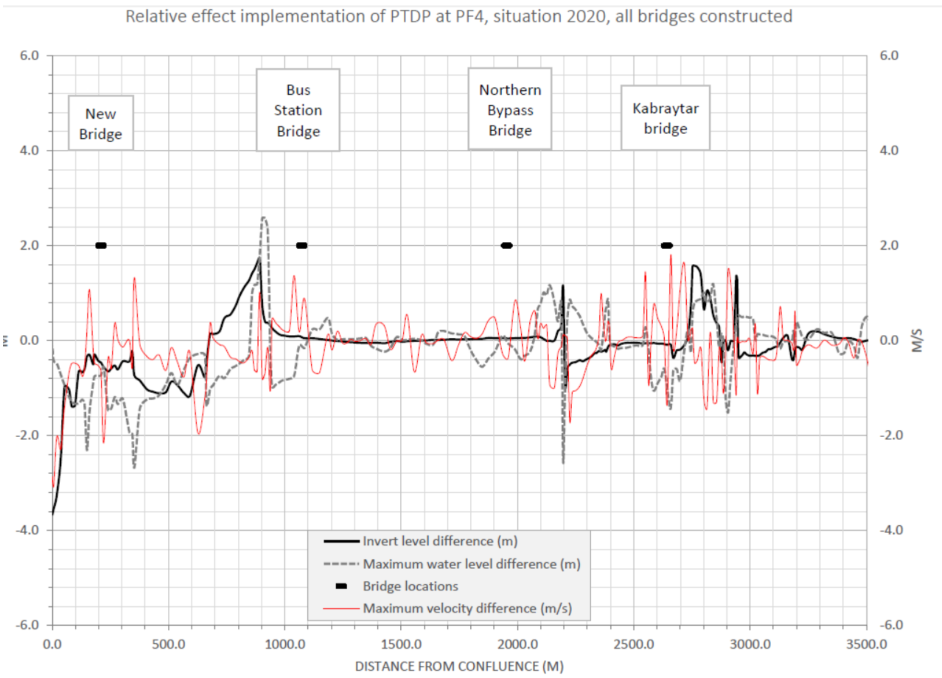
The Phuentsholing Township Development Project (PTDP) is ultimately aiming to develop 464 hectares of riparian land adjacent to the Amochhu River near the city of Phuentsholing on Bhutan's southwestern border with India. The project will provide protection to the new township from floods and erosion, and construct smart urban infrastructure to allow phased urban expansion. The project will also protect the existing town of Phuentsholing from floods and riverbank erosion associated with the Amochhu, which currently threatens lives and livelihoods and disrupts connectivity with nearby communities. The downstream end of Zone A of the PTDP, which is constructed at the left bank of the Amochhu, is located at the Indian border. The Omchhu will cross the PTDP enabled by an outlet construction with concrete walls, see Figure 58.

Figure 58: Amochhu-Omchhu junction of the PTDP



We have looked at the impact of the implementation of the PTDP outlet for three probable floods, with topography 2020 and all bridges constructed. The relative effect of the implementation of the PTDP at the reference probable flood, PF4 with Qmax = 251 m^3/s is shown in Figure 59.

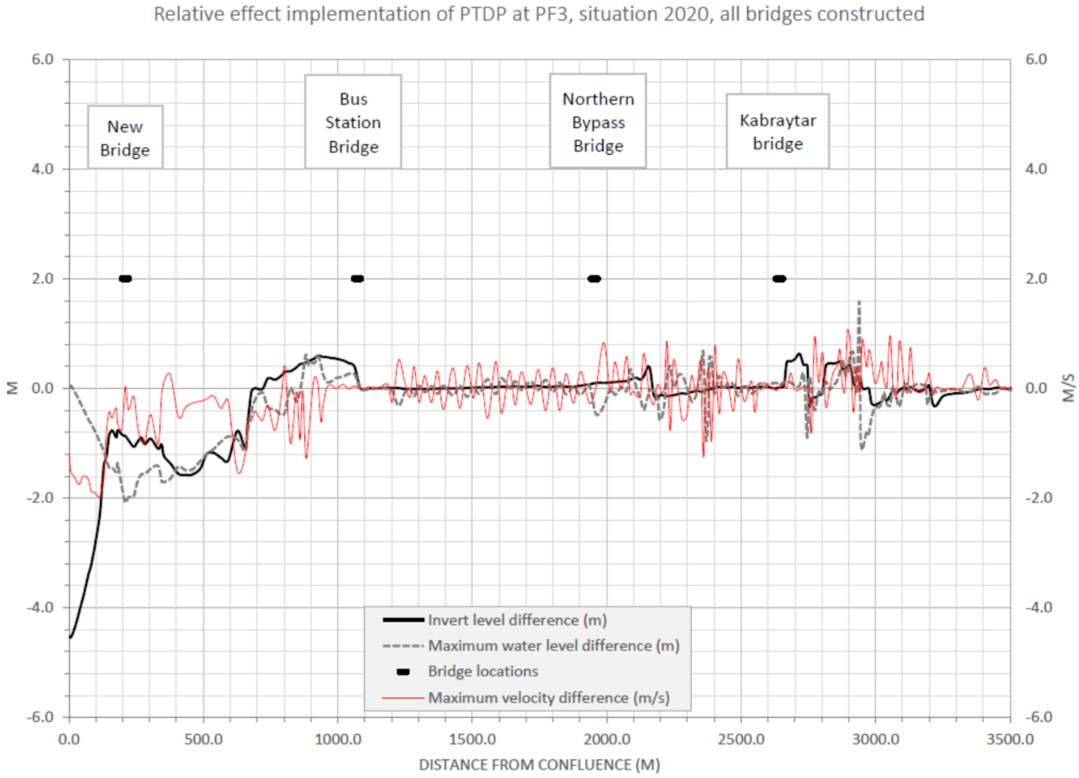
Figure 59: Relative effect implementation of PTDP at PF4, situation 2020, all bridges constructed



Implementation of the PTPD outlet construction to accommodate the confluence of the Omchhu and the Amochhu may have effects on the morphology in the lower section of the Omchhu. Indeed, Figure 59 shows that from about 900 m to 700 m upstream of the confluence, a higher bed level is simulated but from 700 m to the confluence a lower bed level is simulated than in case of the situation without implementation of the PTDP outlet. The differences in the maximum stream velocities is on average less than 10 % of the average maximum reference stream velocity, being about 4 m/s. Locally, higher maximum stream velocity differences are simulated due to numerical side effects of the ST-model.

The relative effect of the implementation of the PTDP at the 10-year probable flood, PF3 with Qmax = 83.5 m3/s is shown in Figure 60.

Figure 60: Relative effect implementation of PTDP at PF3, situation 2020, all bridges constructed



As discussed at the effect of the implementation of the PTDP with the reference probable flood PF4, also with probable flood PF3, a similar effect on the bed level in the lower section of the Omchhu can be observed in the model simulation results. In this case the sedimentation starts more upstream, near the bridge at the bus station. For the model section from the bridge at the bus station to Kabraytar bridge, the differences with the situation without implementation of the PTDP at the 10-year probable flood PF3 seem relatively small. In this section the velocity difference series is ’saw-tooth’-like which is due to the numerical approach of the ST-calculations, progressing from one cross section to the next cross section. The signal of the velocity difference can be smoothed to uncover the trend over longer sections of the river reach. Figure 60 is almost similar to Figure 59, however the velocity difference signal has been averaged over 4 cross sections resulting in a smoothened signal.

Figure 61: Relative effect implementation of PTDP at PF3, situation 2020, all bridges constructed, with averaged velocity difference signal

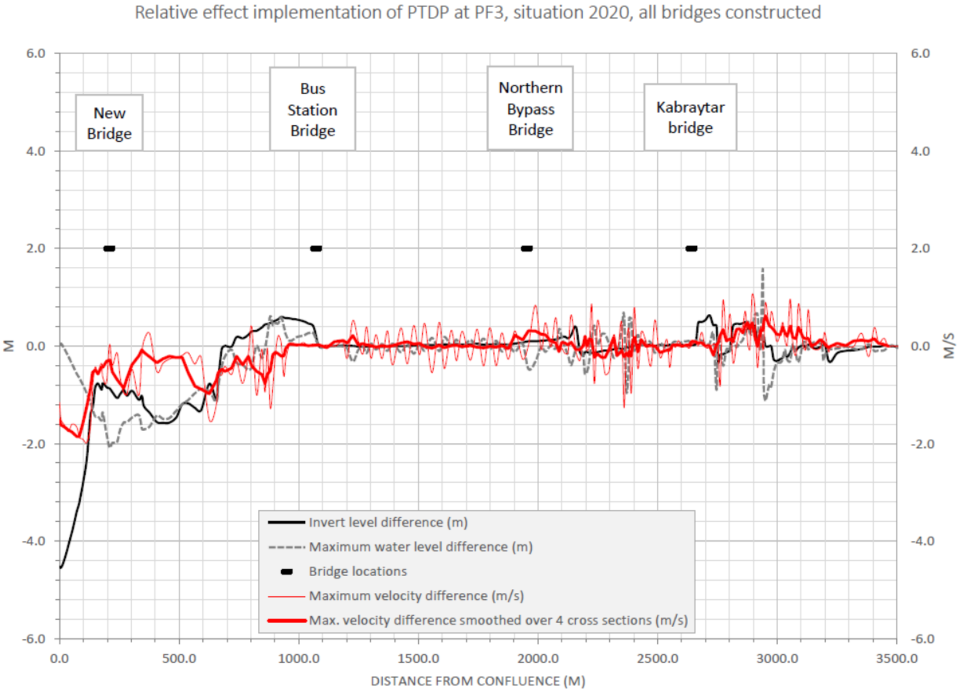
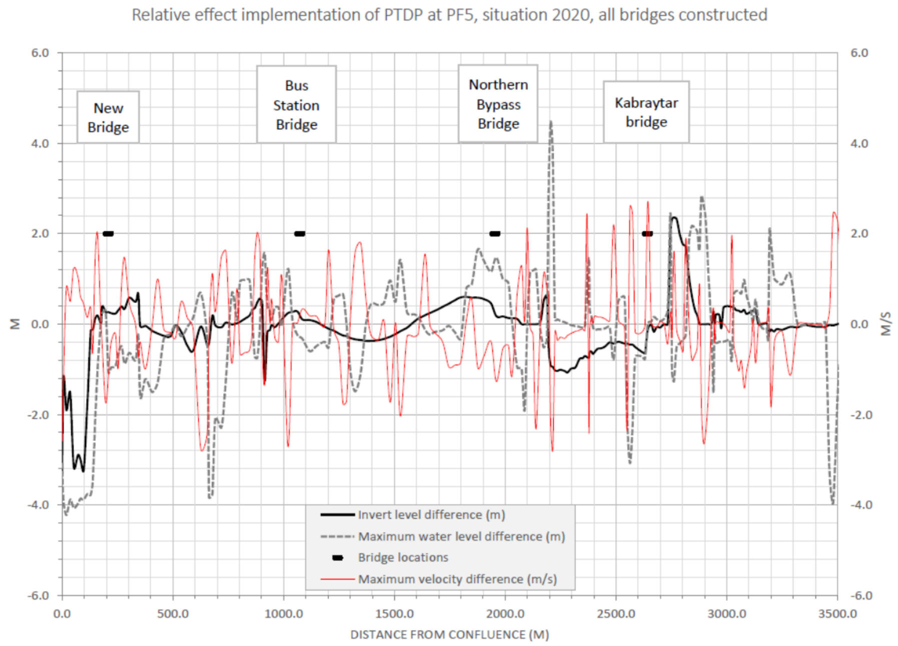


Figure 61 shows that when the velocity difference signal is smoothened over 4 cross sections a less irregular course of the signal is presented. This way of smoothening may be a valid approach to filter the irregularities resulting from the ST-calculations in case of modelling of steep mountainous river streams. Regarding the bed level change, the section between the bridge at the Bus station and Kabraytar bridge shows limited difference. The relative effect of the implementation of the PTDP at the 100-year probable flood, PF5 with Qmax = 660 m^3/s, is shown in Figure 62.

Figure 62: Relative effect implementation of PTDP at PF5, situation 2020, all bridges constructed



The effects of the implementation of the PTDP with the reference probable flood PF4 and with the probable flood PF3 on the bed level in the lower section of the Omchhu which can be observed in the model simulation results seems different in the simulation with the PF5 probable flood. In the PF5 case, the sedimentation as simulated starting from 900 m upstream of the confluence for the PF4 and PF3 flows, is not simulated. From 150 m to the start of the model (=0 m) the bed level is lowering to about-2.5 m, compared to the situation without the implementation of the PTDP. For the model section from the bridge at the bus station to Kabraytar bridge, the differences of the invert level with the situation without implementation of the PTDP at the 100-year probable flood PF5 seem relatively bigger for the PF5 flow than for the PF4 and PF3 flows. However, this may well be related to the sensitivity of the sediment transport solving method in relation to the hydraulic calculation method when steep mountainous streams are modelled.

6.3.6 Simulated bed level change and sediment concentrations

In Section 6.3.2 Reference situation we already discussed that the average sediment concentration at the top of the flood event and at the end of the flood event will be summarized in Table 31 in this paragraph. This table also contains the total mass bed level change in tonnes and the total volume bed level change in m^3 for each simulation for 2 sections of the modelled river reach as mentioned above.

Table 31: Sediment concentrations and bed level change

Case ID	System state	Q max (m^3/s)	Average sediment concentra-tion (mg/l)	Maximum sediment concentra-tion (mg/l)	Mass bed level change section 1 (tonnes)	Volume bed level change section 1 (m3)	Mass bed level change section 2 (tonnes)	Volume bed level change section 2 (m3)	Max bed level change New bridge (m)
Case01	Bridge Situation 2020 (Reference)	251.0	1828.6	3420.5	10114.2	6361.1	15909.4	10005.9	1.68
Case02	All bridges constructed	251.0	1704.1	3088.8	7038.3	4426.6	16705.8	10506.8	1.74
Case03	Bridge Situation 2020	83.5	2999.2	4572.7	17589.1	11062.3	17861.8	11233.8	2.07
Case04	All bridges constructed	83.5	2923.9	4591.1	14955.9	9406.2	17657.0	11105.0	2.08
Case05	Bridge Situation 2020	660.0	853.9	2476.8	5260.6	3308.6	14830.6	9327.4	1.36
Case06	All bridges constructed	660.0	827.0	2347.3	6046.7	3802.9	17413.5	10951.9	1.31
Case07	All bridges and PTDP constructed	251.0	1862.3	3378.5	13604.4	8556.2	14744.2	9273.1	1.86
Case08	All bridges and PTDP constructed	83.5	2853.1	5711.0	19842.4	12479.5	17378.4	10929.8	1.50
Case09	All bridges and PTDP constructed	660.0	822.7	1712.1	-6021.6	-3787.2	7187.4	4520.4	1.95

The average sediment is calculated as an average for the entire modelled river reach during the top of the flood event. The maximum sediment concentration is calculated as the maximum found in the entire modelled river reach during the top of the flood peak. We already pointed out the locations of both sections in the Introduction paragraph of this chapter. For completeness we list the locations as shown below:

- Section 1: starting at the Kabraytar bridge and ending at the original[27] confluence with the Amochhu; and,
- Section 2: starting at the footbridge and ending at the original confluence with the Amochhu.

Section 2 has been addressed separately to check on the 2019 sedimentation which occurred after the July 2019 flood event and affected the lower part of the Omchhu in the vicinity of the new bridge.

From Table 31 it can be observed that the main part of the bed level change (deposition) is simulated in section 2 of the modelled river reach, downstream of the foot bridge, RS=629.2m. The last column of the table shows the maximum bed level change for the most downstream section starting at the New bridge (RS=210 m). It can be observed that for the bed level change higher values are simulated at a lower maximum flow, PF3, than at higher flows, PF4 and PF5. This seems consistent with the results of the sensitivity analysis, as discussed in the Hydraulic Modelling Report. For the simulations Case03 and Case04 at the PF3 probable flood show the highest simulated bed level change at the location near the New bridge, 2.1 m.

6.4 Conclusions and recommendations

From the morphology simulation results it can be concluded that the amount of sedimentation and erosion is highly dependent on the discharge rate in the flow channel. As discussed earlier, the steepness of the river bed combined with the flow rate can induce severe erosion at certain locations but also sedimentation at other sections. We have performed 9 simulation cases with different boundary conditions and model systems, as listed in Table 32.

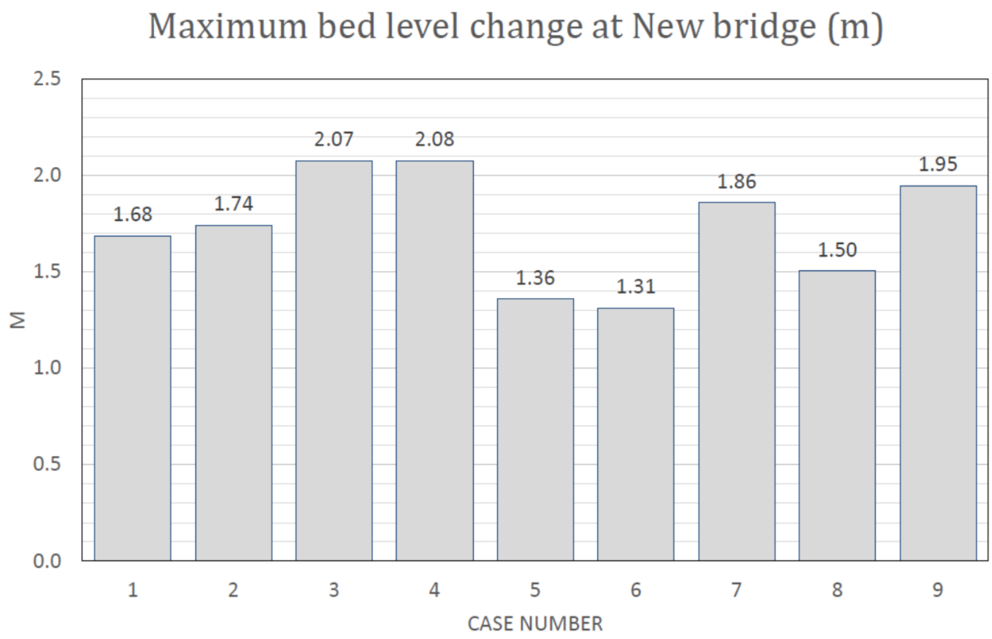
Table 32: List of performed 1D-morphological simulations

Case ID	System state	Q max (m3/s)
Case01	Bridge Situation 2020 (Reference)	251.0
Case02	All bridges constructed	251.0
Case03	Bridge Situation 2020	83.5
Case04	All bridges constructed	83.5
Case05	Bridge Situation 2020	660.0
Case06	All bridges constructed	660.0
Case07	All bridges and PTDP constructed	251.0
Case08	All bridges and PTDP constructed	83.5
Case09	All bridges and PTDP constructed	660.0

All simulations were performed with a fixed water level at the downstream boundary of 193 m. One of the interesting points to be addressed was the heavy siltation which occurred after the flood event of July 2019 near the bridge at the Youth Development Fund (YDF) building. Figure 63 shows the maximum bed level change at the bridge location for all 9 simulations.

[27] Situation at the confluence before construction of the PTDP

Figure 63: Maximum bed level change for 9 simulations



It can be observed from Figure above that the simulations with inflow hydrographs PF3 (83.5 m^3/s), Case 3 and Case 4, show the highest maximum bed level change (2.1 m). After construction of the PTDP, Case 8, the maximum bed level change for the same flood event is 1.5 m, so 0.6 m less than before the construction. It seems that construction of the PTDP outlet has a positive effect on the siltation of the river section from the bridge at the YDF building to the confluence with the Amochhu at moderate discharges, i.e., 83.5 m^3/s.

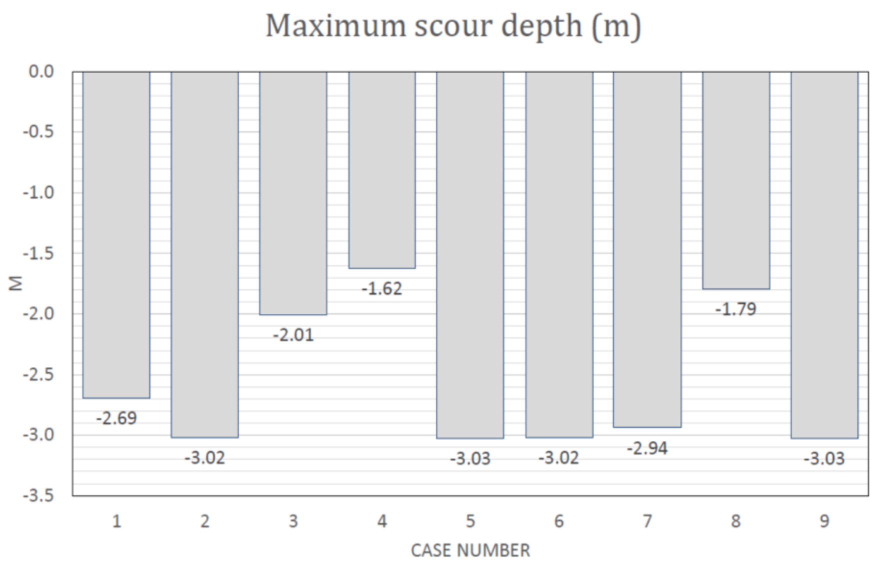
From the sensitivity analysis (see Hydraulic Modelling Report) it shows that at an increased lower boundary stage (194 m) more siltation is simulated near the bridge at the YDF building. This indicates that a combination of a high-water surface level at the Amochhu with a moderate discharge at the Omchhu (i.e., 83.5 m^3/s) will probably lead to significant siltation near the bridge. A high-water level at the Amochhu causes back water effects on the Omchhu, limiting its discharge capacity and leading to sedimentation at the confluence.

At the maximum simulated flow hydrograph, PF5 (660 m^3/s), the construction of the PTDP seems to increase the maximum bed level change compared to the situation before the construction, Case 9 vs. Case 5 and Case 6.

The results of the 1D-hydraulic simulations indicate that the bed level of the Omchhu to be maintained at the Omchhu bridge (near YDF building) is 192.5 m as per the design in 2020. The low chord of the bridge is situated at 196.3 m, which indicates that a 'clearance' of 196.3 – 192.5 = 3.8 m is recommended to be maintained to prevent the area from flooding at extreme flood events.

The maximum scour depth in the modelled section of the flow channel is shown in Figure 64.

Figure 64: Maximum scour depth in the modelled flow channel



From the figure it can be observed that maximum scour depth values are calculated is the maximum simulated flow hydrograph PF5 (660 m^3/s), Case 5, Case 6 and Case 9. However, for the reference hydrograph PF4 (251 m^3/s), also significant maximum scour depth values are simulated. The maximum scour depth is simulated at locations where the slope of the river bed shows changes from steep to less steep, see also Figure 34 Invert level, maximum water level and stream velocity for the reference situation. The simulated scour depth values seem to be in the range of calculated scour depth values with the methods of Lacey and Blench.

It is recommended to use the scour depth values as calculated with Lacey's formula when redesigning the embankment retaining walls. This also holds for river bed protection works at the bridge abutments. Since none of the modelled bridges comprises any piers, a separate bridge scouring analysis was not applied.

Based on the sediment modelling results we can derive an estimate of the maximum allowable sedimentation at the location of the new bridge near the YDF building, before the Thromde should start removing the sediment. This could give the Thromde some flexibility in planning the maintenance works without maintaining the design level at all times. We assume all planned bridges and the PTDP works to be present. We have run 3 simulations with different maximum inflows at this system state and compared the maximum water levels and the maximum bed level change at the new bridge, see Table 33.

Table 33: Bed level change and maximum water level at the new bridge

Case ID	AEP (%)	Q max (m^3/s)	Maximum water level at New bridge (m)	Maximum bed level change at New bridge (m)
Case08	10	83.5	193.6	1.50
Case07	5	251.0	195.2	1.86
Case09	1	660.0	197.8	1.95

From the table it can be observed that it can be expected that the maximum water level at the new bridge equals the lower bridge soffit (=196.3 m) for discharges above 251 m^3/s. It should be noted that the maximum bed level change is calculated after the flood. During the maximum flood peak, the simulated bed level change itself is less due to the flushing effect of the stream power. From the hydraulic simulations it can be concluded that a flow of 503 m^3/s can still be accommodated at the new bridge without back water effects when the design river bed level is maintained. We expect that allowing some sedimentation to about 1.0 m may accommodate at least the 10 % AEP flow of 251 m^3/s, but to about 400 m^3/s at maximum[28].

[28] The hydraulic simulations with the cross-section conditions from the Omchhu DPR of 2017-2018 combined with the construction of the new bridge (see Paragraph 3.4 Hydraulic Model Results) show, that only the PF1 flow (Q=503 m3/s) passes the bridge without any back water effects. The PTDP design level will be 1.5 m lower, so one could assume to allow 1.5 m sedimentation with a maximum flow of 503 m3/s. To be on the safe side (and taking possible uncertainties into account) and assuming that gravel removal may not start immediately when required due to other circumstances, a safe estimate would be allowing 1.0 m sedimentation and lowering the maximum allowable flow to 400 m3/s.

7 Omchhu River Morphology

7.1 Introduction

A morphological analysis of the Omchhu was conducted using field observations and aerial photo analysis in order to provide context for interpreting the hydraulic modelling results and to identify additional morphological hazards not readily identified with hydraulic modelling. Phuentsholing (and portions of neighboring Jaigaon in India) is situated on an alluvial fan (Figure 65: Phuentsholing and portions of neighboring Jaigaon are built on an alluvial fan (outlined by the dashed yellow line) built by the Omchhu out into the wide Amochhu valley through the deposition of sediment carried from the steeper confined upper Omchhu watershed. Alluvial fans are cone-shaped landforms found at the mouths of steep confined narrow valleys where they debouche onto flatter unconfined valleys just as is the case in Phuentsholing. The declining slope and flow expansion in these settings results in sediment deposition and allows fans to build out onto the valley bottom over long time periods. Alluvial fans are often appealing areas for human settlements as the fan surfaces are elevated above the more frequently flooded valley bottoms. However, sudden changes in channel position, known as channel avulsions, are possible on alluvial fans, difficult to predict with hydraulic models, and can cause severe damages and loss of life. The dense urban development of the alluvial fan in Phuentsholing (Figure 66) and past flood control efforts in the city belie this potential hazard, but avulsions remain a very real risk in the lower elevation portions of the town. (Elevated areas of town such as by the hospital and bottling company as well as by the Park Hotel and Lhaki Hotel are older abandoned portions of the fan and are not at risk of an avulsion or any of the other morphological hazards discussed below.)

Figure 65: Phuentsholing and portions of neighboring Jaigaon are built on an alluvial fan (outlined by the dashed yellow line)

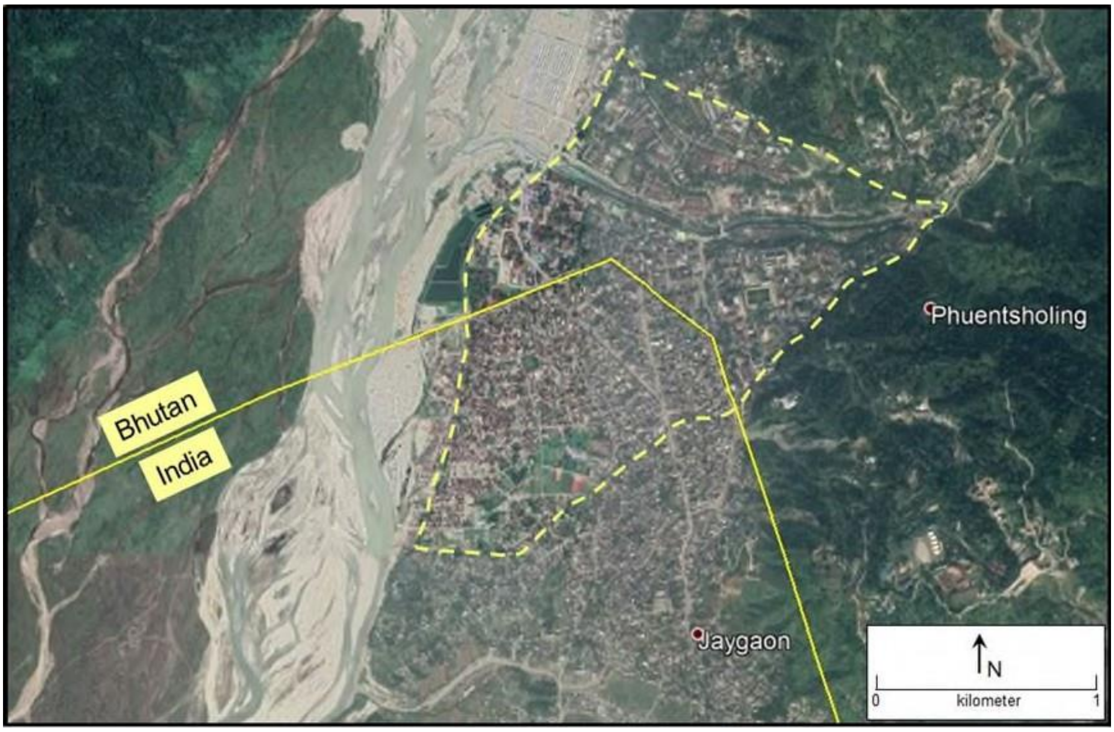


Figure 66: The alluvial fan in Phuentsholing and Jaigaon is heavily developed

Source 14: https://www.east-himalaya.com/phuentsholing.php

Past flooding in Phuentsholing has led to the channelization of the Omchhu through the city in an effort to mitigate future damages. As a result, the Omchhu's course is nearly straight across the alluvial fan from the point it exits the confined upper watershed to its confluence with the Amochhu (Figure 65). In addition, bunds flank the channel (Figure 67) along both banks to contain floodwaters within the channel and prevent inundation of the city. The hydraulic modelling confirms that even the largest modelled flows are contained within the channel except at the downstream end where the channel is not as deep and afflux upstream of the new bridge increases water surface elevations. However, by constraining the flow to a narrow-channelized flow path on an alluvial fan, where the flow would naturally spread out over a wide area, flood flow velocities are artificially elevated and the concrete and gabion armouring protecting the banks of the channelized Omchhu are failing in places (Figure 68) and are frequently reinforced and rebuilt.

Figure 67: Bunds flank the Omchhu along both banks through Phuentsholing. Note the ground elevation behind the bund is lower than the walking path on top of the bund.

Figure 68: Armor on the banks of the Omchhu have failed in places. Photo taken in 2016.

7.2 Types of hazards

In addition to the potential flow inundation initiated upstream of the new bridge as identified by the hydraulic modelling, at least four other potential morphological hazards must be considered that the hydraulic modelling is not equipped to identify. First, failure of the bank armouring and ensuing erosion could locally pass through the bunds flanking the Omchhu. While such erosion during a single event is unlikely to reach any buildings (although the new parking garage and vegetable market are just behind the bunds on the left descending bank), the loss of the bund would reduce the height of the banks constraining the Omchhu's flow (see Figure 67), potentially leading to flow inundation over a much wider area than currently anticipated by the hydraulic modelling.

Second, significant deposition in the channel could locally elevate the bed of the channel high enough for flows to overtop even the bunds flanking the channel. This concern is already captured by the sediment modelling component of the hydraulic model for the area upstream of the new bridge. The completed sediment modelling considers sediment aggradation over a series of flows but does not adequately capture the potential for the rapid introduction of sediment through landslides at the upstream limits of town (Figure 69) that could obstruct the channel and cause deposition immediately upstream.

Figure 69: A landslide at the upstream end of Phuentsholing could block the channel and cause rapid deposition upstream.

Third, land sliding can and has led in the past to the temporary damming of the Omchhu in the upper watershed (Figure 70). Even landslides and subsequent damming of the Omchhu in the confined upper watershed could lead to hazardous conditions in town. The temporary dams could rapidly store large volumes of water and sediment that can then be unleashed by the sudden failure of the landslide dams (not unlikely given the loose nature of the sediment and the pressure of the water building up on the upstream side). The wave of water moving downstream may attenuate quickly but given the channelized water course through town overtopping of the bunds and inundation far beyond what is currently identified by the hydraulic model is possible.

Figure 70: A large revegetated landslide scar in the upper watershed created a temporary landslide dam in the past as evidenced by remnants of sediment deposited in the related upstream impoundment (dashed line at top of sediment – sediment is level despite angle

Finally, all of the three identified morphological hazards above, if severe enough, can lead to a channel avulsion whereby the current position of the Omchhu would switch to a new location. This would be particularly true in the case of a landslide event, either in town or the upper watershed, where a large slug of sediment could plug the channel and leave the Omchhu to create an alternate path elsewhere through town. A channel avulsion, although unlikely in any given year, will invariably and eventually occur in the future given that the alluvial fan landform upon which Phuentsholing rests was built over time through a series of channel avulsions.

7.3 Location of hazards

Erosion through the bund that could lead to flow inundation beyond that identified by hydraulic modelling is most likely where the bund is at its lowest and/or weakest. Ongoing construction of the new road along the right descending bank of the Omchhu has led to the narrowing of the backside of the bund during construction and may be permanent given the position of the roadside drainage. At other locations along the right bank, the bund has been lowered to provide a wider flatter space for temporary shelter for construction workers (Figure 71). Consequently, almost the entire length of the bund on the right bank can be considered at greater risk of being breached by erosion compared to the left bank. On the left bank near the parking garage, bank erosion has caused a portion of the walkway on top of the bund to collapse (Figure 72), so this might be the most likely location of a breach in the bund on the left bank. Every effort should be made to reinforce these areas as construction of the road is completed to reduce the risk of a berm breach by erosion. Again, the FMC will more carefully pinpoint the exact locations of the highest risk areas.

Figure 71: Portions of the right bank bund along the Omchhu was lowered to place shelters for laborers constructing the new road.

Figure 72: The bund on the left bank of the Omchhu has been narrowed by erosion near the new parking garage. Photo taken in 2016 prior to garage construction.

The likely location of hazards associated with sedimentation will be harder to pinpoint but some general suppositions can be made. The hydraulic modelling established that sedimentation would worsen inundation problems associated with afflux upstream of the new bridge, although no additional areas of overtopping the existing channel are identified by the sediment modelling. This must be understood in the context that typically two classes of floods occur in alluvial fan settings, especially in the subtropical climate of Phuentsholing: water-rich floods and sediment-rich floods. Water-rich floods predominate during smaller flood events that might have a recurrence interval of 1 to 10 years as such events are generally insufficient to initiate landslides that can introduce large quantities of sediment. Hydraulic modelling, even those with sediment modelling included, are most accurate for these types of flood events but will generally be less accurate for sediment-rich floods given the potentially dramatic changes in bed elevation during the course of a single flood. Water- rich floods also carry sediment and can cause deposition, as demonstrated by the modelling, but are unlikely to cause rapid and severe sedimentation.

Sediment-rich floods will be generated by larger floods with a recurrence interval typically greater than 10 years as such events are more likely to cause landslides in the upper watershed that will rapidly introduce vast quantities of sediment to the flood waters. As a result, the Omchhu might experience significant aggradation during these larger flood events with subsequent smaller water-rich floods actually downcutting through these sediments along the upper part of the Omchhu through town (i.e., upstream of the old bridge in the centre of town). (The falling limb of a sediment-rich flood can also result in downcutting and remobilization of sediment similar to a separate water-rich flood.) This remobilized sediment will be redeposited further downstream such that all floods will likely result in sedimentation at the new bridge, reinforcing the need identified in the hydraulic model for frequent removal of the aggrading sediment, potentially several times each monsoon season. During the large sediment-rich floods, deposition can be expected anywhere along the Omchhu but will be most significant wherever flow velocities are even slightly reduced due to minor bends in the channel (e.g., near the new parking garage), constrictions in the channel (e.g., both bridges but especially the new bridge), or other channel obstructions develop (e.g., landslide dams), and large concrete slabs falling into the river as the result of undermining of the bank armour (Figure 73).

Figure 73: Concrete slabs collapsing into the channel due to undermining of the bank armour could constrict the channel and lead to upstream deposition. Photo taken in 2017.

While landslides themselves will be restricted to the upstream end of town and the upper watershed, hazards associated with them (i.e., inundation and deposition) can result anywhere through town (as such landslides can initiate significant deposition in the channel even downstream of their occurrence). Scars of past landslides are prevalent on the steep slopes at the upper edge of town (Figure 5.5) and further upstream higher in the watershed (Figure 70). The risk of further landslides from the same locations is high as considerable loose sediment and weathered bedrock are still present. Identifying new locations of landslides is more difficult but could be achieved through careful mapping that locates areas where steep slopes, the concentration of flow in swales on the slopes, and loose sediment or weathered bedrock all occur together. The most dangerous situation would be if landslides occur in narrow sections of the Omchhu valley (and could easily dam the valley) immediately downstream of wider low gradient sections of the valley (allowing a greater volume of water to be impounded upstream of the dam). Fortunately, such wide areas do not appear prevalent in the upper watershed, thus minimizing the risk of a catastrophic dam break flood, but further reconnaissance is warranted during the FMC to better define this risk.

The risk of channel avulsion along any river or alluvial fan is greatest if deposition occurs where the channel banks are lowest such that flow can escape more easily from the existing channel. In addition to overtopping, an avulsion will only occur if the overflow becomes concentrated in low areas on the floodplain or alluvial fan surface, and, thus, capable of generating enough stream power to carve a new channel back towards the point where flow is overtopping the existing channel. Given this, the highest risk of an avulsion is currently just upstream of the new bridge, which the hydraulic modelling shows is the only location where overtopping of the channel occurs under existing conditions. Since this location is downstream of most of the development in town, damages and loss of life, while still possible, would be less than an avulsion occurring further upstream. The most catastrophic avulsion would be one initiated at the upstream end of town as the newly forming channel would extend for a much greater distance and pass through more densely populated and developed areas such that a higher potential for significant damage and loss of life exists. Unfortunately, despite the high banks in this area, an avulsion at this location is possible given the potential for a landslide dam to form or for elevated water levels to result from a landslide-dam break upstream. The avulsion risk will be highest where the overtopping of the bank occurs near a depression on the alluvial fan surface where the flow exiting the channel can become concentrated. This currently seems more likely on the right bank compared to the left bank given the lowering and weakening of the bund due to construction of the new road with flows likely to concentrate along the road itself that appears lower than the immediately surrounding areas.

7.4 Conclusion

Hydraulic modelling has a limited ability to identify the full panoply of potential fluvial hazards in Phuentsholing that rests on an alluvial fan where rapid deposition and/or erosion can lead to dramatic and rapid changes in channel position during a single flood event. The channelization of the Omchhu reflects an attempt to control these hazards and the hydraulic modelling is useful for identifying where floods may overtop this confined channel. However, significant bank erosion, rapid deposition, or flood wave due to a landslide-dam break upstream could lead to a catastrophic channel avulsion whereby the channel could relocate elsewhere through the town in the midst of a large flood. Quantifying the amount of sediment available during a large flood is difficult without extensive study but given the steep unstable hillslopes of the upper watershed upstream of town, the likelihood exists that the sediment supply will exceed the transport capacity of the stream such that rapid deposition should be expected wherever the transport capacity is reduced even slightly (e.g., at the new bridge). Predicting the location of such events is difficult with hydraulic models but the impacts of such events could be modelled by making assumptions regarding the magnitude of such changes. Model runs can be made of altered conditions that mimic bank erosion breaching the bunds in certain locations or rapid deposition infilling the channel to various heights. The resulting inundation areas under these scenarios could be used to identify where flow could become concentrated, thus allowing the location of potential avulsions to be identified (but only if detailed topography of the entire low portion of town is available). Hydraulic modelling could also be useful for predicting the impacts of dam break floods by reconstructing the size of past landslide dams (Figure 70) and mimicking the sudden release of the water impounded behind such dams (although this would similarly require detailed topography of portions of the upper watershed). Given the limitations of hydraulic modelling to adequately identify the potential locations of rapid changes that might result from bank erosion, deposition, and dam- break floods at the new bridge and elsewhere, a plan must be developed to take quick action to save lives and property as will be developed by the FMC from PTDP.

8 Geological & Geotechnical Assessment

A comprehensive geotechnical study was carried out on geology and geotechnical engineering. At the Preliminary Investigation stage, Desk study and reconnaissance survey was carried out. After the desk study, field study visit was made for a more detailed study including a check on the findings of the initial study and to collect field data wherever necessary. Geotechnical mapping was carried out by dividing the material into different geotechnical units. Slope Stability Probability Classification (SSPC) form developed by Dr. Hack (ITC, the Netherlands) was used to collect the basic data on rock and to assess the rock slope stability.

In terms of the field investigation, Standard Penetration Tests (SPT), Plate Load Test (PLT), Portable Penetrometer Test (PPT), and Seismic Refraction Tomography (SRT) test were carried out along the entire project location stretch. This was supplemented by the laboratory test of the collected materials. This section describes the summary of the Geotechnical Report, which is a standalone report for the project.

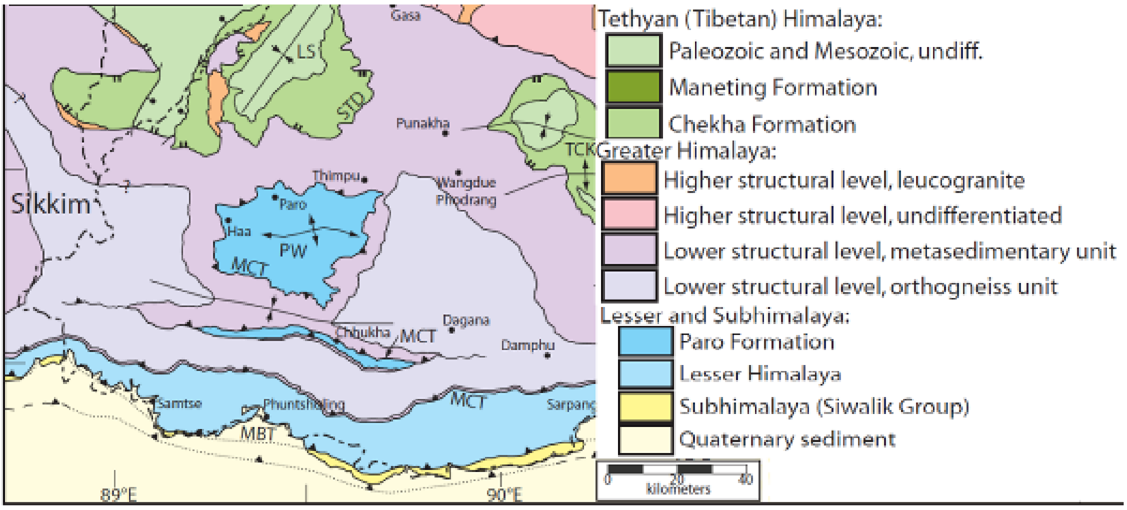
8.1 General Geology

On the basis of tectono-stratigraphic position, gross lithological assemblage and the grade of metamorphism, Bhargava et. al (1995) established the regional geological set-up of Bhutan Himalaya as shown in Table 34 below.

Table 34: Litho-stratigraphic set up of the Bhutan Himalayas (Bhargava ed., 1995)

<table>
<tr><th colspan="2">Quaternary succession</th><th>Recent sediments mostly fluvial materials. With clay beds.</th></tr>
<tr><td colspan="3">-------------------------------------MFT (Main Frontal Fault) ---------------------------------------</td></tr>
<tr><td colspan="2">Siwalik Group</td><td>Sandstone, siltstone, shale, clay, boulder bed (semi-consolidated conglomerate).</td></tr>
<tr><td colspan="3">-------------------------------------MBT (Main Boundary Thrust) -----------------------------------</td></tr>
<tr><td colspan="2">Damuda Subgroup</td><td>Sandstone, siltstone, shale, coal beds</td></tr>
<tr><td colspan="3">-------------------------------------Baxa Thrust--</td></tr>
<tr><td rowspan="3">Baxa Group</td><td>Pangsari Formation</td><td>Dolomite, Quartzite and Phyllite and local conglomerate</td></tr>
<tr><td>Phuentsholing Formation</td><td>Variegated Phyllite with white, purple and grey quartzite</td></tr>
<tr><td>Manas Formation</td><td>Light grey dolomite, limestone, grey and carbonaceous phyllite</td></tr>
<tr><td colspan="3">------------------------------------ Shumar Thrust ---</td></tr>
<tr><td colspan="2">Shumar Formation</td><td>Interbanded quartzite and phyllite with limestone and basic sills, garnetiferous mica schist, marble, calc-Sandstone and graphite schist</td></tr>
<tr><td colspan="3">-------------------------Jaishidanda Thrust ---</td></tr>
<tr><td colspan="2">Jaishidanda Formation</td><td>Biotite-garnet-staurolite schist with slivers of granite gneiss.</td></tr>
<tr><td colspan="3">-------------------------------Thimphu Thrust ---</td></tr>
<tr><td colspan="2">Thimphu Group</td><td>Augen gneiss, banded gneiss, granite gneiss, mica schist and quartzite</td></tr>
<tr><td colspan="3">------------------------------- Unconformity Thrust --</td></tr>
<tr><td colspan="2">Tethyan Sequence (Chekha Formation)</td><td>Quartzite and phyllite</td></tr>
</table>

Figure 74: Regional Geology of the Study Area (after Sean Long, et al.)



The study area falls under Lesser Himalayan formation rocks placed under Lesser and Sub-Himalaya Group as per Sean Long.

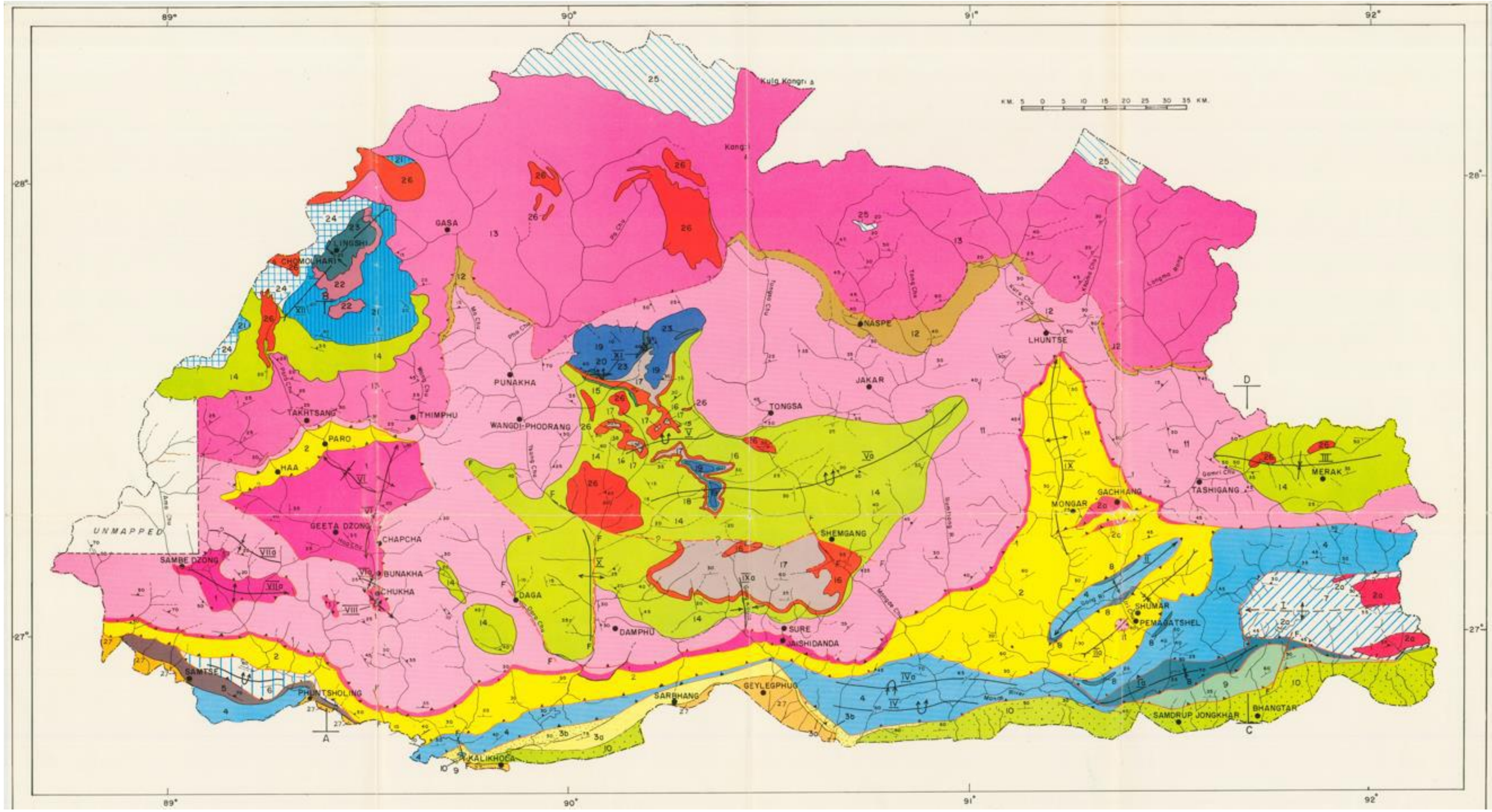

GASA
LINGSHI
CHOMOLHARI
PUNAKHA
THIMPHU
TAKHTSANG
PARO
HAA
WANGDI-PHODRANG
TONGSA
JAKAR
NASPE
LHUNTSE
MONGAR
GACHKANG
TASHIGANG
MERAK
SHEMGANG
GEETA DZONG
CHAPCHA
SAMBE DZONG
BUNAKHA
CHUKHA
DAGA
DAMPHU
SURE
SAISHIDANDA
SHUMAR
PEMAGATSHEL
SAMTSE
PHUNTSHOLING
SARBHANG
GEYLEGPHUG
KALIKHOLA
SAMDRUP JONGKHAR
BHANGTAR
UNMAPPED
Kula Kongri
89°
90°
91°
92°
28°
27°

Figure 75: Geological Map of Bhutan by Bhargava, et. al (1995)

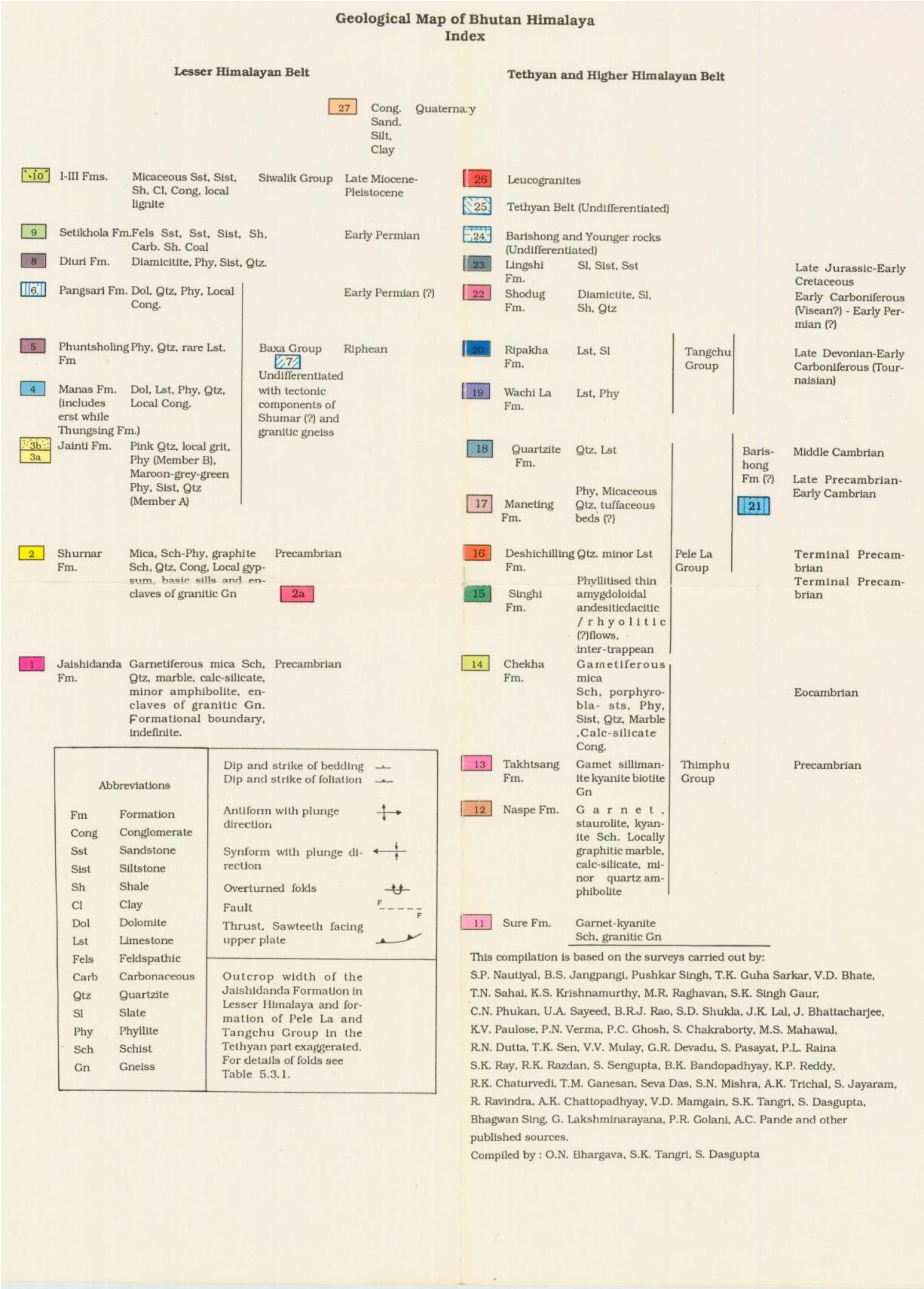
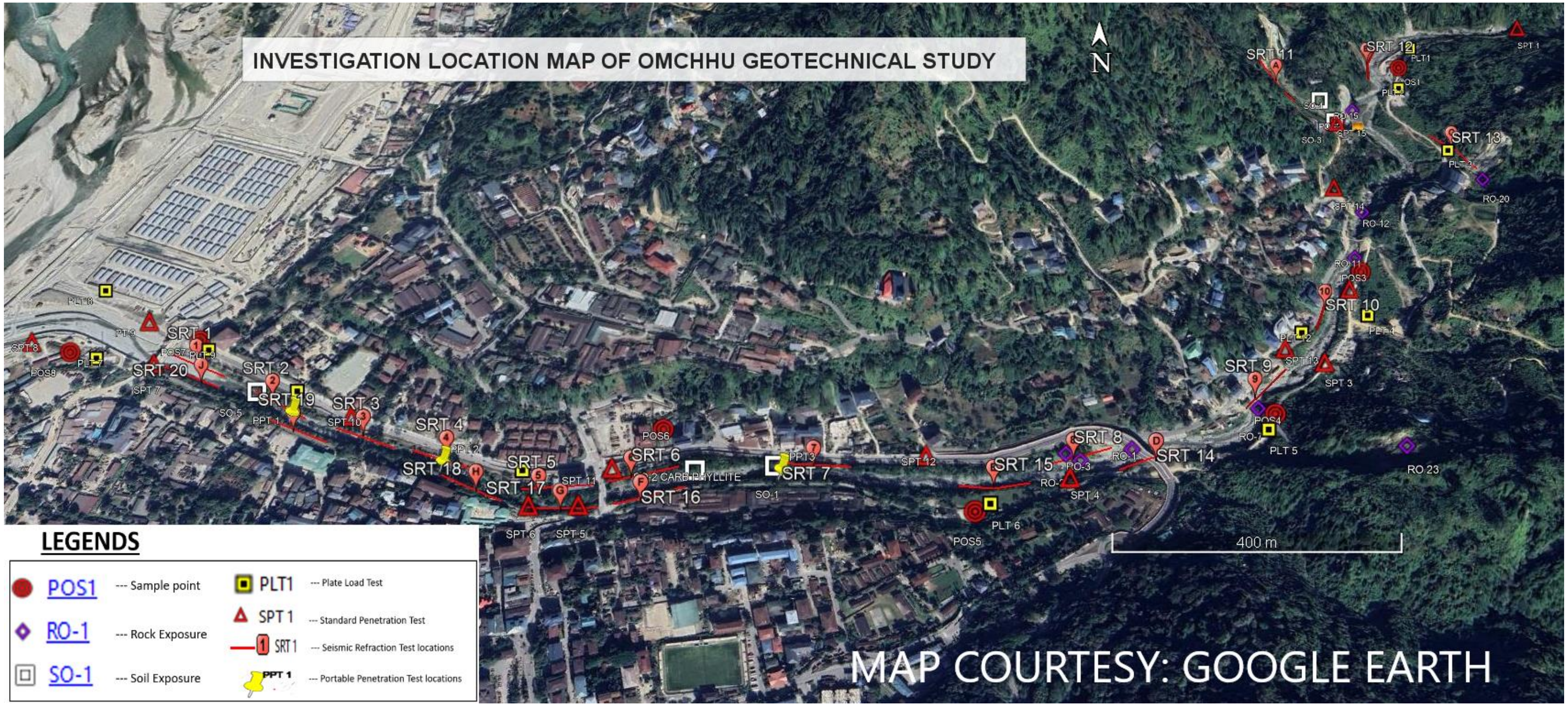


Figure 76: Investigation Location Map of Geotechnical Study



8.2 Soil Slope Characterization

In all 5 soil slopes were characterized at five different locations. Each observation is discussed below in some details. Detail investigation location map is shown in Figure 76.

8.2.1 Slope Exposure - 1

This exposure was observed on the left bank of Omchhu stream below Archery field and about 200 metres above the old bridge. Medium dense grey coloured sandy gravelly layer was observed. This exposure is located at 26.863855° North latitude and 89.3869665° East longitude at an approximate elevation of 219m above mean sea level.

Figure 77: Photographs of Slope 1

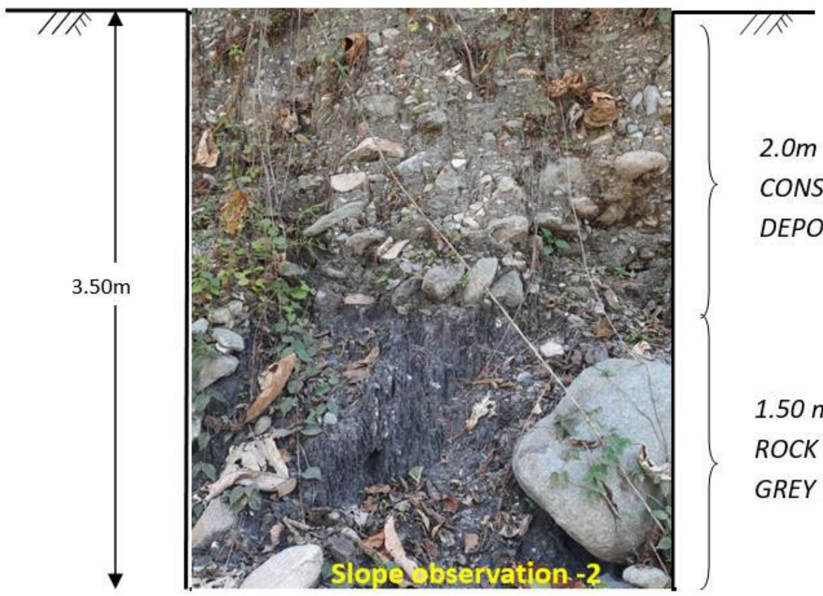
The top part of about 1.50m is grey coloured sandy gravel with organic contents. It is medium compacted, equidimensional with rough texture. It is composed of phyllite and quartzite and has a thick bedding size. It has heterogeneous structure and is an alluvial type deposit. Below this top layer is a 1.00m thick grey coloured gravelly layer with low sand percentage. It has rough texture, loosely compacted and has heterogeneous structure. The bedding size is thick with wide spaced discontinuity and it is composed of quartzite and phyllite rocks. The particle angularity is sub-rounded. Details of particle size distribution and sieve analysis chart is appended as A.1 under main geotechnical report.

8.2.2 Slope Exposure - 2

This slope observation was near the old bridge also on the left bank of Omchhu. This pit is located at 26.86382737° North latitude and 89.38569553° East longitude at an approximate elevation of 210m above mean sea level.

Top soil layer is up to 2.00m thick. It is grey in colour and has sub rounded angularity. It displays rough texture, equi-dimensional particle form, mildly compacted sandy gravel layer. The bedding size is thick and has heterogeneous structure. The material mostly composed of quartzite and phyllite. Below this layer is the bed rock exposure of carbonaceous phyllite, highly friable. It is exposed only up to a metre from the river level. Overlying this layer is an alluvial deposit consisting of sandy gravel to gravelly sand deposits. Details of particle size distribution and sieve analysis chart is appended as A.2 under main geotechnical report.

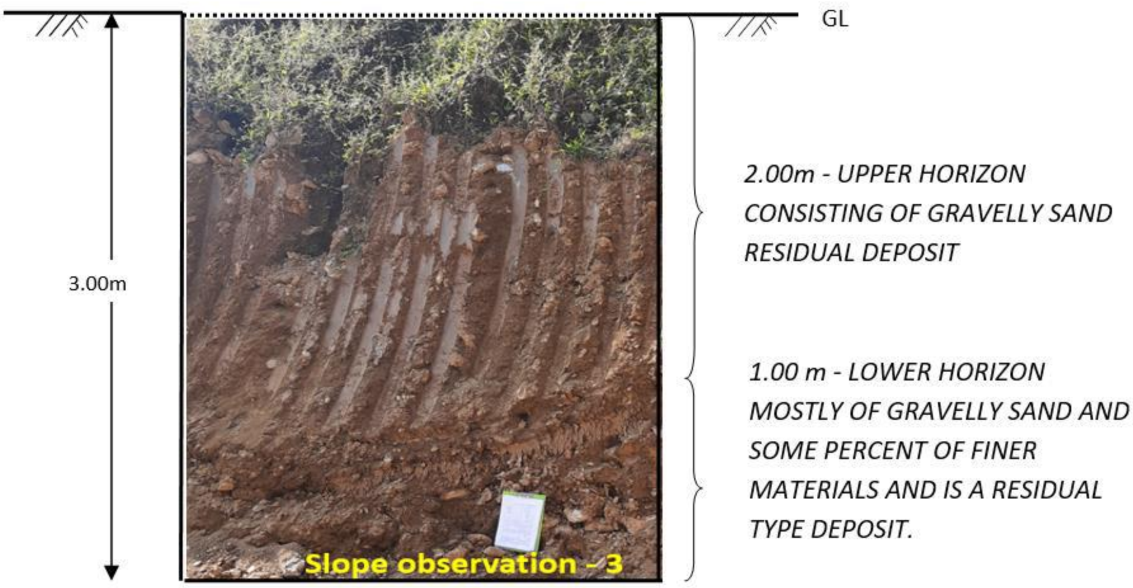
Figure 78: Photograph of Slope-2



8.2.3 Slope Exposure - 3

This pit was at the upper western part of the study area, just above the confluence of Namantri and Omchhu, beside the cowshed. This exposure is located at 26.8706612° North latitude and 89.39764576° East longitude at an approximate elevation of 279 m above mean sea level. Top soil layer is up to 2.00m thick. It is brown in colour and has rounded angularity. It displays smooth texture, equi-dimensional particle form, mildly compacted sand layer with very low percentage of coarser fractions. The bedding size is very thick and has homogeneous structure. The material mostly composed of quartzite.

Figure 79: Photograph of Slope - 3



The lower layer is a 1.00m thick brownish grey coloured gravelly sand layer. This layer has rough texture with thick bedding size and is mildly compacted. The gravels are elongated and sub angular. The boulders are of mostly quartzite. This shows the properties of a residual type deposit. Details of particle size distribution and sieve analysis chart is appended as A.3 under main geotechnical report.

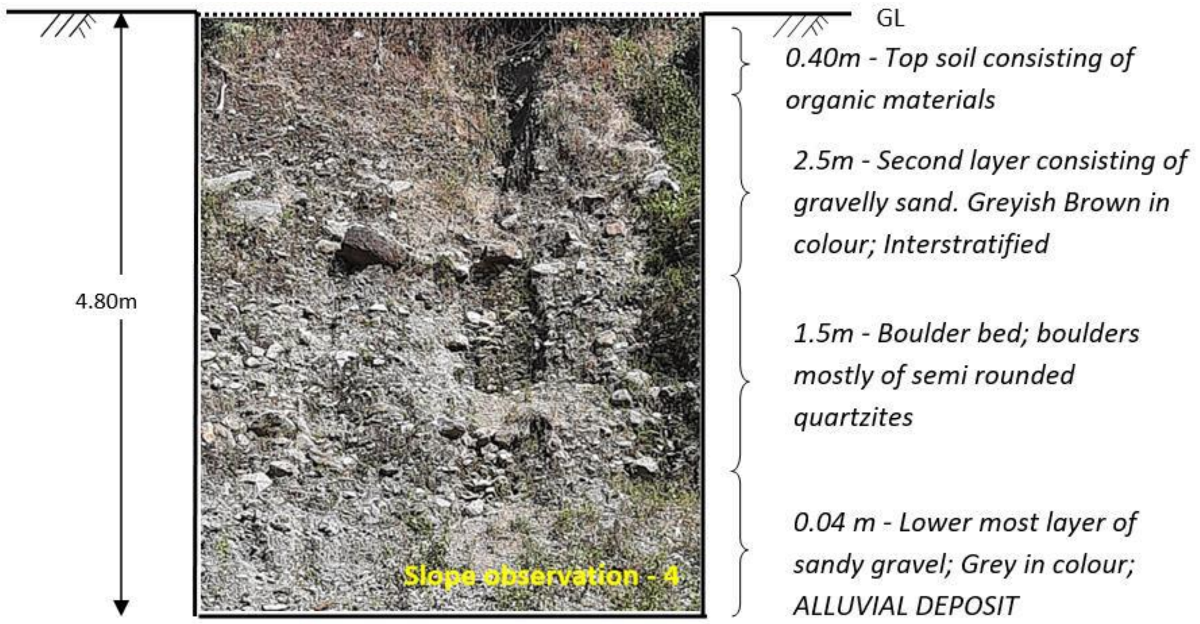
8.2.4 Slope Exposure - 4

This slope material observation was at the western flank of Bogataybari, on the Namantri stream left bank. This is located at 26.87120366° North latitude and 89.39751098° East longitude at an approximate elevation of 282 m above mean sea level.

Top soil layer is up to 0.04m thick. It consists of dark colour soil with sub-rounded angularity and organic matters like roots and rotten leaves of plants. It displays smooth texture, flat and elongated particle form, loosely compacted sandy layer. The bedding size is medium and has heterogeneous structure. The material mostly composed of quartzite and phyllite. Below this layer is a 2.50m thick greyish brown coloured gravelly sand layer. This layer has rough texture with very thick bedding size and is medium compacted. This layer contains significant very low percentage of gravels/boulders. The boulders are elongated and sub angular. The boulders are of mostly quartzite.

Below this layer is a 1.50m thick brownish grey coloured sandy gravel layer. This layer has rough texture with thick bedding size and is densely compacted. This layer contains significant high percentage of gravels/boulders. The boulders are oblate to sub rounded. The boulders are also of mostly quartzite. The lowest exposed layer is a 0.40m thick grey coloured sandy gravel layer. This layer has pitted texture with medium bedding size and is also densely compacted. This layer contains significant high percentage of gravels. The gravels are sub rounded. The boulders are also of mostly quartzite. This exposure shows the properties of an alluvial type deposit. Details of particle size distribution and sieve analysis chart is appended as A.4 under main geotechnical report.

Figure 80: Photograph of Slope - 4

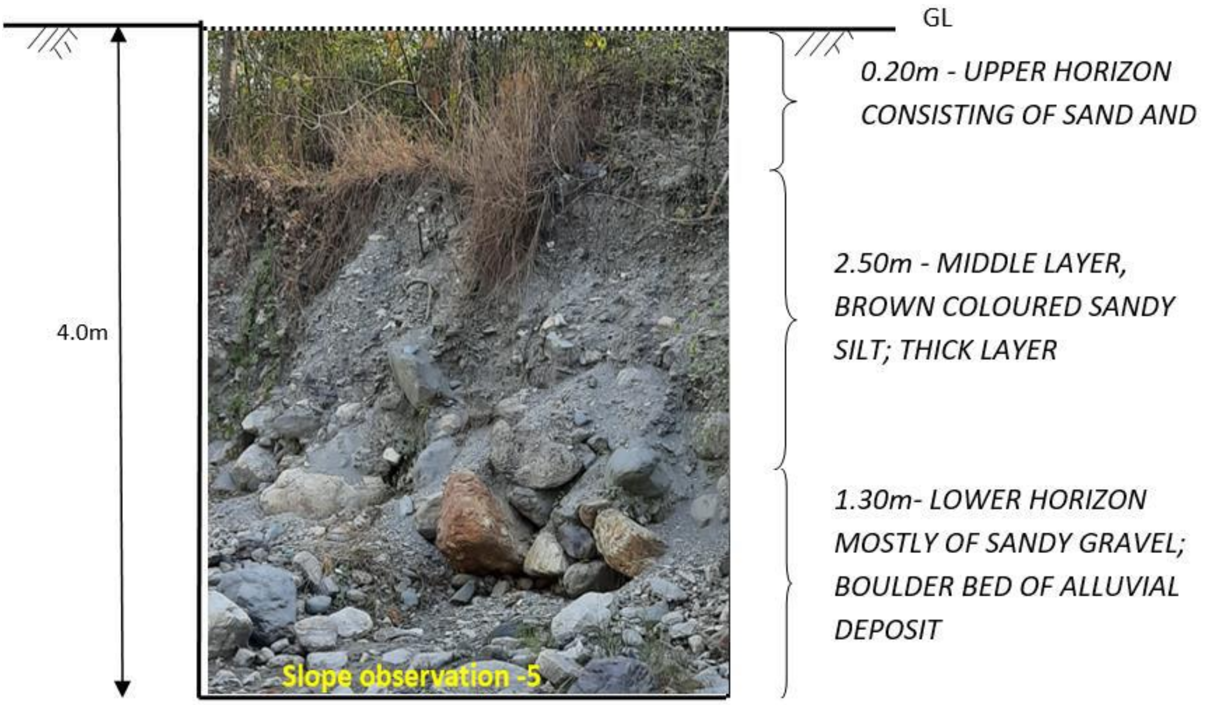


8.2.5 Slope Exposure - 5

This pit was at the lower most part of the study area on the right bank of Omchhu, about 300 metres from Omchhu II bridge. This pit is located at 26.840134° North latitude and 89.432585° East longitude at an approximate elevation of 199 m above mean sea level.

Top layer is up to 0.20m thick. It is dark in colour and has sub-rounded angularity. It displays smooth texture, flat and elongated particle form, loosely compacted sandy layer. The bedding size is thin and has homogeneous structure. The material mostly composed of quartzite and phyllite. Below this layer is a 2.50m thick brownish grey coloured gravelly sand layer. This layer has smooth texture with very thick bedding size and is medium compacted. It has heterogeneous structure and is a transported soil. The particle form is flat and elongated. The third layer is 1.30m thick. It is grey in colour and has rounded to sub-rounded angularity. It displays rough texture, elongated particle form, loosely compacted sandy gravel layer. The bedding size is thick and has heterogeneous structure. The material mostly composed of quartzite and phyllite. This shows the properties of an alluvial type deposit. Details of particle size distribution and sieve analysis chart is appended as A.5 under main geotechnical report.

Figure 81: Photograph of Slope - 5



8.3 General Observation

Apart from these specific observations, collecting samples and geotechnical logging of soil, observations were also made on the road cut slopes and natural slopes formed by both alluvium and other soil types.

Most of the river basin has alluvium type deposit, which is usually sorted mass with rounded to sub-rounded boulders. Some slopes are barren or has thin soil cover with under lying bed rock of either phyllite or quartzite. The most prominent rock unit present in the study area is grey to greenish grey, very fine grained phyllite in the lower part and grey to light grey medium to fine grained quartzite in the upper part of the study area. In the upper part where quartzite is the prominent rock type, minor bands of phyllite occurs with the quartzite.

In general, the soil composition in Omchhu area is gravelly sand to sandy gravels with variations in cobbles to gravels fractions. It was generally observed that the soil is not densely compacted and also contains high percent of gravels, which are rounded to sub rounded in shape, which suggests that the deposit type is mostly alluvial.

8.4 Field Investigation

The Investigation by Standard Penetration Test (SPT) method and Plate Load Test (PLT) method was carried out as part of detailed geotechnical investigation. Fifteen numbers of SPT tests and adequate numbers of PLT tests were planned and executed in the field. The test locations were uniformly distributed and the planning was done taking the viewpoints of the client in all respects. as follows.

The SPT test was conducted at a depth of 1.5m from the natural ground level and PLT test at 1.00 m depths. We have also maintained test record, soil type, ground conditions, Test No./location ID, Name of locations, GPS coordinates and date of tests carried out. The test results are recorded in ArcGIS as a point feature with its results as attributes and stored as a shapefile or geodatabase format.

8.4.1 Standard Penetration Tests

The guidelines outlined in IS: 2131 – 1981 was followed to carry out the Standard Penetration Test. The SPT is a penetration test, by which the blows are counted; necessary to let a standardized sampler a (split-spoon sampler 50mm) penetrates 0.30m into the soil, using a standardized percussion hammer with a mass of 63.5kg with a dropping distance of 0.75m.

Figure 82: SPT in Progress

The blows are counted over traject of 0.15m, 0.15m and 0.15m by which the first traject does not contribute to the test as it is considered as disturb due to drilling. The next 0.30m is penetrated into undisturbed soil which is considered for the calculation of bearing capacity. In the dense or hard formation, the maximum number of blows is generally limited to 50 or 60.

Standard penetration test conducted by means of the split spoon, furnishes data about resistance of the soils to penetration which can be used to evaluate standard strength data, such as N values (number of blows per 30 cm of penetration using standard split spoon) of the soil.

Determination of the bearing capacity of the founding strata

The SPT-N value computed and corrected was used to estimate the allowable bearing pressure of a soil for a given value of allowable settlement. The Terzaghi equation given below is used to calculate the net bearing pressure for a founding stratum.

qu = c’Nc + Df Nq + 0.5 B Nγ---------- equation 1

By the modification of above equation, equations for square and circular footings are also given and they are. For square footing

qu = 1.2 c’Nc + Df Nq + 0.4 B Nγ--------- Equation 2

For circular footing

qu = 1.2 c’Nc + Df Nq + 0.3 B Nγ------------ Equation 3

Using the formula in Equation 2 for a circular footing and the relationship chart between Standard Penetration Resistance (SPT-N) and Ultimate bearing pressure (qu) for different values of B (Width of footing) (Terzaghi & Peck, 1948), is shown in the table below.

Table 35: The Ultimate Bearing Capacity for N values and different footing sizes

	Test ID	Field SPT Value ‘N’	Bulk Unit Weight (kN/m^3)	Depth, D_f (m)	Overburden Pressure, $\sigma'=\gamma.D_f$ (kN/m^2)	Correction for Overburden Pressure N'=4*N/(1+0.04 P_0)	φ° (After Peck, Hanson and Thornburn, 1974)	φ' =$\tan^{-1}$(2/3tanϕ)(Effective friction angle)	N_q	N_γ	Width of the foundation, B (m): 1	1.5	2	2.5	3	3.5	4
SL. No.	Correlation analysis										Ultimate bearing capacity = q_{ult} (kN/m^2)						
1	SPT-01	14	17.80	1.50	26.70	27	35.50	25.40	11.20	11.23	379.00	418.98	458.96	498.93	538.91	578.89	618.87
2	SPT-02	19	19.20	1.50	28.80	35	37.50	27.00	13.20	14.47	491.29	546.85	602.42	657.98	713.55	769.11	824.68
3	SPT-03	17	18.30	1.50	27.45	32	36.50	26.26	11.95	12.68	420.85	467.25	513.66	560.07	606.48	652.89	699.30
4	SPT-04	15	18.00	1.50	27.00	29	35.70	25.60	11.28	11.35	386.28	427.14	468.00	508.86	549.72	590.58	631.44
5	SPT-05	18	18.50	1.50	27.75	34	37.00	24.21	9.61	9.62	337.87	373.46	409.05	444.65	480.24	515.84	551.43
6	SPT-06	8	15.00	1.50	22.50	17	32.50	23.00	8.66	8.20	244.05	268.65	293.25	317.85	342.45	367.05	391.65
7	SPT-07	20	19.80	1.50	29.70	37	38.00	27.51	13.95	15.60	537.87	599.64	661.42	723.20	784.97	846.75	908.52
8	SPT-08	10	16.00	1.50	24.00	20	33.00	23.41	8.89	8.60	268.40	295.92	323.44	350.96	378.48	406.00	433.52
9	SPT-09	18	18.50	1.50	27.75	34	37.00	24.21	9.61	9.62	337.87	373.46	409.05	444.65	480.24	515.84	551.43
10	SPT-10	17	18.30	1.50	27.45	32	36.50	26.26	11.95	12.68	420.85	467.25	513.66	560.07	606.48	652.89	699.30
11	SPT-11	17	18.30	1.50	27.45	32	36.50	26.26	11.95	12.68	420.85	467.25	513.66	560.07	606.48	652.89	699.30
12	SPT-12	18	18.50	1.50	27.75	34	37.00	24.21	9.61	9.62	337.87	373.46	409.05	444.65	480.24	515.84	551.43
13	SPT-13	19	19.20	1.50	28.80	35	37.50	27.00	13.20	14.47	491.29	546.85	602.42	657.98	713.55	769.11	824.68
14	SPT-14	7	14.50	1.50	21.75	15	27.00	18.76	5.66	4.47	149.03	161.99	174.96	187.92	200.88	213.85	226.81
15	SPT-15	14	17.80	1.50	26.70	27	35.50	25.40	11.20	11.23	379.00	418.98	458.96	498.93	538.91	578.89	618.87

Since the soil type is coarse grained sand with very high percent of gravels and boulders and surface to near surface bed rock, cohesion is considered zero. This value was also obtained from the Direct Shear Box test, so it is safe to consider cohesion to be ‘0’, though few samples have cohesion slightly above 0.

The Ultimate Bearing Capacity should be divided by a suitable factor of safety to arrive at the safe bearing capacity depending on the type of structure and the risk it poses to life and property.

8.4.2 Plate Load Test

The plate-bearing test was carried out to establish the stress-deformation relation of the top layers. A circular plate with a diameter, not smaller than 0.30 m and not larger than 0.75 m is placed on a flat area on the surface. The load is applied in stages, load controlled, and in most cases released after each load-step to the value of the preceding load, while corresponding deformations are measured and recorded.

TEST PROCEDURE

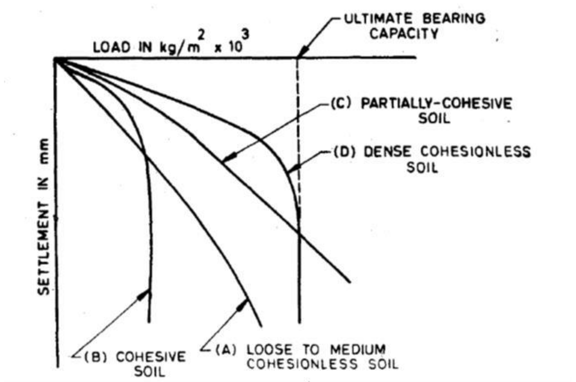
Test pit of 2.5 x 2.5 m and depth 1.0 m was dug for conducting the plate load test. The test plates used was a 45 cm square. Plate load test has been carried out as per I.S. code 1888(1982).

Figure 83: Set-up for Plate Load Test:

DETERMINATION OF ULTIMATE BEARING CAPACITY/SAFE BEARING PRESSURE/SETTLEMENT

Shape of the Load/Settlement Curve- A load settlement curve is plotted out to arithmetic scale. From this load settlement curve, the zero correction which is given by the inter-section of the early straight lines or the nearly straight-line part of the curves with zero deadline is determined and subtracted from the settlement readings to allow for the perfect seating of the bearing plate and other causes.

Figure 84: Load Settlement Curve determination as per IS-1888 (1982)



The Ultimate Bearing Capacity from the result obtained from the field test is as shown in the table below.

Table 36: Ultimate Bearing Capacity from PLT

Test ID	Ultimate bearing capacity (qult)
PLT-01	Not determined
PLT-02	275 kN/m2
PLT-03	315 kN/m2
PLT-04	Not determined
PLT-05	325 kN/m2
PLT-06	220 kN/m2
PLT-07	360 kN/m2
PLT-08	310 kN/m2
PLT-O9	320 kN/m2
PLT-10	251 kN/m2
PLT-11	270 kN/m2
PLT-12	320 kN/m2

The Ultimate Bearing Capacity should be divided by a suitable factor of safety to arrive at the safe bearing capacity depending on the type of structure and the risk it poses to life and property.

8.4.3 Portable Penetration Test (PPT)

As laid out in the manual by Susumo Sato (Sato, 2003), with this test, different soil parameter interpretation and classification can be done from PPT results.

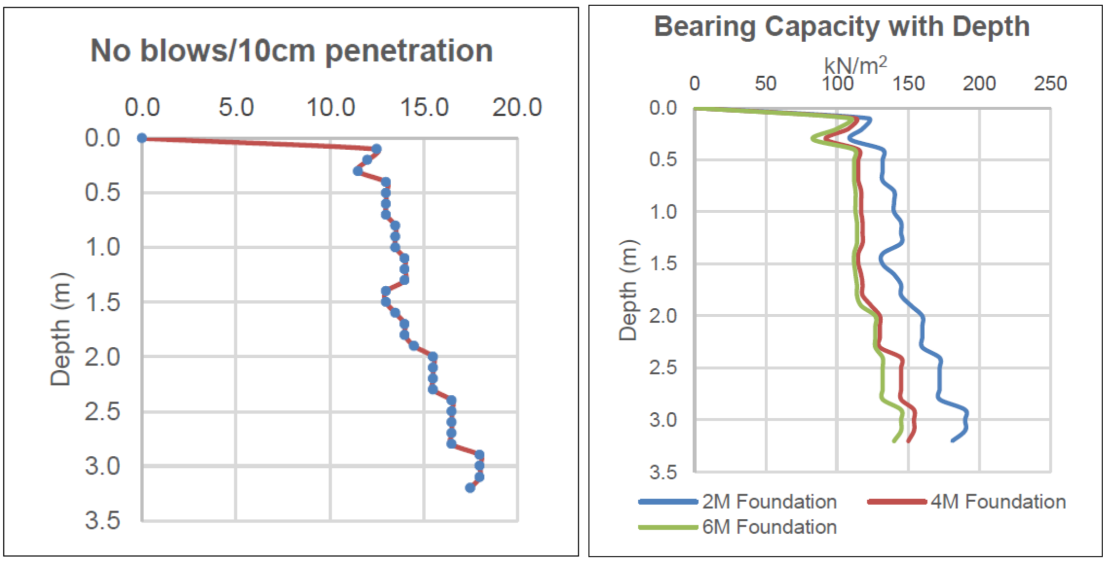
1. Some studies have shown that it is possible to classify geological layers by using “Nc-value”.
2. The relation between “Nc-value” by PPT and “N-value” by Standard Penetration Test (SPT) is given by the following expressions. Nc = (1~3) N

However, it is better to use as “Nc=N” because there is a possibility of obtaining a larger value than the actual one in the case of “Nc=3N”.

PPT – 1

In the case of PPT-1, the bearing capacity gradually increases though with light fluctuations till 1.30 metres depth and then slightly decrease. After this, the ‘N’ values gradually increase with minor fluctuations and again reaches maximum at 2.90 metres depth. The ‘N’ value reading at 2.90 metres depth is 21, which gives the maximum allowable bearing capacity of 190 kN/m2 for 2 metres foundation width, 154 kN/m2 for 4 metres width and 145 kN/m2 for 6 metres width. The details are shown in Figure 85 below. The maximum penetration depth achieved was 3.1 metres.

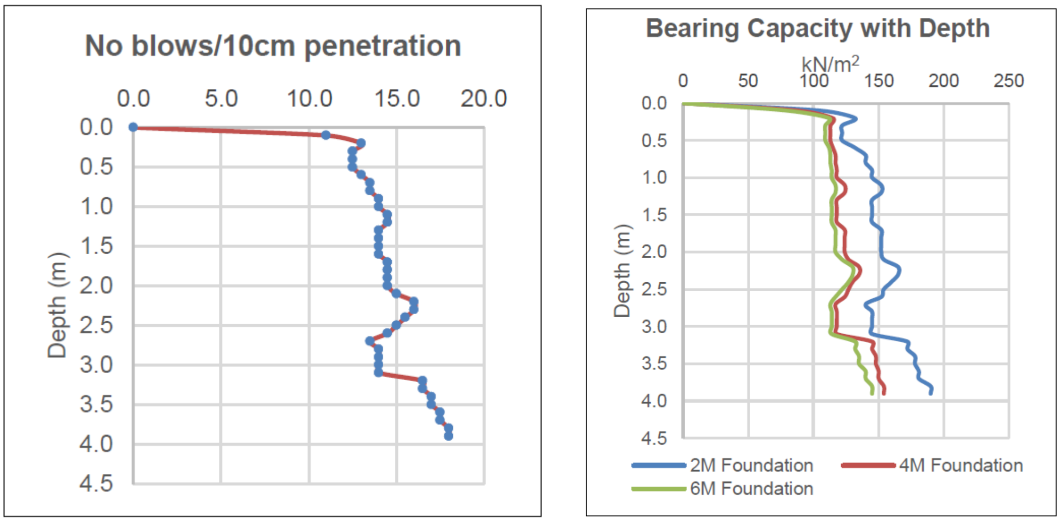
Figure 85: PIT-1 PPT 'N' values and Bearing capacity (in kN/m2)



PPT – 2

In the case of PPT-2, the highest 'N' obtained was 21 at 3.8 metres depth. The maximum allowable bearing capacity for this 'N' value at this depth is 190 kN/m2 for 2 metres foundation width, 154 kN/m2 for 4 metres width and 145 kN/m2 for 6 metres width. It is seen that bearing capacity is generally low (N>21) for infra structure development works. Bearing capacity can be increased by increasing the footing sizes, depth of foundation, draining out water seepages (maintaining the foundation dry). The detail of the PPT test is as shown in Figure 86 below.

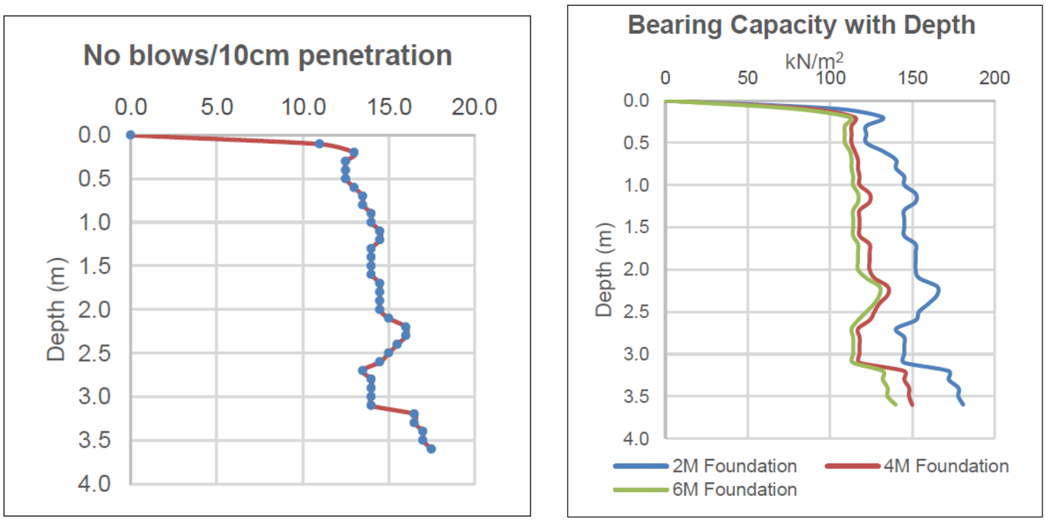
Figure 86: PIT-2 PPT 'N' values and Bearing capacity (in kN/m2)



PPT – 3

In the case of PPT-3, the bearing capacity gradually increases though with light fluctuations till 2.20 metres depth and then again reaches the minimum at 2.70 metres depth. After this, the 'N' values gradually increase and again reaches maximum at 3.60 metres depth. The 'N' value reading at 3.60 metres depth is 20, which gives the maximum allowable bearing capacity of 181 kN/m2 for 2 metres foundation width, 150 kN/m2 for 4 metres width and 140 kN/m2 for 6 metres width. The details are shown in graph below. Further penetration after this depth was not possible due to obstruction by boulders.

Figure 87: PIT-3 PPT 'N' values and Bearing capacity (in kN/m2)

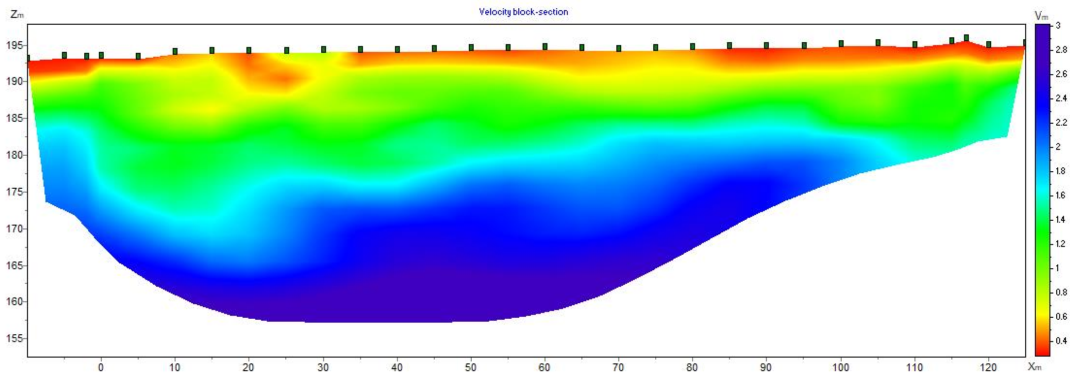


8.4.4 Seismic Refraction Tomography Test (SRT)

The Refraction Seismic Survey was conducted at 20 different locations of the project area for Omchhu Geotechnical study. The purpose of the survey was to determine and estimate the thickness of overburden, underlying fractured and sound (compact) rock system based on the seismic waves through these layers generated by the Seismic Refraction Technique.

Seismic Refraction survey was conducted using 24 channel Seismograph (RAS-24) with 24 geophones and inter Geophone spacing of 5m interval have been kept according to the field condition and on the basis of available ground stretch with an overlapping of 12 geophones. The lines were laid on the ground as per the plan of the Project area. With the help of GPS, Seismic Refraction locations were marked on the ground provided with pegs.

Figure 88: SRT Test Result for Profile No. 1



APECS Consultancy	PASIGEA24
Phuentsholing Thromde	Rohit Adhikari, Bikash Nepal
Omchhu River	RMS error = 1.64/1.6%
SRT 1/Revision of DPR for Omchhu River	April 2022

In all the 20 profiles, four to five layers are deciphered in all the seismic lines, using pit log data seismic P-wave velocities are calibrated with the type of sub-surface material. However, it is expected that the first layer with compression velocities in the range of 400 m/s to 1800 m/sec could be related to unconsolidated overburden materials as Silt, Sand and boulders and the lower layer with recorded velocity in a range of 1800 m/s to 4000 m/s indicates the presence of bed rock, consisting of highly weathered and fractured phyllite or quartzite.

The detailed SRT study report is in the form of a separate report attached with the Main Geotechnical Report.

8.5 Laboratory Tests

Samples were collected from the pits at 1.0m and 1.5m depths for the following test. Sample testing were carried out in APECS Test House in Thimphu. The following laboratory tests shall be carried out from the random samples collected from the field. There were 8 samples collected and tested as per the relevant IS and International Standards. The test details have been attached as a part of the Main Geotechnical Report.

- Grain Size analysis
- Specific Gravity
- Natural Moisture Content
- Density Test (Dry and Bulk Density)
- Atterberg's Limit (Liquid and Plastic Limit)
- Proctor Compaction Test
- Direct Shear Test
- Consolidation test
- Permeability test

8.6 Mapping

Extensive walk over surveys were conducted to gather information which ultimately helped in the preparation of geological and engineering geological maps. Different types of forms were used to gather information on rock types, geotechnical units, soil types, colour, homogeneity, compressibility, spread, locality, drainages, marshes, seepages, springs, landslides, slope orientation, stability and so on. The data thus collected were used to prepare different types of maps, which are discussed below.

8.6.1 Geological and Instability Map

A geological map is a special-purpose map made to show geological features. Rock units or geologic strata are shown by colour or symbols to indicate where they are exposed at the surface or near to the surface usually beneath a thin soil layer. Bedding planes and structural features such as faults, folds, foliations, and lineation are shown with strike and dip or trend and plunge symbols which give these features' three-dimensional orientations. Mainly two rock types have been mapped in the study area. They are as follows:

Figure 89: Major rock types mapped in the study area

The rocks in the field work area belong to the Phuentsholing Formation. This Formation is represented by an alternating sequence of grey-greenish phyllite, phyllitic quartzite sequence with rare carbonates, intercalated with thick white-light grey fine to medium grained, locally gritty quartzite. The general trends of the rock are from ENE-WSW and NNW-SSE with low to moderate dips directed towards north to north-east.

8.6.1.1 Phyllite

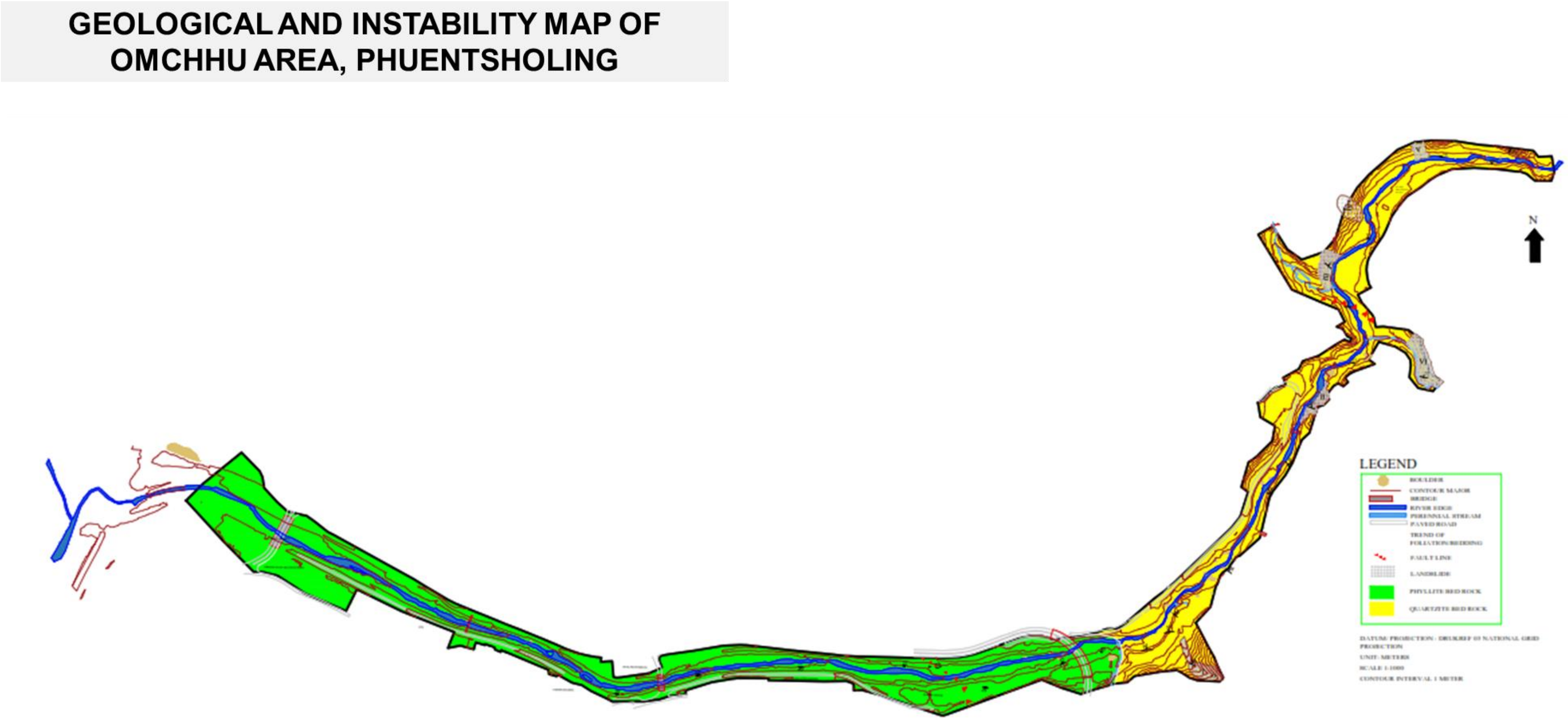
In the lower part of the study area, it is mostly covered by quaternary deposit of mostly debris flow deposit with the underlying bed rock as crushed grey phyllite which is talcoisic at some place. This rock is exposed on the both sides of the Omchhu river areas and also in the slopes to the north of the study area as an alternating but the dominant layer to quartzite. The strike of the band is NE-SW with low to medium dips towards North. The grey phyllite is overlain by carb-phyllite and also talcoisic phyllite which is highly fractured but these bands are not mappable as they occur as a thin lenticular band. Phyllite is grey mostly to greenish grey at places, visible pucker lineation and varying strike and dip directions. Few exposures show slaty cleavages in the phyllite. It is very fine grained, moderate to highly weathered and is fractured intensely due to the tectonic affects and it being a non-competent rock can easily break and shatters under pressure. Planar and wedge failures could be observed in the slopes having this rock types.

8.6.1.2 Quartzite

Overlying this greenish grey phyllite is a thick band of Quartzite but with phyllite partings. It strikes at E-W with dip moderate dips towards North. This quartzite is medium bedded, dark grey to grey to light grey coloured, slight to moderately weathered and is slightly wavy in structure. The phyllitic partings within the quartzite are talcoisic in some areas. Sheared quartz veins observed within this band is an indication that this area underwent an intense pressure due to the tectonic activities. Even though quartz is a competent rock, the fractured faces this band exhibits is a strong testimony to the prevailing pressure condition as a result of which few local faults could also be mapped in this area.

Quartzite is formed from the re-crystallization of quartz-rich sandstone. It has a sugary appearance and is made up of interlocking small grains of quartz. The grains have been welded together during the high temperatures of metamorphism so that quartzite is much less friable than the parent sandstones. Quartzite comprises equidimensional minerals i.e., quartz and feldspars and are non-foliated. Granular texture/structure (Granoblastic) makes them most competent rock amongst all other metamorphic rocks. Quartzites are compact, hard and strong; very less porous and less permeable than the parent sandstone. Quartz (SiO_2) with hardness value of 7 on Mohs scale is the main mineral in quartzite. Predominance of Quartz makes the rock very hard and suitable for road metal and it can be used as concrete aggregate. It also acts as strong foundation for any civil structure. Quartzite is magnificent decorative material that is why it was used in decoration of the building. This quartzite unit appears fine grained because quartz grains have undergone grain size reduction due to dynamic re-crystallization (mylonitic). Thus, the fine grain texture is not a sedimentary feature. Moreover, plastic deformation micro-fabrics in quartz grains suggest deformation at high temperature. The Geological and Instability map is shown as Figure 90 .

Figure 90: Geological and Instability Map of Omchhu Area



8.6.2 Engineering Geological Map

A material/geotechnical/engineering geological map was prepared taking into consideration all the geological units as shown in the map attached at the end of this report. An Engineering Geological Map is prepared by dividing the area under study into different geotechnical units, recording existing geology, and estimating likely conditions in terms helpful to the selection of construction techniques and ground treatment, and to the prediction of the reaction between ground and structure. Thus, for the design of engineering works, such as foundations, excavations (for roads, mines, etc.) the site must be subdivided in “homogeneous zones” of uniform geotechnical behaviour, thus in homogeneous zones with more or less uniform geotechnical parameters. The delineation of these homogeneous zones must be defined 3-dimensionally (as deep as the influence zone of the construction is reaching) with the help of detailed field mapping, with trenches, bore holes, geophysical resistivity and/or seismic refraction techniques.

It is not feasible to delineate boundaries of homogeneous zones with help of systematic geotechnical field testing. The zoning of the ground is usually done on the basis of geotechnical soil and rock classification and is based on the assumption that soils within one of the classes will have uniform geotechnical parameters, such as:

- cohesion
- angle of friction
- compaction characteristics
- modulus of elasticity, etc

In order to comprehend realistic geotechnical units for the Omchhu area, extensive walk over survey for detailed field mapping, pitting (SPT/PLYT Pits), seismic studies and laboratory testing of samples was carried out.

On the basis of the methods detailed above, four different geotechnical units could be identified. They are the colluvial deposits, alluvial deposits, Quartzite bed rock and Phyllite bed rock. These geotechnical units have a great significance on the type of land forms as well as on the stability of the slopes. In the Omchhu area alluvial deposits are mainly the most extensive and is present all along on the banks and in the flow channel. The area being gentle, this flow deposit has covered a large area at the middle and lower portions of the study area. The Geological and Instability map is attached as Figure 91 .

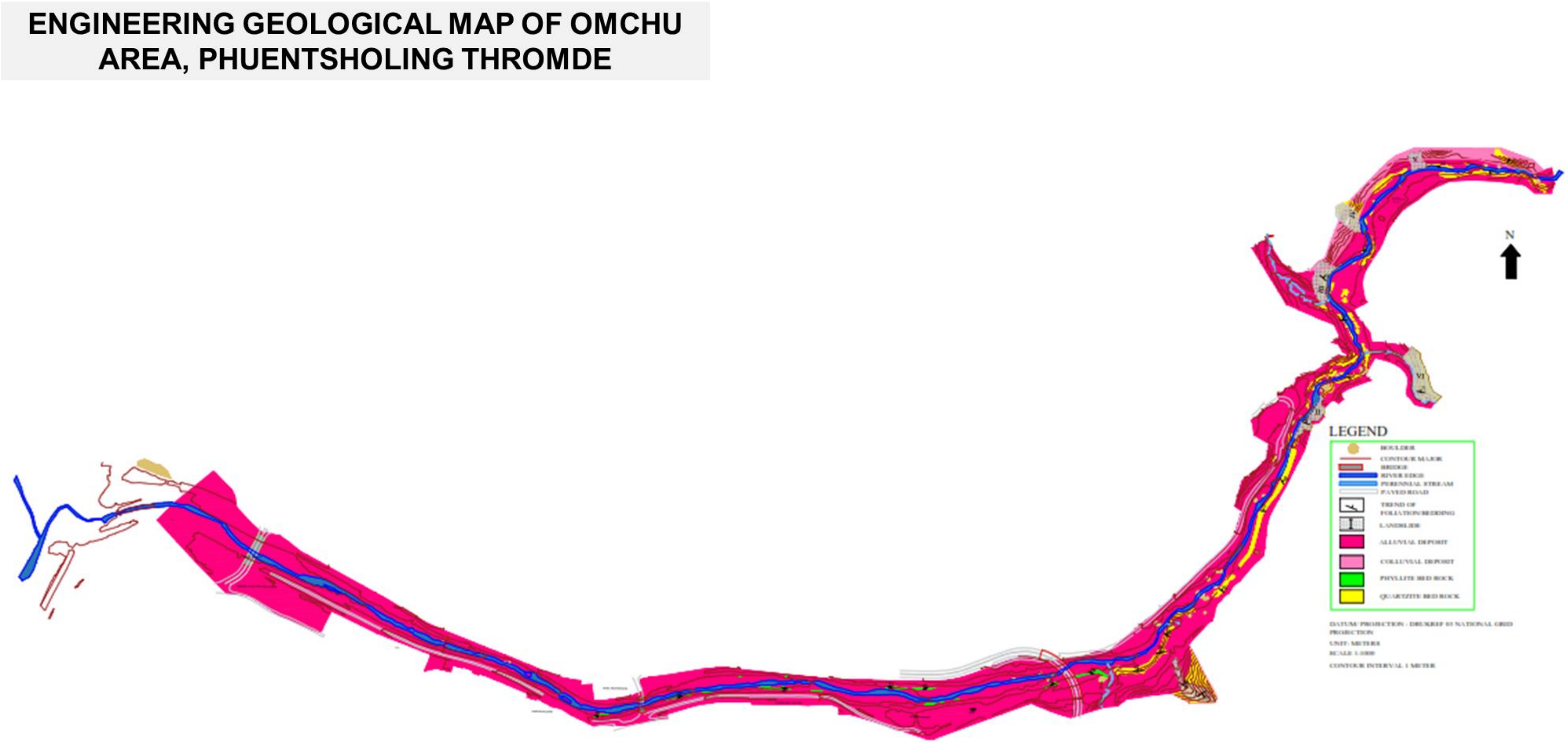
8.6.2.1 Alluvial Deposits

More than 80% of the mapped area is covered by the sediments transported and deposited by Omchhu, which are called alluvial deposits. In the Omchhu, alluvial soils are mainly restricted to the following types:

- riverbed deposits
- river terraces

In the riverbed, sediment is transported as rolled bed load and in suspension. These types of deposits can be seen in and around both the river bank terraces. River terraces are remnants of older higher floodplains, abandoned when the river cut to a lower level. They are subject to erosion as the present riverbed widens. There are at least three different terraces formed by Omchhu.

Figure 91: Engineering Geological Map of Omchhu Area



Irregularities in the river gradient due to differences in the resistance of bedrock against (often glacial) erosion may form lakes along the river course, which is not seen in Omchhu.

In general, alluvial soils are sorted and bedded. However, during the process of transport and sedimentation in a river, grain size selection takes place that influences to a very high degree the geotechnical characteristics of the resulting type of alluvial soil. In an environment of high energy (transport in a river with a very steep gradient) the size of the particles transported can be very large, up to cobbles and boulders. In low energy environments (lakes, floodplains of rivers with very low gradients, temporary lakes in landslide dammed rivers) even very small (silt and clay) particles may be deposited, which in the river normally are carried along in suspension. A variation of river discharge speed is reflected in inter-bedded finer and coarser layers.

It is evidently clear that in the upper parts of Omchhu, large sized boulders are observed along and on the river banks, brought down by high energy river (mostly during flooding) and as it reaches the lower levels, in the area after the curvilinear bridge, the deposits are finer with decreased boulder sizes and as it passes the lower most Omchhu bridge, the deposit is mostly sand.

Figure 92: Alluvial Deposit at Omchhu

8.6.2.2 Colluvial Deposits

The weathering products which are deposited after some downward movement lower on the slopes are the colluvial deposits. The main processes causing down slope movement of weathered and/or disintegrated rock are landsliding and erosion. In both processes' gravity is the main driving force, although water usually plays an important role in triggering the processes.

Figure 93: A colluvium soil exposure

In the case of landsliding, the water pressures at the basis of a coherent sliding mass (often called “sliding body”) cause the triggering of the sliding. Erosion is much more a surface process where individual particles are carried down slope by gravity but mobilized by water. Sorting of colluvium is usually poor; it is composed of widely different grain sizes, from large boulders with gravel, sand, and clayey materials between the coarser fragments.

In the study area, this type of deposit is the most extensive and is seen as long extensions on the slopes and at the slope bases especially on the right bank of Omchhu at the upper most part of the study area. The materials are poorly sorted and the rock type is the parent rock of Quartzite and Phyllite. The boulders are angular to sub angular in shape. In the colluvial overburden, the thickness of colluvium soil is judged to be in the range of 2 to 4 metres.

8.6.2.3 Phyllite Bed Rock

The boundary between soil and rock is often transitional in the case of weathered rocks. Weathering is a process that may take 10,000 years or more to reach deeper into the rock but it depends strongly on the combination of the type of rock and the climatological circumstances and it depends as well on the position in the landscape.

In the few levels of the study area, especially in and around the Curvilinear bridge are, is green-grey coloured phyllite with very thin bands of grey quartzite. It is fine grained, moderate to highly weathered with tight to open joints. At least three sets of regular joints and some irregular joints are observed. These joints are closely spaced and filled with talcoisic material, which further degrades its strength parameter. As it already has low tensile strength due to several joint sets, steep slopes (high gradient) compounded by very heavy rainfall these areas receive, the stability of these rock slopes is heavily compromised. It is because of these characteristics; we can observe may slides within this phyllitic zone. In some areas, water is also seeping from these slopes further destabilising these slopes.

8.6.2.4 Quartzite bed rock

The other rock type is hard and compact, medium to small blocky and thin to medium bedded at the lower horizons, fine to medium grained quartzite. It is white to light grey to grey coloured and forms a competent rock stratum. This rock extends all over the upper part of the study area, occurring as a major insitu rock and also as thin bedded intrusion within Phyllite in the boundary areas. It forms steep escarpment and sharp ridges. Due to its steep topography and different joint set combinations and further decimated by few local faults, rock slides in the form of topple, wedge or planar failures are also observed. Most of the instability in this formation are major, with prominent scars. Megascopic observation reveals the presence of quartz, mica and feldspar (mainly K) as dominant minerals.

8.6.3 Structural Mapping

Rock units or geologic strata exposed at the surface exhibit discontinuity structures. Discontinuities (such as bedding planes, joints, cleavages, etc.) are usually occurring in sets of parallel individual discontinuity planes. In sedimentary rocks discontinuities are often developed in two sets perpendicular to the bedding planes and are perpendicular to each other. The orientation of these sets is the result of the history of consolidation and tectonic stress fields during the geological history of the rocks.

The orientation of discontinuities is of great importance in most geotechnical analyses. The presence and orientation of one single discontinuity plane may endanger the stability of a natural slope or a man-made excavation. The presence of discontinuities in different orientations and their discontinuity spacing and persistence causes the rock mass to be disintegrated into individual rock blocks. Different joint sets may have different values of characteristic spacing. This will lead to a rock mass composed of blocks of a certain size and form.

Apart from the orientation and spacing of discontinuities there are the characteristics of the discontinuities which determine if the discontinuities will facilitate or resist relative movement of blocks along them. Planes with well-developed roughness will resist movement stronger than smooth planes, planes coated with moist plastic clay will slide more easily than planes without infilling.

Bedding planes and structural features such as faults, folds, foliations and lineation are shown with strike and dip or trend and plunge symbols which give these features' three-dimensional orientations. The major discontinuities present in the study area are the beddings, foliation, joints and faults. The general regional strike of the rock is from East-West with northerly dips ranging from 15° to as high as 71°.

8.6.3.1 Faults

Three sets of local faults could be mapped in the study area. The mapped faults have the following dip directions and dip amount.

i. 024/21
ii. 021/57
iii. 229/24

In few cases, slight displacement of about 10 cm to 15 cm was observed in these faults, so the displacement could not be shown in the geological map. Fault gouge can be observed in few fault lines and especially near to the confluence of Namantri and Omchhu. This faulting is also considered one of the contributing factors for the failure of the slope.

Figure 94: Photographs of some of the faults observed in the field

8.6.3.2 Other discontinuities

The other prominent discontinuities are the beddings, joints and foliations. The table below shows other major discontinuities mapped in the study area. Apart from these major ones, there are many random joint sets.

Table 37: Table showing discontinuity sets

EXPOSURE	FOLIATION/BED	JOINT 1	JOINT 2	JOINT 3
R01	223/52	342/50	87/24	256/85
RO2	192/29	353/37	133/75	
RO3	351/10	336/79	203/47	
RO7	347/39	219/47	131/53	
RO11	47/60	330/66	139/69	
RO12	20/47	231/29	124/45	
RO15	63/59	357/32	243/39	
RO20	Nov-71	263/50	89/61	

These discontinuities play a major role in the stability of rock slopes. With the increase in the number of joints, the probability of creating instability in the rock slope increases.

8.7 Stability Study

Slope instability can be manifested as landslides—including mudflows ("mudslides") and rock falls — or by more subtle processes such as soil creep. Slope instability is a complex phenomenon that can occur at many scales and for many reasons. Landslides can be fast or slow, wet or dry, small or large, shallow or deep, old or recent. Examples of triggering mechanisms include earthquakes, grading, poor surface drainage, erosion, rainstorms, landscape irrigation, or broken utility lines. Fast-moving mudflows and debris flows are of particular concern in wildfire burn areas and in areas that have previously experienced landslide activity.

Buildings and infrastructure located on or in the path of a landslide can be seriously damaged or destroyed. Slope movements do not need to be large to be destructive — even slope creep or small; early-stage landslide movements can cause substantial structural damage to critical facilities such as bridges and roads, resulting in major economic damage and loss of life. Conversely, earth movements initially suspected to be caused by landslides might instead be the result of other processes such as fill settlement, shrink-swell action of expansive soil, or hydro-compaction of collapsible soil. The instabilities could either be due to the natural process or manmade.

The most common landslide triggers are rainfall events and seismic events (earthquakes). Because these triggers act on a slope in different ways, it is important to distinguish between those landslides that are rainfall triggered versus those that are seismically triggered so that appropriate risk mitigation measures can be identified.

8.7.1 Factors Determining Slopes Stability

There are several factors determining stability of a slope. These are broadly categorised as Preparatory, Anthropogenic (aggravating) and Triggering factors (Anderson, Halcombe, 2013).

Preparatory factors determine the stability of a slope over a period of time. They are as follows:

- Slope geometry
- Soils and geology
- Slope hydrology

Anthropogenic (aggravating) factors reduce the stability of a slope without necessarily triggering a landslide and depend on:

- Surface water
- Groundwater level
- Slope angle (cut)
- Load (building)
- Vegetation

Triggering mechanisms are the dynamic events that result in a landslide and is due to the following:

- Rainfall
- Seismic activity

Rainfall-triggered landslides occur in most mountainous landscapes and can have an enormous effect on the landscape, properties, and people. Intense or prolonged rainfall infiltrates the slope surface, causing an increase in soil pore water pressure and an associated lowering of slope material strength. The forces that act to stabilize the slope are thus reduced, and the slope fails along the zone where the destabilizing forces (gravity and loading) overcome the stabilizing forces.

These various factors will act and interact across a particular slope to determine its stability state at any point in time. Each factor must be considered and their combined influence assessed in order to understand the stability of a slope.

8.7.2 Factors Affecting Stability in the Study Area

Extensive surveys were carried out to map the instabilities in the study area. It is to be understood that Omchhu, is in the southern foot hills where the annual rainfall is very high compounded by fragile geology as the rock type is mostly phyllite in the lower slopes, which does not exhibit strong compressive as well as tensile strengths and is further shattered by faults. The hill slopes have gradients mostly above 30° which will create stability problems for the area. There are several slides in the study area, both active and dormant. The factors responsible are as described below.

8.7.2.1 Slope Geology, hydrology, gradient and Vegetation

To start with, the preparatory factors in the study are very favorable for landslides. It is evident from the engineering geological map, the major units present in the slopes are alluvium soils, highly fractured phyllitic rock and the competent but high slope forming quartzite. The landslide prone areas have high gradient slopes, which are above 35° in the colluviums, 50°-70° in the Phyllitic zones as well as in the Quartzite rocks. Colluviums in the slopes are unsorted and mostly uncompacted, which allows the water to seep into it. In few cases, a thin layer of soil is present on top of the bed rock, and this layer slips down the slope due to the triggering mechanisms, which is the rainfall in most of the cases. As is evident from the field observations, water seepages from the cracks and joints of the rock are some phenomena which saturates the rock soil layer, thereby increasing the pore water pressure.

8.7.2.2 Anthropogenic factors

The anthropogenic factors contributing to the slope stability in the study area are slope water, vegetation and human intervention. There are no proper drains and thus allowing the water to flow freely, whether be it rain, water seepages or spring water along these fragile slopes. Due to frequent sliding and high gradient, the vegetation cover is thin, further compounding the problem. There are settlements nearby these slopes and the people rear cattle. Herds of cattle grazing on these slopes seems to be a daily phenomenon, further degrading the slopes. Another phenomenon observed is the drastic increase of human activity in these areas.

Landslides activated or re-activated by negative intervention of man's activity are of considerable dimensions, bring vast damage to the environment and evoke considerable attention from the people. The fact that inconsiderate human activity and the effects of construction work on old landslides are the most frequent cause of reactivation of movements in areas of intense economic use. Worldwide studies show that above 70% of the landslides, which have taken place, are usually brought about by artificial interventions.

The construction intervention in the slope environment causes in it several simultaneously and consequently acting forces and processes, which may end in various changes on the slopes. These may also reflect back on the state and behaviour of technical works. The changes may be not only of a varied type, but also of varied intensity and duration (temporary and permanent, slow and quick, reversible and irreversible, etc.) The changes may be mediated directly or indirectly. Human interventions on the slopes bring about all negative influences in the form of artificial loading of slopes, slope undercutting, negative anthropogenesis influence of regime changes of groundwaters and the influences of artificial vibrations.

Undercutting of slopes prone to sliding, changes the forces ratio acting on the slope to the advantage of active forces. Negative influences, after slope undercutting, are also manifested by worsening the rock properties in the cut and by the acceleration of erosion processes. The deterioration of the properties of the rock forming the slope appears by loosening the horizontal stresses on the slope, which brings about loosening of the slope and swelling of rocks little resistant against weathering.

8.7.3 Triggering factor

In the study area, the main trigger for landslides is the monsoonal rainfall, though instability can also occur due to seismic activity of higher magnitude, but such occurrences are fewer. Monsoon starts from June and lasts till September-October. The study area being located in the foothills the average annual rainfall ranges from 5000mm- 6500 mm. The Maximum rainfall of 495.3 mm in a day was recorded on 2nd August 2000 in the Phuentsholing weather station. The maximum rainfall was experienced in 1998, where the total rainfall recorded was 6699 mm and the minimum recorded was in 2006 where the total rainfall recorded was 1976mm only. This clearly shows the change in weather pattern and the rainfall has become erratic and the rainfall plays a very important role in the stability of the area.

8.7.4 Relation between Rainfall Threshold and Landslides

Worldwide, there has been several studies carried out to find out the rainfall intensity and landslides, but unluckily, no such studies have been carried out in Bhutan. In case of rainfall-induced landslides, however, the minimum intensity or duration of rainfall necessary to cause or reactivate a landslide is known as the rainfall threshold for landsliding (Varnes, 1978). Also, a threshold may define the rainfall, soil moisture or hydrological conditions that when reached or exceeded are likely to trigger landslides (e.g., Crozier 1996; Reichenbach et al. 1998; Guzzetti et al. 2007). Wieczorek (1996) defined rainfall threshold as rainfall intensity that facilitates slope instability for a given region.

Based on the extent of the geographical area, rainfall thresholds for rain-induced landslides with different types of precipitation measurements can be broadly subdivided into global, regional, or local thresholds based on their geographical extent. A global threshold attempts to establish a general ("worldwide") minimum level below which landslides do not occur, independently of local morphological, lithological and land-use conditions and of local or regional rainfall pattern and history.

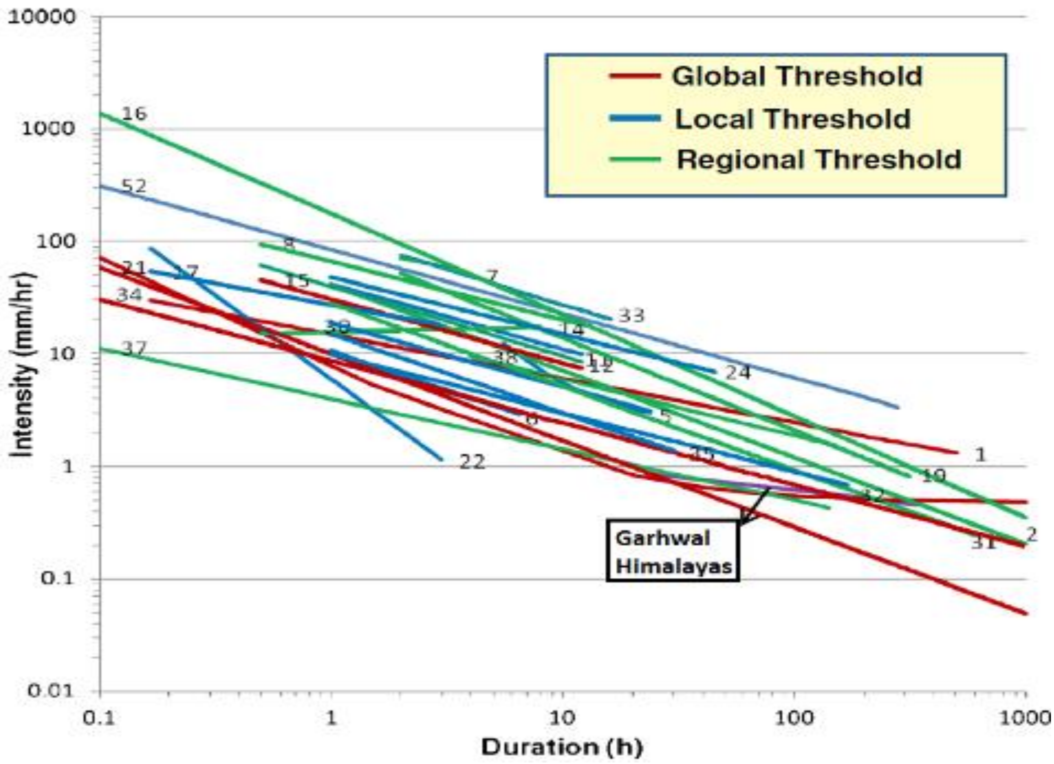
Regional thresholds are defined for areas extending from a few to several thousand square kilometers of similar meteorological, climatic and physiographic characteristics (may be for different countries or for states within the country) and are potentially suited for landslide warning systems based on quantitative spatial rainfall forecasts, estimates, or measurements.

Local thresholds consider the local climatic regime and geomorphological setting and are applicable to single landslide or to groups of landslides in areas extending from a few to some hundreds of square kilometers (may be for local geographical area or for a highway corridor, etc.). In places, distinction between regional and local thresholds is uncertain. The figure below shows some threshold carried out by different authors (compiled by Kanungo & Sharma, August 2014).

In South Korea (Moung, 2016), the quantitative analysis shows 1-day rainfall and 3-day cumulative rainfall had higher correlations with landslides. Based on this, the landslide occurrence threshold in South Korea is defined to be 202 mm for 1-day rainfalls and 449 mm for 3-day cumulative rainfalls.

In a similar study in Nepal Himalayas, Ranjan Kumar Dahal and Suichi Hasegawa (2008) published an article "Representative Rainfall Thresholds in Nepal Himalayas" published in ScienceDirect, Geomorphology 100 (2008) 429 – 443, they concluded that a rainfall of 10 h or less requires a rainfall intensity in excess of 12 mm h− 1 to trigger failure, but a rainfall duration of 100 h or longer with an average intensity of 2 mm h− 1 can also trigger landslides in the Himalaya. Moreover, in the daily rainfall scenario, this study concluded that when daily rainfall amount exceeds 144 mm, there is always risk of landslides in Himalayan slopes. The landslide threshold relation also indicates that as much as three times more rainfall is required to trigger landslides in short duration in the Himalaya than the rainfall amount required to trigger landslides world-wide. Likewise, the difference of rainfall threshold in tropical monsoon and other climates is less significant when rainfall intensity of 100 h is considered.

Figure 95: Comparison of rainfall intensity–duration (ID) thresholds for the initiation of landslides available in literature:



Source 15: Refer Table 2, Guzzetti et al. 2007)

Near home, in research carried out in Paglajhora sinking zone in Darjeeling by Sujit Mondal (Mondal, 2016), his finding was that the Antecedent Cumulative rainfall induced landslide analysis showed that the continuous and uniform rate of minimum amount of rainfall of approx. 89 mm/day for few consecutive days can cross the geomorphic threshold and can introduce slope instability conditions.

8.7.5 Soil Slope Stability Assessment

Extensive surveys were carried out to map the instabilities in the landslide. It is to be understood that the study area, is situated in the southern foot hills where the annual rainfall is very high compounded by fragile geology as the rock type is phyllite in the lower foothills, which do not exhibit strong compressive as well as tensile strengths. The hill slopes have gradients mostly medium to high which will create stability problems in the area. There are several slides in the study area, both active and dormant.

There are few small inactive, old landslide scars all along the two sides of river till the top of the study area. Few of these slides are of smaller dimensions except the slides marked from I to V which are considered major slides. A possible landsliding in minor scale will continue in the area due to the steep topography, fragile geology, unconsolidated alluvium material as well as fractured rock mass on the slopes. To make the matter worse, there are few local fault sets, which is aggravating the slope instability. Rock fall along with debris/rock slide is a possibility as the rock in the active slide areas is a competent grey quartzite, it is intercalated by weak phyllitic rock. Vegetation cover is very thin.

8.7.6 Rock Slope Stability Analysis

The most serious disorders which can occur in slopes are (1) the sliding of block(s) along the discontinuing surfaces which exist at the opening of trenches and (2) rock falls or sliding of block(s) resulting from the rapid change in the mechanical characteristics of the rock bench which supports them. The forecast of simple slides is based on the detailed analysis of the geometry of discontinuing surfaces. This analysis can often be very difficult because of insufficiently precise data on discontinuity distribution and evolution in shear strength.

The prediction of mass rock falls caused by the rapid change in rock benches liable to weathering is based on a detailed geological and mineralogical study and laboratory test results. These weathering tests can give an assessment of the weathering properties of materials taking account of the special physico-chemical conditions of a given structure. In the study area, Slope Stability Probability Classification (SSPC) form was used to collect the field data as well as to analyses the stability of rock slopes. Stereonet projections were also made to investigate the stability of the slopes. Weathering classification has also been carried out.

There were several rock exposures mapped during the detailed field study. In this report 8 rock exposures characterization details using SSPC though rock trend recording is analyzed from more than 30 exposures to create a geological map. The discontinuities parameters have been analyzed using stability probability classification system and it is found that the probability of the slopes to remain stable varies from outcrop to outcrop. The probability of the slope to remain stable is less than 5% for exposures RO-3, RO-12 and RO-20. The probability to be stable is found to be above 95% for the exposures RO-7, RO-11 and RO-15. The probability to be stable is found to be above 70% for the exposure RO-1. The probability to be stable is found to be above 30% for the exposure RO-2. The result is appended in Appendix F of the Main Geotechnical Report. Due to the highly jointed and open nature of the joints compounded by local faults, there is danger of rock falls/slides and/or even wedge, planar and toppling failures.

In the characterized rock slopes the detailed rock mass weathering classification gives 1,-39,-21,-22 and -31 for the exposure RO-1, RO-2, RO-15, RO-20 and RO-23 respectively which corresponds to class C. This class has rocks which are significantly weathered. For the rock exposures RO-7, RO-11 and RO-12, the overall end rating of 51, 29 and 43 were obtained respectively which corresponds to class B. This class has rocks which are slightly weathered. For the rock exposures RO-3 and RO-24, the overall end rating of-110 and-97 were obtained respectively which corresponds to class S1. This class has rocks which are geotechnically soil with relict discontinuities.

The rock mass classification was carried out in the field and is attached as Appendix B of the Main Report. The engineering significance of this class is described below, adapted by Marco Huisman from Price (1993).

- B: Rock material notably affected by weathering and some loss of strength, but not yet adversely influencing engineering work. With further degradation of the rock mass, influence on engineering works will be significant.
- C: Reduction in strength of foliation planes gives problems in slope stability, tunnels, and foundations on slopes.
- S1: Weathered material geotechnically a soil, so all engineering works designed on soil parameters.

8.7.7 Stereographic and Kinematic Analysis

Kinematic analysis, which is purely geometric, examines which modes of slope failure are possible in a jointed rock mass (Wright et al. Z, 1984). Angular relationships between discontinuities and slope surfaces are applied to determine the potential modes of failures (Yoon et al., 2002). Stereographic representation (stereonets) of the planes and lines is used. Stereonets are useful for analysing discontinuous rock blocks. Program DIPS (Visualization software) allows for visualization of structural data using stereonets, determination of the kinematic feasibility of rock mass and statistical analysis of the discontinuity properties (Zulfu et al., 2008).

In this case Ski-wedge, an Excel-program, designed by Michael Maurenbrecher (Professor, TU Delft, the Netherlands), has been used. For the exposure RO-1, the first and second joint set gives a factor of safety of 0.53 which is less than the limiting equilibrium. Therefore, the probability of the failure of rock slopes due to the combination of these two joint sets is high. The second and the third joint set the first and the third joint set give the factor of safety above 1, which is thus considered stable.

For the exposure RO-2, the first and second joint set gives a factor of safety of 2.64 which is more than the limiting equilibrium. Therefore, the probability of the failure of rock slopes due to the combination of these two joint sets is not expected. The second and the third joint set give a factor of safety of 0.88 which is also below the limiting equilibrium and thus the probability of rock slope failure for this combination is probable. The first and the third joint set give the factor of safety of 0.42, which is lower than the limiting equilibrium and thus considered unstable.

For the exposure RO-3, the first and second joint set gives a factor of safety of 4.39 which is much higher than the limiting equilibrium. Therefore, the probability of the failure of rock slopes due to the combination of these two joint sets is not expected. The second and the third joint set give a factor of safety of 0.60 which is also below the limiting equilibrium and thus the probability of rock slope failure for this combination is probable. The first and the third joint set give the factor of safety of 3.13, which is higher than the limiting equilibrium and thus considered stable.

For the exposure RO-7, the first and second joint set gives a factor of safety of 2.31 which is more than the limiting equilibrium. Therefore, the probability of the failure of rock slopes due to the combination of these two joint sets is not expected. The second and third joint set give a factor of safety of 1.04 which is just above the limiting equilibrium. Thus, the probability of rock slope failure for this combination is not expected. The first and the third joint set give the factor of safety of 3.23, which is higher than the limiting equilibrium and thus considered stable.

For the exposure RO-11, the first and second joint set gives a factor of safety of 2.55 which is more than the limiting equilibrium. Therefore, the probability of the failure of rock slopes due to the combination of these two joint sets is not expected. The second and third joint set give a factor of safety of 7.56 which is much higher than the limiting equilibrium. Thus, the probability of rock slope failure for this combination is not expected. The first and the third joint set give the factor of safety of 0.78, which is lower than the limiting equilibrium and thus considered unstable.

For the exposure RO-12, all joint set combination gives safety factors of higher than 1, and thus considered stable. For the exposure RO-15, the first and second joint set gives a factor of safety of 1.28 which is more than the limiting equilibrium. Therefore, the probability of the failure of rock slopes due to the combination of these two joint sets is not expected. The second and third joint set give a factor of safety of 2.41 which is also higher than the limiting equilibrium. Thus, the probability of rock slope failure for this combination is not expected. The first and the third joint set give the factor of safety of 5.96, which is higher than the limiting equilibrium and thus considered stable.

For the exposure RO-20, the first and second joint set gives a factor of safety of 13.97 which is more than the limiting equilibrium. Therefore, the probability of the failure of rock slopes due to the combination of these two joint sets is not expected. The second and third joint set give a factor of safety of 11.86 which is also higher than the limiting equilibrium. Thus, the probability of rock slope failure for this combination is not expected. The first and the third joint set give the factor of safety of 0.36, which is lower than the limiting equilibrium and thus considered unstable.

The rock Slope Stability Probability Classification (SSPC) shows many rock slopes to be unstable in the exposure characterizations giving stability probability to be less than 5% in most cases. The Skiwedge Stereonet also shows significant slope failures due to wedge and/or planar failures. The weathering classification gives values for middle to lower rock classes, which suggests that weathering grade is high.

It is usually observed that Quartzite exposures especially along gentler slopes have slight to moderately weathered rocks whereas those rock exposures of phyllite are highly weathered. These are surface observations and the rock condition is expected to improve with depth as the weathering decreases with the increase in depths and hence will result in better stability probability as well.

8.8 Hazard Assessment

Fatalities and economic losses due to natural catastrophic events have increased in the last decades. This is not only due to the growth of population density in hazard risk zones, but also to the consequent and concomitant increase of possible “cascade effects”.

Assessment and mitigation of the impact of catastrophic events in a given area require innovative approaches allowing a comparison of different risks and accounting for all the possible cascade events. Ranking of the typologies of risks affecting a given area can hardly be made because presently available scenarios are often qualitative; they are related to one reference event and rarely account for the related uncertainties. Moreover, different types of risks (fast mass movements, floods, earthquakes) are often estimated using different procedures so that the produced results are not comparable. Compared to classical analysis of single risks, these methods may provide a formal scheme to compare and rank different kinds of hazardous phenomena (natural, man-made, etc...), and account for “cascade effects” that are usually neglected in single risks analysis.

The hazard study was mainly based on the observation of slope, the landscape, old landslide areas, minutely analyzing any indications of ground movements in and around the study areas, water seepages, marshy land and flooding. Hazard due to the above-mentioned factors are analyzed and suitable mitigation measures proposed accordingly.

Slopes largely remain in an equilibrium situation where resisting forces, forces resisting movement, exceed those forces creating movement. Instead of slope instability slope failure is often used to indicate that the slope is no longer stable and that movement occurs.

The result of instability is the landslides: this is a general, non-precise description for a whole group of mass movement phenomena. A more formal definition of a landslide is: the movement of a mass of rock, debris, or earth down a slope. (Cruden, 1991). Terzaghi (1950) made a distinction between external and internal factors causing mass movement.

Internal causes are the mechanisms which bring about a reduction of its shear strength to a point below the external forces imposed on the mass by its environment, thus inducing failure (Bell, 1983). (Weathering, pore pressure). External causes are those mechanisms outside the slope mass involved, which were responsible for overcoming its internal shear strength, thereby causing it to fail. (Overloading, removal of support of the toe, earthquakes, shocks)

Both internal as external causes may affect the equilibrium conditions of the slope by:

- increase of shearing stress
- decrease of cohesion
- decrease of frictional resistance
- liquefaction

There are only a few landslides which are active and also some few inactive slides seen in and around the study area. There are five significant active slides in the study area and are also shown in the instability map. These instabilities are been discussed herein below.

8.8.1 Landslides

8.8.1.1 Landslide No. I

This landslide is the largest one in the study area given by northern latitude of 26.863576° and the eastern longitude of 89.396124° within the altitude of 263m to 379 metres above mean sea level. It is about 100 metres away on the hill-slope from the eastern flank (left side) of Omchhu and has a dimension of 14000 m2 and 500 m perimeter. Its longer length (top-down) is 225 metres and shorter length (side to side) is 100 metres.

Figure 96: Photograph of Landslide – I

This landslide is concave in shape and the bed rock is exposed in the slide area. There are small slides within the main slide which is in the process of getting stabilized. These minor slides are due to fragile geology, steep topography, loose debris on the slopes and of course the high intensity rainfall which triggers these slides. The rock type observed in the slide area is grey, fine-grained quartzite intercalated by grey to dark grey very fine grained phyllite. Though quartzite is a competent rock with high tensile and intact rock strengths, these phyllitic intercalations renders the rock mass weak and slides occurs within this discontinuity. The problem is further compounded by talcoisic nature of phyllite, which enhances the instability process.

8.8.1.2 Landslide No. II

This landslide is the second largest observed in the study area given by northern latitude of 26.869031° and the eastern longitude of 89.398070° within the altitude of 251m to 282 metres above mean sea level. It is on the eastern flank (left side) of Omchhu and has a dimension of 4500 m2 and 350 m perimeter. Its shorter length (top-down) is only 46 metres but the longer length (side to side) is about 135 metres.

Figure 97: Photograph of Landslide – II

This landslide is semi-circular in shape and the bed rock is exposed in the slide area. There are small slides within the main slide which is in the process of getting stabilized. These minor slides are due to fragile geology, steep topography, loose slided mass on the slopes and of course the high intensity rainfall which triggers these slides. This slide is more of a rock slide than a debris slide. The rock type observed in the slide area is light grey, medium to fine-grained quartzite intercalated by grey to dark grey very fine grained phyllite. Though quartzite is a competent rock with high tensile and intact rock strengths, these phyllitic intercalations renders the rock mass weak and slides occurs within this discontinuity. The problem is further compounded by talcoisic nature of phyllite, which accelerates the instability process.

8.8.1.3 Landslide No. III

This slide is of smaller dimension observed in the study area given by northern latitude of 26.871351° and the eastern longitude of 89.398062° within the altitude of 272m to 300 metres above mean sea level. It is on the western flank (right side) of Omchhu just below Bogataybari and next to the confluence.

It covers an area of approximately 2000 m2 and has a perimeter of 200 metres. Its shorter length (top-down) is only 50 metres but the longer length (side to side) is about 57 metres.

Figure 98: Photograph of Landslide – III

This landslide is rectangular in shape and the bed rock is exposed at the base of the slide area for about 3.5 metres. This is an active slide. This slide is caused due to fragile geology, steep topography, free flowing of storm water due to lack of drainage and of course the high intensity rainfall which triggers these slides. The rock type observed in the lower part of the slide area is light grey, medium to fine-grained quartzite and about 2 metres thick phyllite band which is an intercalation within the quartzite bed rock. This phyllite is grey to light grey very fine grained phyllite. Though quartzite is a competent rock with high tensile and intact rock strengths, these phyllitic intercalations renders the rock mass weak and slides occurs within this discontinuity. The problem is further compounded by talcoisic nature of phyllite, which accelerates the instability process. The top part of the slide is an alluvial deposit consisting of sandy gravel. The gravels are rounded to sub rounded with medium compaction.

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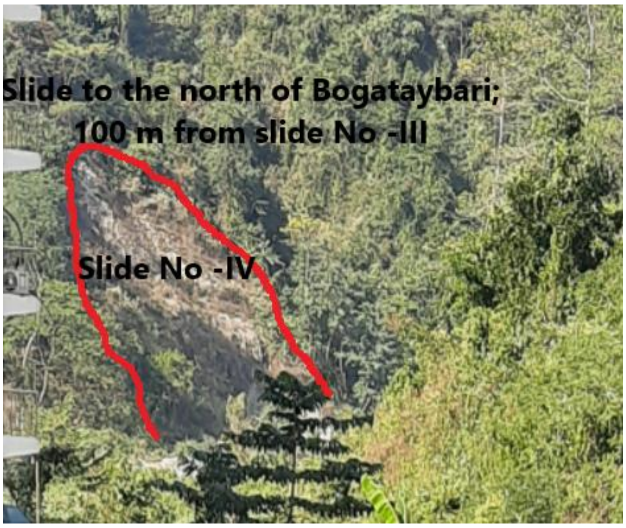
8.8.1.4 Landslide No. IV

This slide is also of smaller dimension observed in the study area given by northern latitude of 26.872559° and the eastern longitude of 89.398627° within the altitude of 270m to 322 metres above mean sea level. It is on the western flank (right side) of Omchhu to the north of Bogataybari and is about 100 meters away from Landslide No. III. It covers an area of approximately 1450 m^2 and has a perimeter of 150 metres. Its longer length (top-down) is about 70 metres and the shorter length (side to side) is about 38 metres.

Figure 99: Photograph of Landslide – IV



This landslide is elongated to semi-circular in shape and the bed rock is exposed in the slide area. This is a shallow slide and is also caused by rain water from the slopes. The rock type observed in the slide area is light grey, medium to fine-grained quartzite intercalated by grey to dark grey very fine grained phyllite.

8.8.1.5 Landslide No. V

This debris mass is more of an accumulation of slide material brought down by the small stream the joins the Omchhu river. It is given by northern latitude of 26.873231° and the eastern longitude of 89.40191° within the altitude of 271m to 336 metres above mean sea level. It is on the western flank (right side) of Omchhu, opposite of the new Thromde water supply establishment. It covers an area of approximately 657 m2 and has a perimeter of 120 metres. Its longer length (top-down) is about 45 metres and the shorter length (side to side) is about 17 metres.

Figure 100: Photograph of Landslide – V

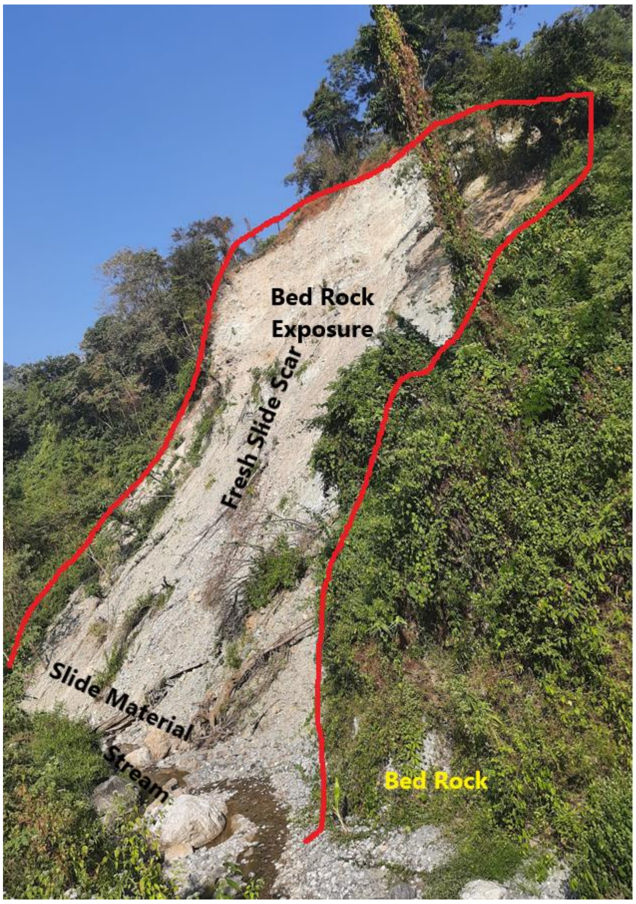
This debris deposit is arch shaped and consist of assorted mass of slide material. Apart from increasing the suspended material and a little possibility of temporarily damming Omchhu, it does not pose much risk.

8.8.1.6 Landslide No. VI

This landslide is the third largest observed in the study area given by northern latitude of 26.869755° and the eastern longitude of 89.400097° within the altitude of 271m to 336 metres above mean sea level. It is about 100 metres away from the eastern flank (left side) of Omchhu in a small tributary of Omchhu. It covers an area of 2450 m2 and has a perimeter of 200 metres. Its longer dimension (top-down) is about 87 metres and the shorter dimension (side to side) is about 47 metres only.

This landslide is elongated semi-arch shaped and the bed rock is exposed in the slide area. This is an active slide, though bed rock is exposed at few places. This slide is caused by fragile geology, steep topography, loose slided mass on the slopes and of course the high intensity rainfall which triggers these slides. The rock type observed in the slide area is light grey, medium to fine-grained quartzite intercalated by grey to dark grey very fine grained phyllite. Though quartzite is a competent rock with high tensile and intact rock strengths, these phyllitic intercalations renders the rock mass weak and slides occurs within this discontinuity. The problem is further compounded by talcoisic nature of phyllite, which accelerates the instability process.

Figure 101: Photograph of Landslide – VI



This is long slide and is located just to the right of a small stream that flows on its toes. There is a housing colony just to the left of this stream and if the slide material blocks this stream, the stream will flow into the housing colony posing risk to life and property. Therefore, this slide needs to be stabilised by constructing RCC base wall of about 3.5 metres and then stepped RRM wall till the top to stabilise the slope. Proper garland drain is also required to be constructed to divert the rain water away from the slide area.

8.8.2 Mitigation Measures

Based on the present study, mitigation measures are proposed to be undertaken to prevent or mitigate present and future landslide hazards. Stability of a slope can be greatly improved by practising the following suggested measures.

8.8.2.1 Changing the slope geometry

In the design stage of an artificial slope, we can adapt the geometry to obtain an acceptable factor of safety. If an existing slope proves to be unstable, changing its geometry is sometimes also possible. But this alone will not stabilise the slope completely. This has to be done in combination with a retaining structure. If any deep excavation is carried out, the created back slope needs to be maintained at proper repose angle to attain the stability of the slope.

8.8.2.2 Minimizing influence of ground and surface water

Water plays a very important role in the stability of slopes. The influence of groundwater on the factor of safety is significantly reduced by the presence of groundwater. To minimize groundwater influence, and to decrease infiltration into a slope after rainfall, drains can be installed in a variety of ways to stabilize slopes. However, in many cases we will have to incorporate different drainage measures in the design of a slope. This will eventually lead to achieving minimum infiltration in the fragile slopes.

Table 38: Recommended actions to minimize water influence

Sl. No	Slide No.	Recommended activity
1	I	Garland drains from the top of the slide (maintain at least 10 metres distance from the crown of the slide)
2	II	Proper road side drain has to be maintained. The water should not flow freely through the top of the slide.
3	III	Divert the domestic and rain water away from the slide, possibly towards Namantri by constructing a lined and stepped drains.
4	IV	Intervention not envisaged at this stage.
5	V	Intervention not required.
6	VI	Garland drains from the top of the slide (maintain at least 10 metres distance from the crown of the slide)

8.8.2.3 Strengthening or supporting

When a slope with a given geometry has to be stabilized, we can do this by increasing the shear resistance of the soils involved (for soil slopes) or by fixing in place individual rock blocks (for rock slopes). The most obvious options are:

- Rock or soil anchors
- Soil nailing
- Installing geofabrics (usually combined with drainage)
- Grouting
- Retaining walls
- Netting (fixed to slope face)

Among these various stabilisation options, construction of retaining walls and the drain is proposed. Drainage is installed, and the shear resistance is increased by a concrete retaining wall constructed at the toe of the slide. The proposed general remedial measures for instable slopes in the study area is as shown in Table 39 below.

Table 39: Recommended slope strengthening measures

Sl. No	Slide No.	Recommended activity
1	I	A series of Gabion check dams of 2 metres height till the confluence of minor drainages within the slide. Bio-engineering works on the active and higher reaches of the slide.
2	II	Regular monitoring of the slide No immediate intervention recommended.
3	III	2.5 metres high RCC wall embedded 0.50 metres into the bed rock (to minimise base erosion of RCC wall). A series of stepped RMM walls till the top.

		Regular Weep holes to be provided in the walls.
4	IV	Regular monitoring of the slide No immediate intervention recommended
5	V	No immediate intervention recommended
6	VI	A series of stepped RCC/RRM walls till the top. The base should be RCC wall and should be at least 1 metre below the scour depth of the stream. Regular Weep holes to be provided in the walls.

Figure 102: Location of Active Landslides along Omchhu

8.9 Recommendations and Conclusion

After carrying out this extensive study for Omchhu, the following conclusion along with recommendations is made.

The soil composition in Omchhu area is gravelly sand to sandy gravels with variations in cobbles to gravels fractions. It was generally observed that the soil is not densely compacted and also contains high percent of gravels, which are rounded to sub rounded in shape, which suggests that the deposit type is mostly alluvial.

Mainly two rock units are mapped in the field. They are Phyllite and Quartzite. The rocks in the field work area belong to the Phuentsholing Formation. This Formation is represented by an alternating sequence of grey-greenish phyllite, phyllitic quartzite sequence with rare carbonates, intercalated with thick white-light grey fine to medium grained, locally gritty quartzite. The general trends of the rock are from ENE-WSW and NNW-SSE with low to moderate dips directed towards north to north-east.

Engineering geological Mapping shows four geotechnical units in the study area. They are the colluvial deposits, alluvial deposits, Quartzite bed rock and Phyllite bed rock. In the Omchhu area alluvial deposits are mainly the most extensive and is present all along on the banks and along the flow channel. The area being gentle, this flow deposit has covered a large area at the middle and lower portions of the study area.

The major discontinuities present in the study area are the beddings, foliation, joints and faults. The general regional strike of the rock is from East-West with northerly dips ranging from 15° to as high as 71°. In few cases, slight displacement of about 10 cm to 15 cm was observed in the faults. Fault gouge can be observed in few fault lines and especially near to the confluence of Namantri and Omchhu. This faulting is also considered one of the contributing factors for the failure of the slope.

Major factors determining the stability of a slope are Slope geometry, Soils and geology, Slope hydrology, slope angle, load, vegetation, surface water and ground water. Triggering mechanisms are in the landslides are mostly Rainfall though Seismic activity also contributes to slides but are not very common. Phuentsholing being located in the foothills the average annual rainfall ranges from 5000mm- 6500 mm, which indeed is high.

Literature review shows that a rainfall of 10 h or less requires a rainfall intensity in excess of 12 mm h^{-1} to trigger failure, but a rainfall duration of 100 h or longer with an average intensity of 2 mm h^{-1} can also trigger landslides in the Himalaya. Moreover, in the daily rainfall scenario, one study concluded that when daily rainfall amount exceeds 144 mm, there is always risk of landslides in Himalayan slopes.

Slope analysis using SSPC, the probability of the slope to remain stable is less than 5% for exposures RO-3, RO-12 and RO-20. The probability to be stable is found to be above 95% for the exposures RO-7, RO-11 and RO-15. The probability to be stable is found to be above 70% for the exposure RO-1. The probability to be stable is found to be above 30% for the exposure RO-2.

The detailed rock mass weathering classification gives 5 rock exposures in class C. This class has rocks which are significantly weathered. Three exposures fall in class B. This class has rocks which are slightly weathered. For the rest 2 exposures, they fall in class S1. This class has rocks which are geotechnically soil with relict discontinuities.

The Skiwedge Stereonet also shows significant slope failures due to wedge and/or planar failures. The weathering classification gives values for middle to lower rock classes, which suggests that weathering grade is high.

Sieve Analysis result shows the material in most of the area is not distributed in a wide range, so is a uniform soil with bad compaction characteristics. The coefficient of Permeability (k) values ranges from 2.20 x 10-2 cm/sec falling in medium grained sand to 2.753 cm/sec falling in fine grained gravel group.

The Proctor Compaction Test result shows Maximum Dry Density ranging from 1.66 to 1.923 g/c.c. The Optimum Moisture Content ranges from 8% to 10.50%. The specific gravity ranges from 2.61 to 2.71 g/cc. The permeability values range from 1.027 X 10-3 cm/sec to 2.991 X 10-3 cm/sec. The Consolidation analysis shows the result of the coefficient of volume change ranging from 1.5 X 10-4 cm2/kg to 2.7 X 10-4 cm2/kg with the corresponding compression index ranging from 1.1269 X 10-1 to 2.5838 X 10-1.

It is observed that the bearing capacity of the soil is low to nominal for foundations as in all the locations as the values are below 24 t/m2. The get a better bearing capacity of the soil, either the footing sizes needs to be increased or the depth of the foundation is to be increased. The best option is to increase both the footing sizes and depths of foundations. Bearing capacity of the foundations can also be improved by keeping the foundation dry which can be attained by constructing proper and deep cut off drains, increasing the footing size and also by increasing the depth of the foundation.

The Seismic Refraction Survey conducted for Omchhu, Phuentsholing Thromde shows four to five layers in all the seismic lines. It is expected that the first layer with compression velocities in the range of 400 m/s to 1800 m/sec could be related to unconsolidated overburden materials as Silt, Sand and boulders and the lower layer with recorded velocity in a range of 1800 m/s to 4000 m/s indicates the presence of bed rock, consisting of highly weathered and fractured phyllite or quartzite.

The major hazard for the study area is from landslide triggered by rain compounded by fragile geology, topography, landuse and geomorphology.

The instability can be minimised by carrying out mitigation measures through slope geometry change, minimizing ground water influence and strengthening or supporting slopes. Few recommended measures are stepped RCC/RMM walls, Gabions and garland drains.

The Standard Penetration Test conducted in the field area shows values from 7 to 20 ’N’ values. The Ultimate Bearing Capacity calculated after necessary corrections for overburden pressure ranges from 149 kN/m2 to 537 kN/m2 for 1 metre foundation width. The Ultimate Bearing Capacity calculated for 3 metres foundation width is from 201 kN/m2 to 785 kN/m2.

The Ultimate Bearing Capacity calculated from the plate load test ranges from 251 kN/m2 to 360 kN/m2. The values obtained from this PLT test are also not very high.

The three (3) Portable Penetration Test conducted in the field area shows values from 7 to 21 'N' values. The Ultimate Bearing Capacity calculated ranges from 105 kN/m2 to 190 kN/m2 for 2 metre foundation width. The Ultimate Bearing Capacity calculated for 6 metres foundation width is from 79 kN/m2 to 145 kN/m2.

The Direct Shear Box Test result shows cohesion from 0 to 0.05 kg/cm2 and the internal friction angle of 21°04' to 26°33'. Taking the obtained shear parameters, bearing capacity was calculated for 3 metres foundation depth for a square footing size of 4 X 4metres, which when calculated using local shear failure criterion, the values range from 13.64 t/m2 to 23.48 t/m2. The factor of safety considered in the calculation of safe bearing capacity is 2.

Considering the nature of sub-surface strata, type of proposed structures, expected scour and loads on foundations, appropriate foundation shall be recommended;

For satisfactory performance of a foundation, the following criteria must be satisfied;
(i) The foundation must not fail in shear.
(ii)The foundation must not settle by an amount more than the permissible settlement.

The smaller of the bearing pressure values obtained according to (I) and (ii) above, shall be adopted as the allowable bearing capacity.

9 Formulation of Climate Resilient Flood Mitigation Measures

Different measures might be adopted to reduce the flood/erosion losses and protect the flood plains. Depending upon manner in which they work, flood protection and flood management measures may be broadly classified as non-structural measures and structural measures. In this section, conceptualization of some climate resilient structural as well as non-structural measures has been outlined and probable design considerations have been described on basis of experiences of study site visit.

9.1 Principle of Climate Resilient Measures

Principles or considerations behind the conceptualization of climate resilient measures to mitigate flood hazard in Omchhu are:

14. Structural measures are climate resilient, environment friendly and sustainable
15. Climate resilient measure has been considered for extreme discharge of 660 m^3/s (i.e., 1 in 100 ARP) including climate change scenario.
16. The entire stretch of Omchhu from Bailey bridge at Kabraytar has been considered for a continuous flood defense scheme.
17. No protection for about 20 m both upstream and downstream of the existing bridges (Bailey Bridge, Curvilinear Bridge, RSTA Bridge, Pedestrian Bridge, and Omchhu bridge) has been proposed as this could potentially undermine the bridge foundations.
18. For the severe active landslides at 3 locations, specific measures have been proposed.
19. Allowing smooth passage of maximum flow considering natural slope of the stream, morphological aspect, required width & uplift pore water pressure.
20. Availability and accessibility to potential construction materials
21. Potential usage and benefit of excavated sediment in or outside of country
22. Ease of construction
23. Potential to develop maximum land for landscaping purpose has been considered.
24. Perceptions of local engineers as well as local stakeholders
25. Budget constraints of local administration for implementation and maintenance
26. Do ability and durability

On the basis of the above principles following structural measures have been outlined which is assumed to address as a strong flood defense mechanism along the entire Omchhu river.

9.2 Design Guidelines

9.2.1 Design Discharge

We have selected 660.89 m^3/s of flow from SCS CN Method using Monte Carlo Simulation as the ultimate maximum flow for 1:100 AEP for the main Omchhu channel. We believe that this flow will also take into consideration the sediment loads, fluctuations in the flows and errors in any estimation considering it as a worst possible scenario.

Further, for the three tributaries, the same model results were used to compute the discharge from each tributary based on their respective catchment area. The summary is as follows;

Table 40: Discharge of tributaries

Tributary Name	Catchment Area (km^2)	Sub-basin	Sub-basin discharge $m^3/s/km^2$)	Tributary Discharge (m^3/s)
Namantari Khola (Bogatey Bari)	0.90	Sub Basin 3	31.5	28.17
Ramitey Chhu (Nima Colony)	1.90	Sub Basin 4	25.2	47.90
Kharaley Chhu (Above Curvilinear)	0.31	Sub Basin 4	25.2	7.82

9.2.2 Scour Depth

The Hydraulic Report carried out the scour depth analysis using Both Blench and Lacey's method. The average scour depth is estimated at 2.3 m and a maximum of 6.83 m using Lacey's method which calculates higher values than the Blench method.

It recommended that the scour depth values as calculated with Lacey's formula when redesigning the embankment retaining walls. However, the individual values from the HEC-RAS model varies within a short span, making it impractical for construction and design purpose. Therefore, a reclassification of the same was carried out as follows;

Table 41: Reclassification of scour depth

Obtained Values	Reclassified Values
2.3	2.3
2.6	2.7
2.67	2.7
2.8	2.8
3.06	3.1
3.68	3.7
3.71	3.7
3.84	3.85
4.39	4.5
5.31	5.5
5.78	5.8
6.83	6.85

Figure 103: Length of protection measures with respective scour depth

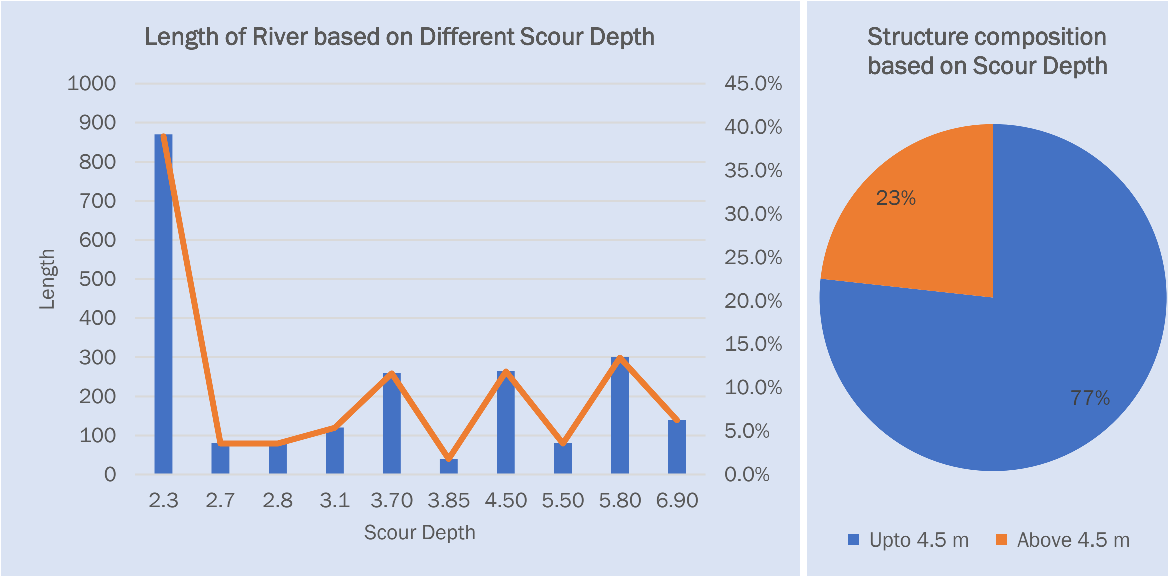
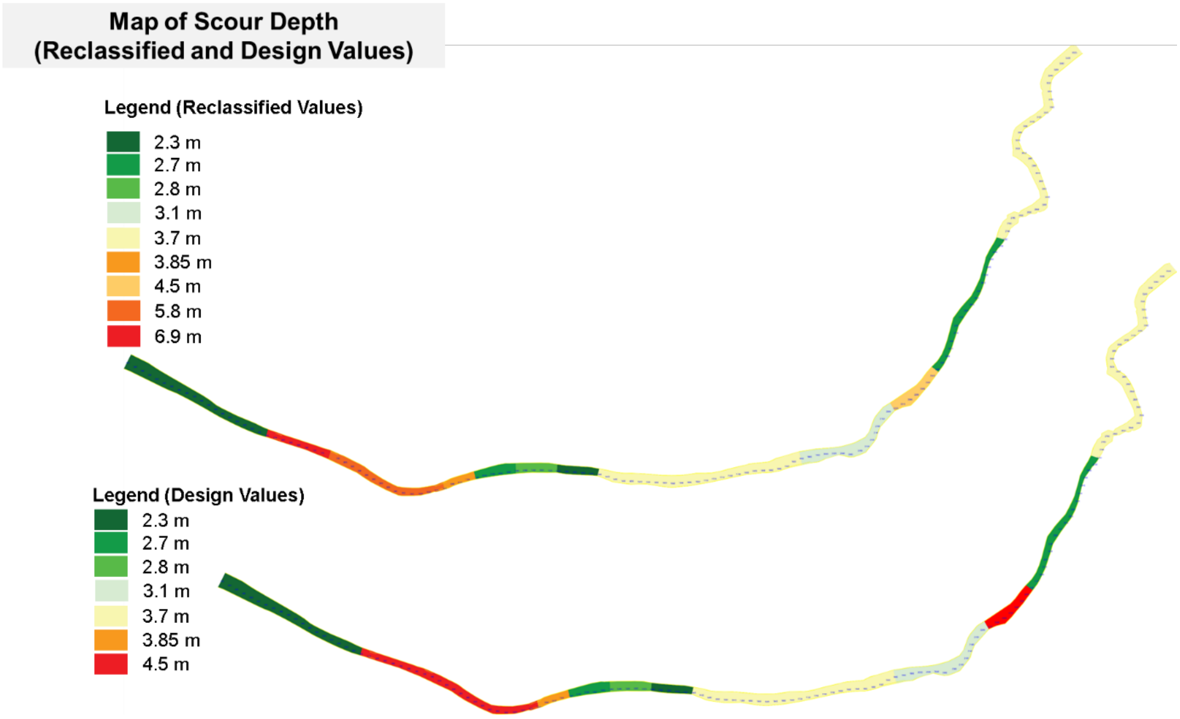


Figure 104: Map of reclassified scour depth values



Based on the engineering design team discussion, it was agreed that the scour depth consideration beyond 4.5 meters would again become challenging for construction, and is likely to meet the rock bed level. It would also not be feasible for excavation resulting in damage to associated building and infrastructure.

Therefore, in order to ensure that the design is robust, alternative engineering solutions to reduce the impact of local scouring has been proposed to account for limiting the scour depth to 4.5m. Stabilization measures such as PCC will be required at that depth if the foundation level is not found suitable, or the foundation length shall be reduced if bed rock is encountered before the foundation level.

9.2.3 Sediment Analysis

The confluence area (Omchhu and Amochhu) has experienced heavy sedimentation. The deposits raise the water surface profile, reducing freeboard and hence the flood protection standard offered by the existing flood embankments. While the PTDP project will be lowering the bed level to avoid backflow effect, the issue of sediment clogging on the immediate upstream of Omchhu bridge would remain.

As documented under the Hydraulic report, "Quantifying the amount of sediment available during a large flood is difficult without extensive study but given the steep unstable hillslopes of the upper watershed upstream of town, the likelihood exists that the sediment supply will exceed the transport capacity of the stream such that rapid deposition should be expected wherever the transport capacity is reduced even slightly (e.g., at the new bridge)."

Therefore, based on the field observations, it is decided to have structures to trap the sediment along the main channel. The consideration here would be from two aspects;

- The accessibility of vehicles to dredge the trapped sediment
- The aesthetic appeal that the perennial flow of water would provide for the Omchhu riverfront development project

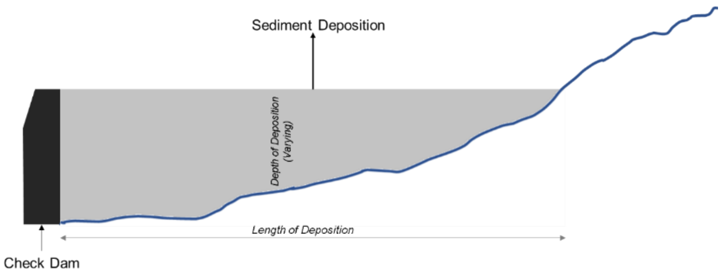
The installation of such sediment trap would naturally result in backflow level increase, and where such structures has been proposed, the wall height has been proportionately increased.

Figure 105: Omchhu Bridge under danger of flooding in July 2019

Source 16: Business Bhutan (July 17, 2019)

Figure 106: Sediment dredging immediately after heavy rainfall (June 2022)

Figure 107: Representational L-Section of the sediment trap



9.2.4 Velocity of the Channel

Velocity of the channel considering the bridges has been provided in the 2020 Hydraulic report. The velocity has been provided at different chainages; however, the information is not based on a uniform increment of chainages. The closest velocity based on the new chainage was selected from the data. This resulted in 44 unique velocities, ranging from 2.71 m/s to 9.85 m/s with an average of 6.59m/s.

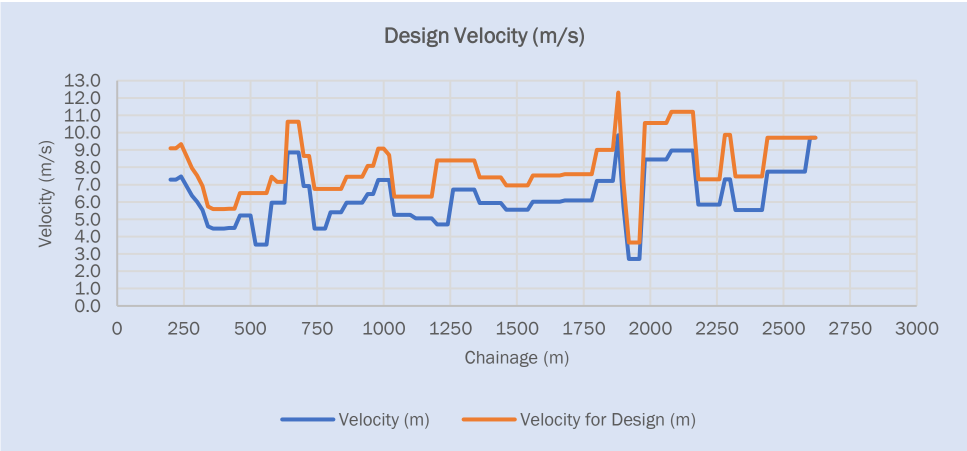
It was observed that the HEC-RAS computed velocities were among other factors, based on Manning's coefficient of are 0.035 and 0.05 for winding natural streams with weeds and mountain streams with rocky beds respectively. It was also noticed that the current velocity profiles changes abruptly within relatively short period based on the model; it is perceived that such changes are due to existing debris along the channel. However, after the clearing of debris along the channel, and construction of reinforced concrete walls along the banks, the roughness co-efficient would decrease considerably. The design considered the manning's co-efficient to be close related to Lined or Constructed Channels (Gravel bottom with sides of formed concrete) which has a normal value of 0.020. An example of the change in velocity with lower manning's value is shown below;

Figure 108: Change in velocity after intervention

Category	n	Discharge	Width	Height	R	Slope	Velocity	Increase
Current-0.035	0.035	660.40	23.01	4.50	3.23	0.01	6.78	
Gravel bottom with sides of formed concrete	0.020	660.40	23.01	4.50	3.23	0.01	11.86	75%

Therefore, based on the site conditions and expected intervention, the velocity for design purpose was increased by 25-35% in certain locations where the preceding or succeeding velocity was modelled to have much higher value. Subsequent impact due to higher local scouring will be considered by proposing protection measure in the design.

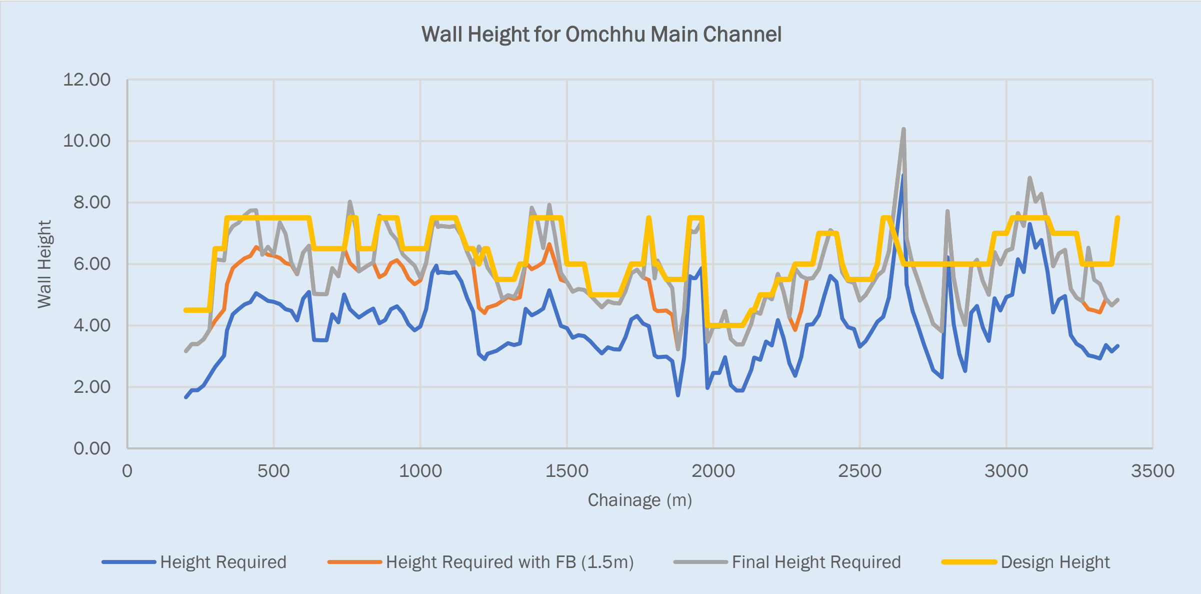
Figure 109: Design velocity for the structures



9.2.5 Wall Height

The resultant wall height based on the above factors, and in line with the current channel width, to accommodate a discharge of 660 m^3/s is shown below. Pursuant to IS Code 12094 (2000), the free board has been kept as 1.5m above the required height. Since the wall height changes continuously, from the design and construction aspect, it is only reasonable to provide a uniform height for the same stretch accommodating the height.

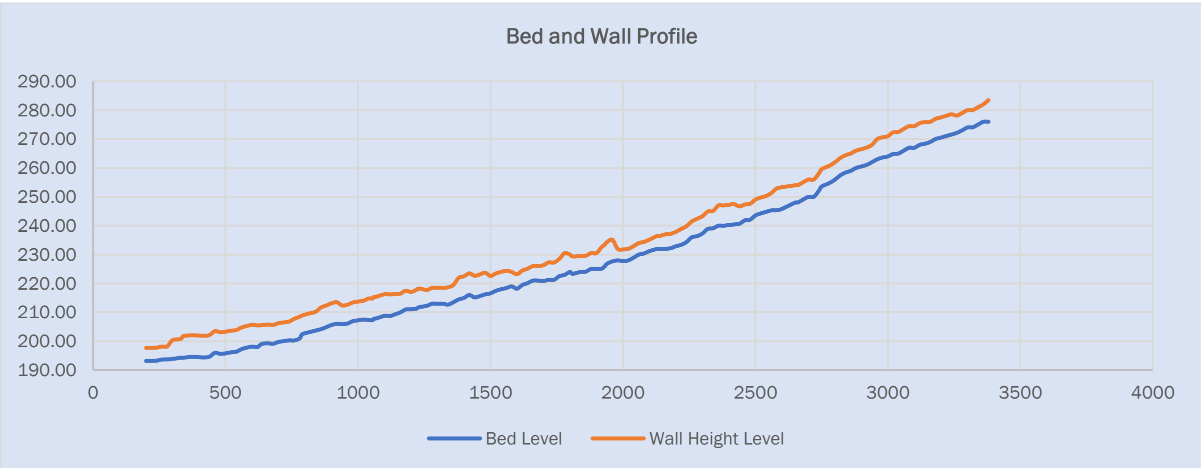
Figure 110: Computed and Design Wall Height



9.2.6 Bed Level

The Bed Level has been fixed as per the Longitudinal Section of the river. The bank level will be lowered to reach the proposed bed level across the river. Both the banks will have the bed level maintained at the same elevation to ensure that the wall height proposed is equally responsive on either side.

Figure 111: Bed Level and Wall Elevation



9.3 Mitigation Options

Mitigation measures have been prepared for a number of possible risks, based on the existing scenario and findings of the study. The following areas of mitigation has been formulated for the Omchhu river;

Table 42: Mitigation interventions identified for Omchhu

Sl.N$^{o.}$	Type of intervention	Purpose	Locations
1.	Bank Protection	Protect the banks from failure, and overflow of the river.	Along the Omchhu main channel stretch, and stretch of tributaries (Ramitey Khola, Namantari Khola, Kharaley Khola)
2.	Scour Protection	Protect against scouring by reducing velocity and impact, thereby reducing net soil loss	Along the Omchhu main channel stretch
3.	Cascades	Reduce the velocity impact to the main channel	Water discharge points from three tributaries
4.	Check Dams	Velocity reduction, perennial water flow (for aesthetics), and sand collection	Along the Omchhu main channel stretch
5.		Reduction of soil erosion and debris	Along the landslide areas (Nima Colony, Bogatey Bari, L/S above Curvilinear Bridge)
6.	Bio Engineering	Stabilizing the land profile	Along the landslide areas (Nima Colony, Bogatey Bari, L/S above Curvilinear Bridge)
7.	Ramps	Vehicular access to collect the debris	Strategic locations along the Omchhu main channel stretch
8.	Culverts/ Bridges	Channelization of water crossing roads	Ramitey Khola, Namantari Khola, Kharaley Khola

9.3.1 Bank Protection

The river/stream bank protection is the most important structure for the flood defense system for the river. The fluvial process of Omchhu resulting in eventual bank erosion, thereby, subsequent failure of road network and collapse of adjacent infrastructure. Therefore, bank protection through a comprehensive and continuous stretch of walls is felt necessary. Further, it was noted that the major drawback with the piecemeal river protection mechanism applied currently by Phuentsholing Thromde is the reason for eventual failure of the structure.

The current structures primarily lack sufficient scour depth levels, therefore, the options explored were considered to be those that could be constructed for heights inclusive of the respective scour depth at that location and the required height (including overboard, and backwater from sediment trap where applicable).

9.3.1.1 Options

Table 43: Wall type options explored

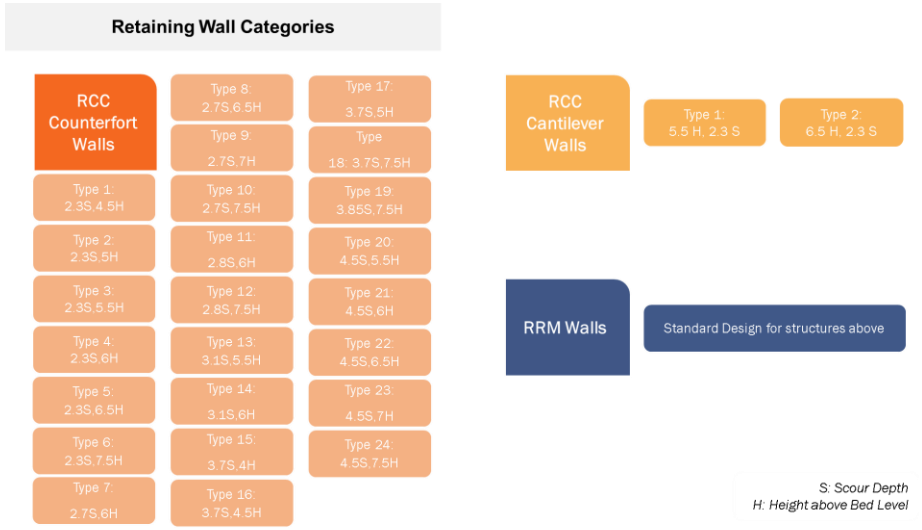
Structure	Steel Pipe Sheet Pile (SPSP)	Anchor Reinforced Earth Walls	Reinforced Concrete Walls	Geotextile Reinforced Earth Wall	Diaphragm Wall	Revetment
Construction Results	Relatively limited to creation of coffer dams	Limited exploration, primarily in hydropower projects	Familiar with constriction and technology in Bhutan	Limited exploration in road projects	Limited exploration at the PTTP Project	Familiar with constriction and technology in Bhutan
Reinforcement Materials	Not Applicable	Anchor plates & bars	TMT Bars	Geotextiles	TMT Bars	Not Applicable
River Cross Sectional Area	Obtain maximum C/S area	Obtain very high C/S area	Obtain high C/S area	Obtain high C/S area	Obtain very high C/S area	C/S Area is greatly decreased
Critical consideration	The front of the SPSP can be inspected and maintained, while inspection and maintenance behind the SPSP are difficult	The REW will fail against scouring, and might be retained only by anchors	The RCW will be protected against scoring based on sufficient base width	Failing of isolated portions resulting in overall weakening of the system	The failure mode will likely not occur based on the depth	Has been a rather unsuccessful mitigation on long term for rivers in Bhutan
Cost	******	****	***	**	*****	*
Judgement	[Rejected]	[Rejected]	[Selected]	[Rejected]	[Rejected]	[Rejected]

While the reinforced concrete walls have been selected based on the above factors, with regard to the Design Parameters, two types of walls had to be categorized. These include;

- Cantilever Walls: For wall height less than 8.00 meters
- Counterfort Walls: For wall height more than 8.00 meters

Counterforts help to reduce the bending moment at the wall stem. Apart from RCC walls, Random Rubble Masonry (RRM) walls also have been suggested at locations

Figure 112: Categories of retaining walls



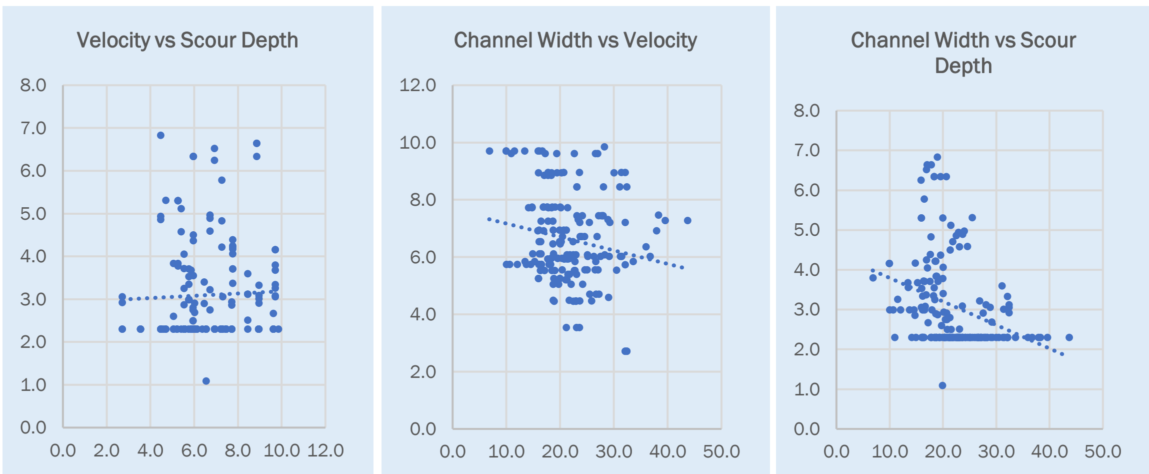
9.4 Scour Protection

The strongest negative correlation is between channel width and scour depth (-31.0%). The highest scour depth is mostly observed at locations where channel width is narrow. The details of individual scour depth are presented in Appendix 2

Both the sediment analysis report as well as the morphological study highlight that there is net erosion of the riverbed instead of sedimentation (however, majority of the sedimentation is at the point just upstream of the Omchhu Bridge).

Further, it is to be noted that the construction of retaining walls along the banks will result in disturbance of the bed, and subsequent compaction may not hold as strong as undisturbed bed. Therefore, to reduce the impact of local scoring a combination of the protection above the footing is proposed as shown in Figure

Figure 113: Relationship between velocity, scour depth, and channel width of Omchhu



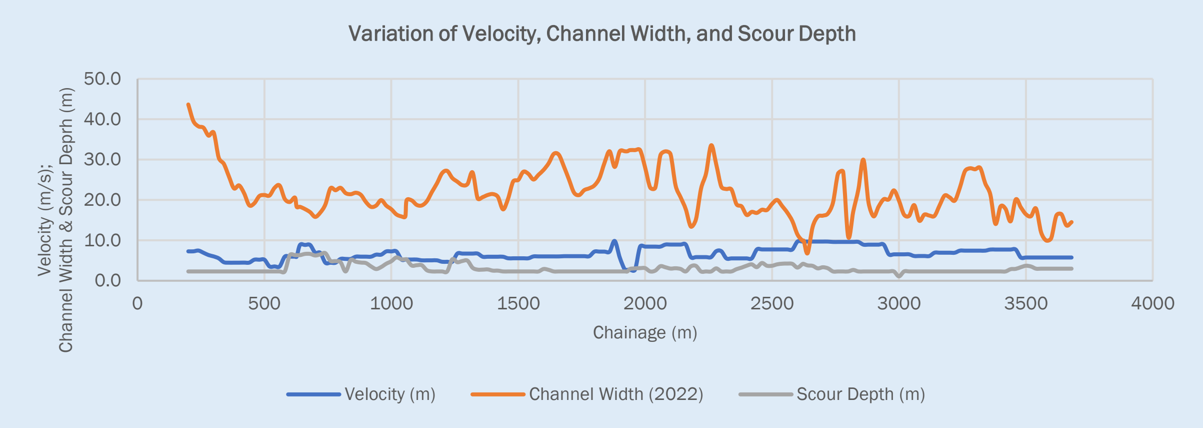


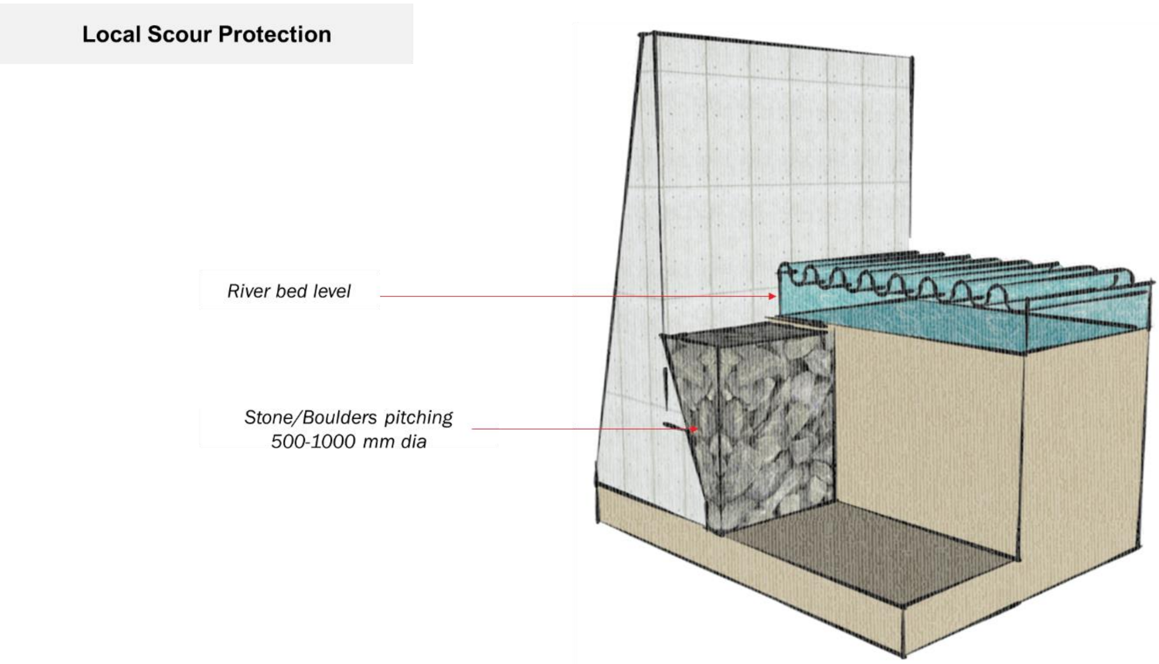
Table 44: Correlation between three variables

Parameter	Velocity	Channel Width	Scour Depth
Velocity	100%	-18%	4%
Channel Width	-18%	100%	-31%
Scour Depth	4%	-31%	100%

9.4.1.1 Options

While sufficient safety has been considered for the scour protection, to enhance the flood defense, it is proposed to have an indigenous mechanism of boulder pitching, followed by gabion pitching on top till the height of bed level.

Figure 114: Selected option for Local Scour reduction



9.5 Check Dams

Check dams/ Sabo dams are popular river structures, primarily for the purpose of erosion control on the hills and flow control along the rivers. There are two e

Generally, the check dams can be used to store water during post monsoon season, reduce erosion, modify the velocity of stream flows, trap amounts of channel sediment, and help stabilize channel side-slope[29]. A Japan International Cooperation Agency (JICA) project titled Data Collection Survey on Urban Development and Environment in the Kingdom of Bhutan (2014)[30], has stressed extensively on the use of Check/Sabo dam along the Omchhu as flood and landslide protection.

9.5.1.1 Options

Four kinds of specific check dams have been proposed considering various locations and its applicability;

- Reinforced concrete cement check dams along the Omchhu main channel for velocity control, sediment trapping, and post-monsoon water retention for aesthetic purpose
- Gabion check dams along the landslide, followed by Sandbag check dam, and bamboo check dam on the top of the slide

9.6 Cascades

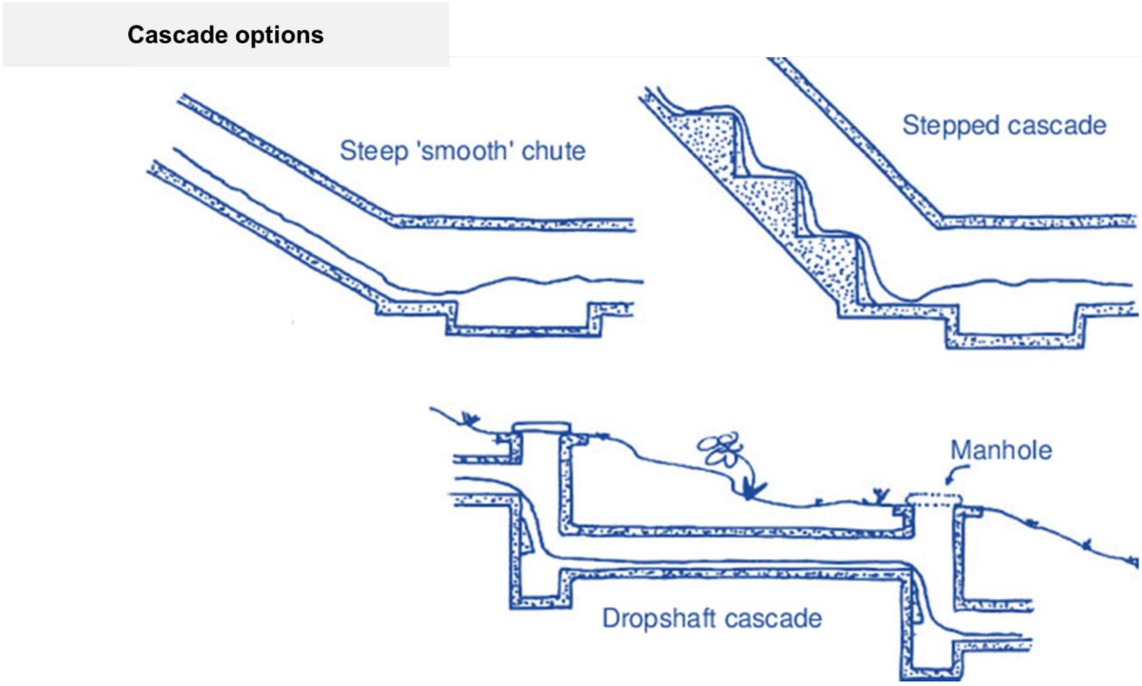
Waterfalls of small height and lesser steepness are called cascades. Cascade drain is meant to collect runoff from both the slope and the catchment area upslope and lead the water to convenient discharge points. The installation of cascade drain will aid in improving the slope stability and avoiding excessive erosion and infiltration process. Due to high flow velocity of rainwater, energy dissipators such as steps are introduced inside the channel.

The options available for this intervention include;

- Sloped Chute Drain
- Slopped-Stepped Chute Drain
- Drop shaft cascade
- Stepped Chute Drain
- Precast drains
- On-site construction

[29] RSPN Bhutan (https://www.rspnbhutan.org/introduction-of-check-dams-in-wamrong/)
[30] Available at: https://openjicareport.jica.go.jp/pdf/12148565_01.pdf

Figure 115: Models of Steep Chute



Source 17: Chanson, H. (2000)

9.7 Landslide Protection

For the three active landslides that are of considerable size, the options selected in the preceding order of its reach from the slope toe includes the following;

- Gabion check dams
- Sandbag check dams
- Bamboo check dams
- Jute matting
- Plantation

Figure 116: Road failure due to debris from landslide

9.7.1 Gabion check dams

The practice of placing gabion check dams is common along highways in Bhutan. These will be placed just immediately above the roads (end of the slide). The primary purpose will be velocity reduction, and not as sediment control and should not be considered a sediment trapping device.

9.7.2 Sandbag check dams

Sandbags are cost effective as compared with concrete check dams. Furthermore, as the distance from the road increases, construction of the concrete or gabion check dam becomes more difficult. Therefore, sand check dams has been proposed.

- Sandbags are typically filled with sand, aggregate, gravel, or compost.
- Compost filled bags are considered to provide improved water treatment through filtration and adsorption. This system included compost-filled Filter Socks.
- Typically used in drains less than 500mm deep, with a gradient less than 10%.
- These check dams are typically small (in height) and therefore less likely to divert water out of the drain.
- Can be used as a minor sediment trap.

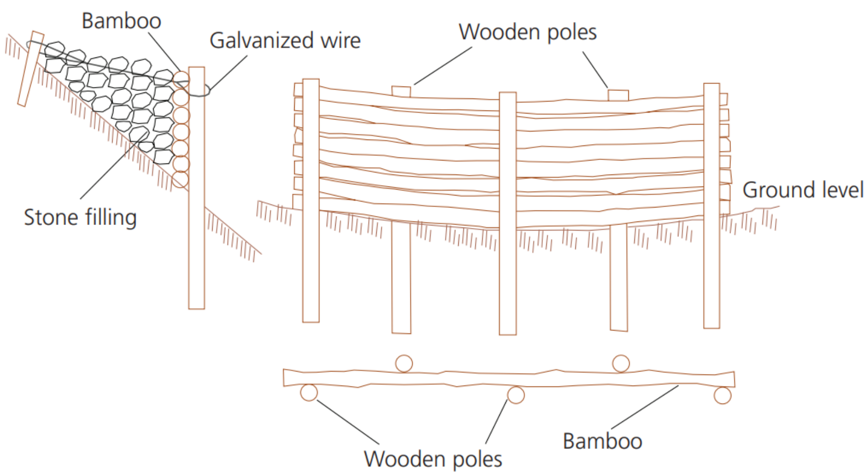
Osuagwu et al., (2013) found that, “Transverse interlocking arrangements of the sandbags yield better results than longitudinal arrangement. Cascading of the downstream face of the sand bags helps in dissipating the fall energy of water flowing over the bags and thus reduces the problem of scouring of downstream toe.”

9.7.3 Bamboo check dams

Bamboo has traditionally been used for soil bioengineering applications and has often been hailed as a species that is useful for reinforcing soil with regard to erosion and shallow landslides (Chaulya et al., 1999; Storey, 2002).

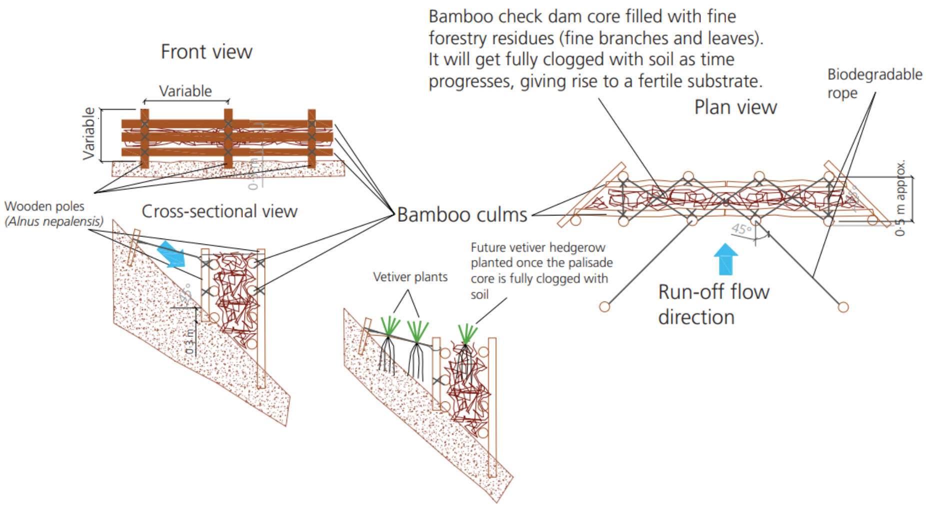
Although bamboo check dam construction is relatively well known in Bhutan, the proposed construction technique chosen for this project is derived from research by Tardio et al., (2017) titled 'Bamboo structures as a resilient erosion control measure' which proposed a novel approach as shown in the Figure 118.

Figure 117: Commonly constructed bamboo check dams



Source 18: Sthapit and Tennyson (2014)

Figure 118: Bamboo check dam general arrangement

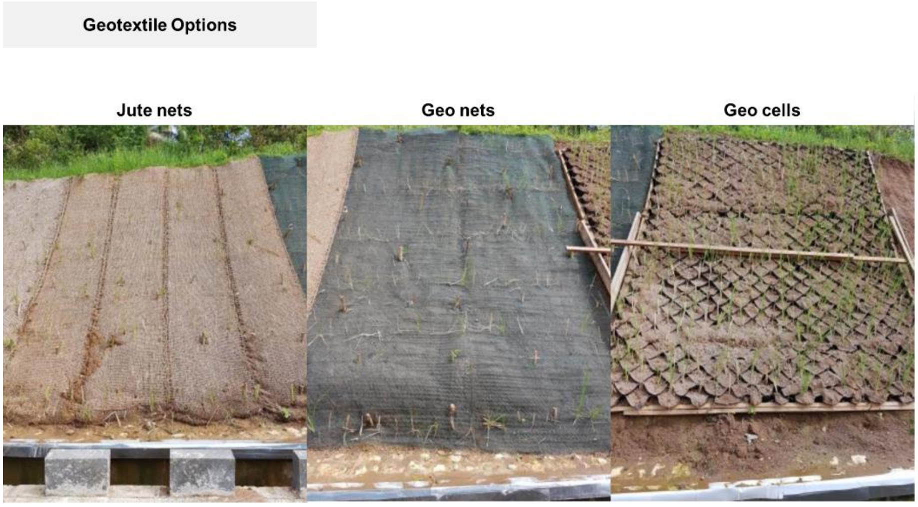


Source 19: Tardio, Mickovski, Stokes and Devkota (2017)

9.7.4 Soil erosion control

On the active landslides, it is determined that prevention and control measure such slope protection are necessary. Vegetation is the most frequent and organic component utilized to control erosion. By absorbing water that the soil is unable to absorb and by blocking wind that may blow topsoil off, it helps to protect the soil and prevent erosion. In addition to vegetation, geosynthetics like geotextiles are utilized to reduce soil erosion. The consultants looked into three of the most popular geotextiles: Cellular Confinement Systems, also known as "Geocell," Geosynthetic Erosion Mat, also known as "Geomat," and Coconut Fiber Net- "Coconet/ Jute net".

Figure 119: 2 Illustration of the Field Application of the Geotextiles



Source 20: Paz et al., (2018)

With reference to literature review and general experience of the consultants, a comparison was made among the available options, and the resultant selection was that of Jute nets.

Parameter	Jute net	Geomat	Geocell
Runoff	High	Low	Medium
Soil Loss	Low	Low	Medium
Vegetation	Medium	Low	High
Tensile Strength	Medium	Low	High
Price	*	**	***
Judgement	[Selected]	[Rejected]	[Rejected]

9.7.5 Dredging Ramps

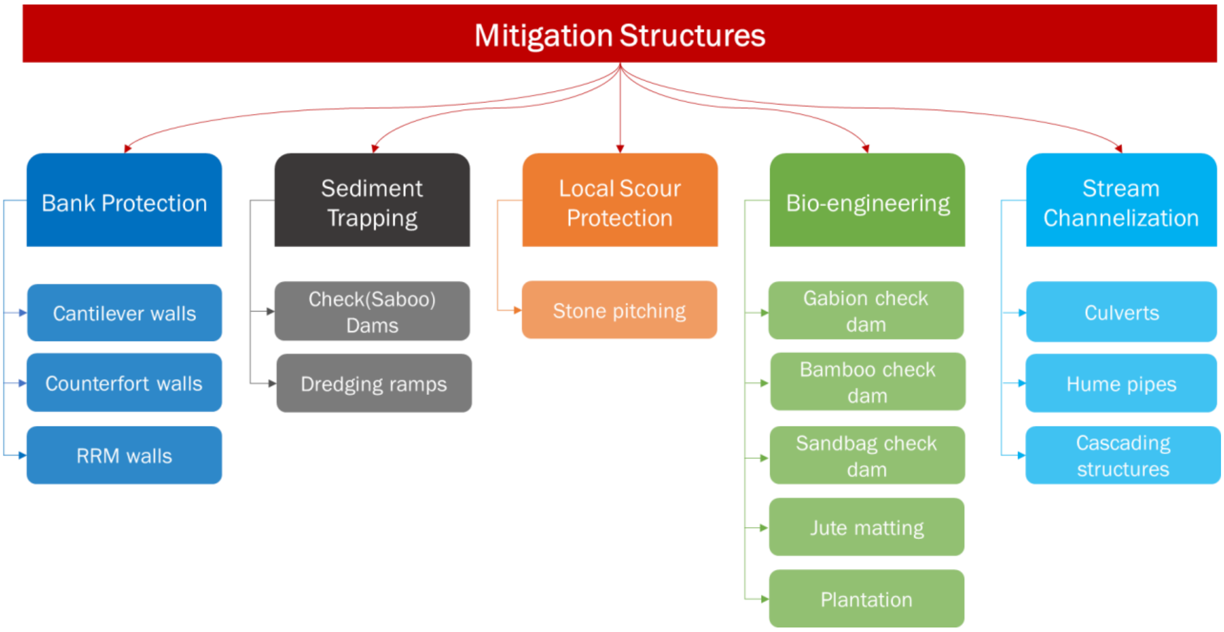
Dredging ramps are necessary for the Omchhu River for sediment dredging. The usual practice during winter months, is for excavators and trucks to enter from the mouth of the river (Amo chhu confluence), and enter below Omchhu bridge to collect the sediments accumulated upstream of the bridge.

However, under serious conditions posing high water level threat to the Omchhu bridge, temporary network is created to reach to the upstream directly from the road which results in damage of utilities and public infrastructure (such as foot paths). Apart from this reason, the fact that the RCC check dams along the Omchhu main channel will have to cleaned necessities the dredging ramps.

10 Detailed Design of Infrastructure

The following chart describes the summary of the selected mitigation structures. The appropriate design standards and procedures for certain structures have been presented in the report. For other structures, such as jute matting, or plantation, or dredging ramps, due to the familiarity of the practice and lack of absolute empirical design process, it has been omitted from structural design consideration.

Figure 120: Selected mitigation structures



10.1.1 Design of Counterfort Walls

A sample design format for Counterfort Wall No. 1-a; categorized as a wall with Scour Depth of 2.3m, Height above Bed Level as 4.5m, and Length of Panel as 9.5m is presented below. The differentiation based on such categories has resulted in 125 dimensions of counterfort wall design.

Table 45: Design Details for Counterfort Wall

PARAMETERS	Remarks	Values	Unit
WALL TYPE		1.a	Type
INPUT DATA			
Grade Of Concrete Fck		25.00	N/mm2
Grade Of Steel F Y		500.00	N/mm2
Angle Of Repose Of Soil Θ		30.00	DEG
Bulk Density Of Soil Ws		18.00	KN/m3
Soil Safe Bearing Capacity Sbc		380.00	KN/m2
Angle Of Surcharge Of Fill C		20.00	DEG
Scour Depth		2.30	
Coefft Of Friction µ		0.60	
Cos C		0.94	
Cos Θ		0.87	
Coefft Of Active Pressure Ka		0.41	0.334
Coefft Of Passive Pressure Kp		2.41	
Height Of Filling Hf		4.50	mtr
Surcharge Height		0.93	
Surcharge Ps		8.33	KN/m
PRELIMINARY DIMENSIONS			
Min Depth Of Fdn H		2.35	mtr
Calculated Base Width B(Min)	Bm	4.77	mtr
Base Slab Thick D(Min)		375.71	mm
Wall Thick At Bottom Stb(Min)		312.31	mm
PROVIDED DIMENSIONS			
Length Of One Panel		9.50	mtr
Total Ht Of Wall Ht=H+D		7.80	mtr
Provide Depth Of Fdn D =	1	1.00	mtr
Const A		0.00	
Clear Spacing Of Counterfort L	2.84	2.65	mtr
Thickness Of Counterfort Tc		0.390	mtr
C/C Spacing Of Counterfort Lc		3.04	mtr
Base Width Of Front Counterfort B2		1.87	mtr
Ht Of Front Counterfort Above Base Slab		2.50	mtr
Toe Length Tl	0.00	1.87	mtr
Heel Length Hl		2.54	mtr
Provide Base Width B	B	4.80	mtr
Base Width Of Rear Counterfort B1		2.54	mtr
Provide Base Slab Thick D		800.00	mm
Provide Stem Thick At Bottom Stb		390.00	mm
Provide Stem Thick At Top Stt	390	390.00	mm
SAFETY CHECK			
Check For Bearing Pressure			
Wt Of Base Slab/Footing	W1	96.00	KN/m
Wt Of Stem/ Wall Rectangle Part	W2	68.25	KN/m
Wt Of Stem/ Wall Triangle Part	W3	0.00	KN/m
Wt Of Rear Soil Over Heel	W4	341.74	KN/m
Wt Of Protection Materials Over Toe	W5	83.97	KN/m
Total Stabilising Vertical Force	W	589.96	KN/m
Horizontal Earth Pressure	Ph	234.31	KN/m
Horizontal Surcharge	Ps	66.05	KN/m
Total Horizontal Pressure	V	300.36	KN/m
Distance Of W1 From Toe Front Tip X1	X1	2.40	mtr
Distance Of W2 From Toe Front Tip	X2	2.26	mtr
Distance Of W3 From Toe Front Tip	X3	2.52	mtr
Distance Of W4 From Toe Front Tip	X4	3.53	mtr

Distance Of W5 From Toe Front Tip	X5	0.93	mtr
Ht Of Hort Force Y1 From Toe Top	Y1	2.33	mtr
Ht Of Sur Force Y2 From Toe Top	Y2	3.50	mtr
Dist Of Σvert Reaction From Toe Front Tip		2.83	mtr
Overturning Moment Mo		619.04	KN-m
Sum Of Moments		1049.35	KN-m
X Bar		1.78	mtr
Eccentricity		0.62	0.80
Lr Distance Of Reactio From Heel			
Safety Against Bearing			
Pressure At Toe Tip Pmax	Max 380	218.37	KN/m2
Pressure At Heel Tip Pmax	Max 380	27.45	KN/m2
Pressure At Toe Face Of Ver Stem	Max 380	144.15	KN/m2
Pressure At Heel Face Of Ver Stem	Max 380	117.18	KN/m2
FOS Against Overturning		2.43	SAFE
FOS Against Sliding		1.51	SAFE
DESIGN OF SHEAR KEY OR INCREASE WIDTH OF FDN			
Required for this wall?	No Shear Key Required		
IF Required, Design Calculations:			
Assumed Depth of the Shear Key(Ds)		0.30	
Passive Earth pressure		347.83	
Total Passive Earth pressure		104.35	
Horizontal Earth Presure		244.71	
Weight of soil between base and shear key base		25.92	
Sum of vertical forces		615.88	
FOS AGAINST SLIDING		1.94	SAFE with Shear Key
Reinforcement		360.00	mm2
Provide dia of steel bar		16.00	mm
Spacing of bars		218.54	mm
Adopted Spacing		220.00	mm
DESIGN OF BASE SLAB WITH CONTOURFORTS			
DESIGN OF TOE SLAB			
Pressure At Toe Wp		144.15	
Wt Of Soil Over Toe W4		69.27	
Wt Of Toe		20.00	
Effective Depth Of Toe D		725.00	mm
Span Of Toe L1		1.87	m
Wt Of Footing W4		20.00	KN/m
Max Bm At Base Of Toe Bottom Near Counterfort Edge Mt		-83.09	KN-m
Net Pressure Intensity Toe Tip		-94.87	
Pressure At Toe Face Below Stem		124.15	
Shear Force Vmax		246.47	KN
DESIGN OF TOE SLAB TO RESIST BENDING MOMENT			
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
Base width		1.00	Mtr
Max BM Mx		83.09	KN-M
BM = (Const*fck) bd^2		3.32	bd^2
Calculated Eff Depth of Slab		158.25	mm
Adopt Effective Depth d		720.00	mm
Use Dia of Slab rft		16.00	mm
Adopt Cover for Slab		75.00	mm
Over all Depth of Base Slab D		805.00	mm
Width of Slab considered for Cal		1000.00	mm
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
a=		8700.00	

b=		-435.00	
c=		0.16	
Min area of Tension Steel Ao=0.85*bd/fy		1224.00	Sqmm
Max area of Tensile Steel = 0.04 bD		32200.00	Sqmm
Provide Area of Tension Steel		1224.00	Sqmm
Area of One Bar		201.14	Sqmm
Spacing of Main Bars		160.00	mm
Adopted		160.00	
Min Area of Steel 0.12 %		966.00	Sqmm
Check for Min rft		OK	
Dist rft 0.15 % of gross area will be provided in the longitudinal direction		1207.50	Sqmm
Use		16.00	mm Dia bars as distribution Rft
Area of One Bar		201.14	Sqmm
Spacing of Distribution Bars		160.00	mm
Adopted		160.00	
DESIGN/ CHECK FOR TOE SLAB TO RESIST SHEAR			
Grade of Concrete M		25.00	
Effective Depth		720.00	mm
Over allDepth of Slab		805.00	mm
Dia of Shear rft		16.00	mm
Area of One Bar		201.14	Sqmm
Spacing of Bars		160.00	mm
Max Shear Force wL/2		246.47	KN
Percentage of Tensile Steel 100At/2bd = (at the end, alternate bar are bent up)		0.17	%
Design Shear Strength		0.311	N/ Sqmm
Calculated k Value		1.00	
For 805 mm thick slab, k=		1.00	
Permissible Max Shear Stress		0.311	N/ Sqmm
Nominal Shear stress Vu/bd		0.34	N/ Sqmm
Shear Check		**UNSAFE- Use Stirrups**	
Design of Stirrups			
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
Effective Depth of Beam		720.00	mm
Over all Depth of Beam		805.00	mm
Width of Beam		1000.00	mm
Max Shear Force wl/2 Vu		246.47	KN
Strength of Shear rft Vus=Vu-Tc bd		22242.61	N
Dia of Shear rft		12.00	mm
Area of One Bar		113.14	Sqmm
No of legged vertical stirrups		4.00	Nos
Area of Vertical Stirrup Rft Asv		452.57	mm
Spacing of Shear rft x=0.87 fy Asv d/ Vus		6370.00	mm
Check for Spacing		OK	
Min Area of Shear rft 0.4 b x /fy		336.00	Sqmm
Check for min Shear rft Area		OK	
DESIGN OF HEELSLAB			
Spacing between contourforts		2.65	
SPAN OF HEEL L2		2.54	M
PRESSURE AT HEEL Wp		27.45	KN/m2
WT OF SOIL OVER HEEL W4		126.00	KN/m
WT OF HEEL W1a		20.00	KN/m
MAX BM AT HEEL TOP at Counterfort Mh		103.83	KN-m
SHEAR FORCE Vmax		235.35	KN
Net download intensity of load		118.55	
DESIGN/ CHECK FOR HEEL SLAB TO RESIST SHEAR			
Grade of Concrete M		25.00	

Effective Depth		720.00	mm
Over allDepth of Slab		805.00	mm
Dia of Shear rft		16.00	mm
Area of One Bar		201.14	Sqmm
Spacing of Bars		160.00	mm
Max Shear Force wL/2		235.35	KN
Percentage of Tensile Steel 100At/2bd = (at the end, alternate bar are bent up)		0.17	%
Design Shear Strength		0.311	N/ Sqmm
Calculated k Value		1.00	
For 805mm thick slab, k=		1.00	
Permissible Max Shear Stress		0.311	N/ Sqmm
Nominal Shear stress Vu/bd		0.33	N/ Sqmm
Shear Check		**UNSAFE-Use Stirrups**	
Design of Stirrups			
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
Effective Depth of Beam		720.00	mm
Over all Depth of Beam		805.00	mm
Width of Beam		1000.00	mm
Max Shear Force wl/2 Vu		235.35	KN
Strength of Shear rft Vus=Vu-Tc bd		11126.24	N
Dia of Shear rft		12.00	mm
Area of One Bar		113.14	Sqmm
No of legged vertical stirrups		4.00	Nos
Area of Vertical Stirrup Rft Asv		452.57	mm
Spacing of Shear rft x=0.87 fy Asv d/ Vus		12740.00	mm
Check for Spacing		OK	
Min Area of Shear rft 0.4 b x /fy		336.00	Sqmm
Check for min Shear rft Area		OK	
DESIGN OF STEM WALL			
SPAN / HT OF STEM WALL L3		7.00	M
HORIZONTAL PRESSURE ON STEM WALL	Ph	59.12	KN/m2
CLEAR SPACING OF COUNTERFORT L	L	2.65	M
MAX BM AT BOTTOM OF WALL Mw	Mw	51.78	KN-m
SHEAR FORCE Vmax	V	78.25	KN
DESIGN OF STEM WALL TO RESIST BENDING MOMENT			
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
Base width		1.00	Mtr
Max BM Mx		51.78	KN-M
BM = (Const*fck) bd^2		3.32	bd^2
Calculated Eff Depth of Slab		124.92	mm
Adopt Effective Depth d		310.00	mm
Use Dia of Stem Wall rft		16.00	mm
Adopt Cover for Stem wall		40.00	mm
Over all Depth of Base Slab D		360.00	mm
Width of Slab considered for Cal		1000.00	mm
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
a=	0.87 *(fy^2/fck)	8700.00	
b=	-0.87 fy	-435.00	
c=	m= Mu/(bd^2)	0.54	
p %= (-b- sqrt(b^2-4ac))/2a		0.127	
At		395.00	Sqmm
Min area of Tension Steel Ao=0.85*bd/fy		790.50	Sqmm
Area of One Bar		201.14	Sqmm
Spacing of Main Bars		250.00	mm
Min Area of Steel 0.12 %		432.00	Sqmm

Check for Min rft		OK	
Dist rft 0.15 % of gross area will be provided in the longitudinal direction		540.00	Sqmm
Use		16.00	mm Dia bars as distribution Rft
Adopted		370.00	
DESIGN/ CHECK FOR STEM WALL TO RESIST SHEAR			
Grade of Concrete M		25.00	
Effective Depth		310.00	mm
Over allDepth of Slab		360.00	mm
Dia of Shear rft		16.00	mm
Area of One Bar		201.14	Sqmm
Spacing of Bars		250.00	mm
Max Shear Force wL/2		78.25	KN
Percentage of Tensile Steel 100At/2bd =		0.26	%
(at the end, alternate bar are bent up)			
Design Shear Strength		0.371	N/ Sqmm
Calculated k Value		1.00	
For 360 mm thk slab, k=		1.00	
Permissible Max Shear Stress		0.371	N/ Sqmm
Nominal Shear stress Vu/bd		0.25	N/ Sqmm
Shear Check		**Safe**	
DESIGN OF FRONT COUNTER FORT (OVER TOE) TO RESIST BENDING MOMENT			
Necessity of Toe Contourfoot	Required		
Length of Front Counter Fort	1.87	Mtr	
Earth Pressure at tip of Counter Fort w1	602.44	KN/Sqm	
Earth Pressure at Stem of Counter Fort w3	377.04	KN/Sqm	
Total upward Pressure	913.86		
Ht of Front Counterfort Above Base Slab	2.15	Mtr	
C/C dist between Counter Fort	3.04	Mtr	
x bar	0.86		
Max BM due to Earth Pressure	787.23	KN M	2788.06
Max SF due to Earth Pressure	491.98	KN	total upward pressure-((moment/ht of cft)*(ht/toe length))
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
Base width		0.39	Mtr
Max BM Mx		787.23	N-MM
BM = (Const*fck) bd^2		3.32	bd^2
Calculated Eff Depth of Slab		779.95	mm
Adopt Effective Depth d		780.00	mm
Use Dia of Slab rft		20.00	mm
Adopt Cover for Slab		75.00	mm
Over all Depth of Base Slab D		865.00	mm
Width of Slab considered for Cal		390.00	mm
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
a=	0.87 *(fy^2/fck)	8700.00	
b=	-0.87 fy	-435.00	
c=	m= Mu/(bd^2)	3.32	
Min area of Tension Steel Ao=0.85*bd/fy		517.14	Sqmm
Max area of Tensile Steel = 0.04 bD		13494.00	Sqmm
Provide Area of Tension Steel		2857.00	Sqmm
Area of One Bar		314.29	Sqmm
No of Main Bars		10.00	Nos
Min Area of Steel 0.12 %		1038.00	Sqmm
Check for Min rft		OK	

Temp rft 0.15 % of gross area will be provided in the longitudinal direction		1297.50	Sqmm
Use		16.00	mm Dia bars as distribution Rft
Area of One Bar		201.14	Sqmm
Spacing of Distribution Bars		160.00	mm
Adopted		160.00	
Provided Depth of Front Counter Fort	1250	mm	1250
Shear Force at d away from Stem			
Inclined Bars		6.00	Nos
DESIGN/ CHECK FOR FRONT COUNTER FORT WALL TO RESIST SHEAR			
Grade of Concrete M		25.00	
Effective Depth		780.00	mm
Over allDepth of Slab		865.00	mm
Dia of Shear rft		20.00	mm
Area of One Bar		314.29	Sqmm
No of Bars		10.00	Nos
Max Shear Force wL/2		491.98	KN
Percentage of Tensile Steel 100At/2bd =		1.03	%
(at the end, alternate bar are bent up)			
Design Shear Strength		0.649	N/ Sqmm
Calculated k Value		1.00	
For 865mm thk Slab, k=		1.00	
Permissible Max Shear Stress		0.649	N/ Sqmm
Nominal Shear stress Vu/bd		0.63	N/ Sqmm
Shear Check			**Safe**
Provide 10 mm dia 2 legged stirrup at 200 C/C to connect with stem			
DESIGN OF REAR COUNTER FORT (OVER HEEL) TO RESIST BENDING MOMENT			
Height of Front Counter Fort	7.00	Mtr	
Base Widh of Front Counter Fort	2.54	Mtr	
Inclination of Counter Fort θ=	0.3486	Radian	19.97
Ht of Earth Filling Above GL	6.80	Mtr	
earth pressure acting on contourfort	154.05		
Net downward pressure at heel edge	349.10		
Net downward pressure at heel below stem	76.59		
Shear force	523.78		
Moment	1222.15		
Max BM	943.31	KN m	
SF/HorizThrust	416.17	KN	
Effective depth d	2,311	mm	
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
Base width		0.39	Mtr
Max BM Mx		943.31	N-MM
BM = (Const*fck) bd^2		3.32	bd^2
Calculated Eff Depth of Slab		853.78	mm
Adopt Effective Depth d		1000.00	mm
Use Dia of Slab rft		20.00	mm
Adopt Cover for Slab		50.00	mm
Over all Depth of Base Slab D		1060.00	mm
Width of Slab considered for Cal		390.00	mm
Grade of Concrete M		25.00	
Grade of Steel Fe		500.00	
a=	0.87 *(fy^2/fck)	8700.00	
b=	-0.87 fy	-435.00	
c=	m= Mu/(bd^2)	2.42	
m= Mu/(bd^2)		2.42	
p %= (-b- sqrt(b^2-4ac))/2a		0.637	
At		2486.00	Sqmm

Provide Area of Tension Steel		2486.00	Sqmm
Area of One Bar		314.29	Sqmm
No of Main Bars		8.00	Nos
Min Area of Steel 0.12 %		1272.00	Sqmm
Design of Horizontal Ties			
Force causing seperation		207.33	
Steel area required		753.92	
Use		16.00	mm Dia bars as horizontal ties
Area of One Bar		201.14	Sqmm
Spacing of Distribution Bars		270.00	mm
Adopted		270.00	
Design of vertical Ties			
Net downward pressure at heel end		414.92	
Net downward pressure at heel side below stem		100.86	
Net downward pressure at heel end		361.64	
Net downward pressure at heel side below stem		87.91	
Steel area near the heel end		1315.05	
Steel area on heel side near the stem		319.68	
Use		16.00	mm Dia bars as horizontal ties
Area of One Bar		201.14	Sqmm
Spacing of bars near heel		150.00	mm
Spacing of bars near stem		630.00	mm
Adopted		150.00	
DESIGN/ CHECK FOR REAR COUNTER FORT WALL TO RESIST SHEAR			
SF/HorizThrust wh^2*(1-sinφ)/(1+sinφ)*3/2	416.17	KN	
Net SF= F-M*tan θ/d'	93.96	KN	
Grade of Concrete M		25.00	
Effective Depth		1000.00	mm
Over allDepth of Slab		1060.00	mm
Dia of Shear rft		20.00	mm
Area of One Bar		314.29	Sqmm
No of Bars		8.00	Nos
Max Shear Force wL/2		93.96	KN
Percentage of Tensile Steel 100At/2bd =		0.64	%
(at the end, alternate bar are bent up)			
Design Shear Strength		0.542	N/ Sqmm
Calculated k Value		1.00	
For 1060 mm thk slab, k=		1.00	
Permissible Max Shear Stress		0.542	N/ Sqmm
Nominal Shear stress Vu/bd		0.09	N/ Sqmm
Shear Check			Safe
Provide 10 mm dia 2 legged stirrup at 200 C/C to connect with stem			

10.1.2 Design of Cantilever Walls

A sample design format for Cantilever Wall Type 2; categorized as a wall with Scour Depth of 2.0 m, Height above Bed Level as 2.5 m, and Length of Panel as 10.0 m is presented below. The differentiation based on such categories has resulted in two dimensions of cantilever wall design.

PARAMETERS	Remarks	Values	Unit
WALL TYPE	Ramite Wall-2		
INPUT DATA			
Grade Of Concrete Fck	-	25	N/mm2
Grade Of Steel F Y		500	N/mm2
Angle Of Repose Of Soil Θ		30	DEG
Bulk Density Of Soil Ws		18	KN/m3
Soil Safe Bearing Capacity Sbc		380	KN/m2
Angle Of Surcharge Of Fill C		18	DEG
Coefft Of Friction µ		0.6	
Cos C		0.951	
Cos Θ		0.866	
Coefft Of Active Pressure Ka		0.395	0.334
Coefft Of Passive Pressure Kp		2.532	
Scour Depth		2.000	
Height Of Filling (Hf)		2.50	mtr
Const A		0.000	
Surcharge Ps	ps	0.00	KN/m
PRELIMINARY DIMENSIONS			
Min Depth Of Fdn H		2.35	mtr
Calculated Base Width B(Min)	Bm	3.10	mtr
Base Slab Thick D(Min)	460	370	mm
PROVIDED DIMENSIONS			
Total Ht Of Wall Ht=H+D		5.50	mtr
Length Of 1 Panel		10	m
Toe Length	TL	1.00	mtr
Heel Length	HL	1.65	mtr
Provide Base Width B	B	3.30	mtr
Provide Base Slab Thick D		550	mm
Wall Thick At Bottom T(Min)	550.0	550	mm
Provide Wall Thick At Bottom T1		650	mm
Provide Wall Thick At Top T2		300	mm
Provide Depth Of Fdn D	1	3.00	mtr
CHECK FOR BEARING PRESSURE			
Wt Of Base Slab/Footing	W1	45.38	KN/m
Wt Of Stem/ Wall Rectangle Part	W2	37.13	KN/m
Wt Of Stem/ Wall Triangle Part	W3	21.66	KN/m
Wt Of Rear Soil Over Heel	W4	162.94	KN/m
Wt Of Backfill At Toe	W5	112.82	KN/m
Total Stabilising Vertical Force	W	379.91	KN/m
Total Earth Pressure At Base	Ph	107.00	KN/m
Horizontal Surcharge	Ps	0.00	KN/m
Total Horizontal Pressure	V	107.00	KN/m
Distance Of W1 From Toe Front Tip X1	X1	1.65	mtr
Distance Of W2 From Toe Front Tip	X2	1.50	mtr
Distance Of W3 From Toe Front Tip	X3	1.23	mtr
Distance Of W3 From Toe Front Tip	X4	2.48	mtr
Distance Of W3 From Toe Front Tip	X5	0.50	mtr
Ht Of Hort Force Y1 From Toe Top	Y1	1.65	mtr
Ht Of Sur Force Y2 From Toe Top	Y2	2.48	mtr
Dist Of Σvert Reaction From Toe Front Tip		1.62	mtr
Overturning Moment(Mo)		195.7	KN-m
Sum Of Moments		421.3	KN-m
X Bar		1.1	mtr
Eccentricity		0.54	0.51612793
Safety Against Bearing			
Pressure At Toe Tip Pmax	0	228.39	KN/m2

Pressure At Heel Tip Pmax	0	1.86	KN/m2
Pressure At Toe Face Of Ver Stem		159.75	KN/m2
Pressure At Heel Face Of Ver Stem		115.12	KN/m2
FOS Against Overturning		2.84	SAFE
FOS Against Sliding		2.13	SAFE
DESIGN OF SHEAR KEY OR INCREASE WIDTH OF FDN			
Required For This Wall?	No Shear Key Required		-
If Required, Design Calculations:			-
Assumed Depth Of Shear Key(Ds)	0.4		
Passive Earth Pressure	404.4		Pp=Kp*P
Total Passive Earth Pressure	161.8		
Horizontal Earth Pressure	123.8		$Ph=KaYH^2/2$
Wt Of Soil Bw Base And Shear Key Base	23.8		
Sum Of Vertical Forces	403.7		
FOS Against Sliding	3.26418528	safe with shear key	
Reinforcement	480	mm2	
Provide Dia Of Steel Bar	16	mm	
Spacing Of Bars	219	mm	
Adopted Spacing	220	mm	
DESIGN OF BASE SLAB			
DESIGN OF TOE SLAB			
Effective Depth Of Toe D		475	mm
Span Of Toe L1		1.00	m
Wt Of Footing W4		13.75	KN/m
Weight Of Fill		32.37	
C.G Of Force		0.52	
Max Bm At Base Of Toe Mt		191.45	KN-m
Net Pressure Intensity Toe Tip		182.27	
Pressure At Toe Face Below Stem		146.00	
Shear Force Vmax		246.20	KN
DESIGN OF TOE SLAB			
DESIGN OF TOE SLAB TO RESIST BENDING MOMENT			
Grade of Concrete M		25	
Grade of Steel Fe		500	
Base width		1.0	Mtr
Max BM Mx		191.45	KN-M
BM = (Const*fck) bd^2		3.318	bd^2
Calculated Eff Depth of Slab		240	mm
Adopt Effective Depth d		470	mm
Use Dia of Slab rft		16	mm
Adopt Cover for Slab		75	mm
Over all Depth of Base Slab D		555	mm
Width of Slab considered for Cal		1000	mm
Grade of Concrete M		25	
Grade of Steel Fe		500	
a=	0.87 *(fy^2/fck)	8.7	
b=	-0.87 fy	-241425	
c=	m= Mu/(bd^2)	191450707.5	
m= Mu/(bd^2)		0.87	
area of steel= (-b- sqrt(b^2-4ac))/2a		817.060	
Min area of Tension Steel Ao=0.85*bd/fy		799	Sqmm
Min Area of Steel 0.15 % (Temperature Rft)		832.5	Sqmm
Max area of Tensile Steel = 0.04 bD		22200	Sqmm
Provide Area of Tension Steel		833	Sqmm
Area of One Bar		201.14	Sqmm
Spacing of Main Bars		240	mm
Adopted Spacing		240	
Min Area of Steel 0.12 %		666	Sqmm
Check for Min rft		OK	
Temp rft 0.15 % of gross area will be provided in the longitudinal direction		832.5	Sqmm

Use		16	mm Dia bars as distribution Rft
Area of One Bar		201.14	Sqmm
		16	mm Dia
Spacing of Distribution Bars		240	mm
Adopted Spacing		240	
DESIGN/ CHECK FOR TOE SLAB TO RESIST SHEAR			
Grade of Concrete M		25	
Effective Depth		470	mm
Over allDepth of Slab		555	mm
Dia of Shear rft		16	mm
Area of One Bar		201.14	Sqmm
Spacing of Bars		240	mm
Max Shear Force wL/2		246.20	KN
Percentage of Tensile Steel 100At/2bd = (at the end, alternate bar are bent up)		0.18	%
Design Shear Strength		0.314	N/ Sqmm
Calculated k Value		1.00	
For 555 mm thk slab, k=		1.00	
Permissible Max Shear Stress		0.314	N/ Sqmm
Nominal Shear stress Vu/bd		0.52	N/ Sqmm
Shear Check		Un safe	
Design of shear reinforcement			
Grade of Concrete M		25	
Grade of Steel Fe		500	
Effective Depth of Beam		470	mm
Over all Depth of Beam		555	mm
Width of Beam		1000	mm
Max Shear Force wl/2 Vu		246.20	KN
Strength of Shear rft Vus=Vu-Tc bd		98453	N
Dia of Shear rft		12	mm
Area of One Bar		113.14	Sqmm
No of legged vertical stirrups		4	Nos
Area of Vertical Stirrup Rft Asv		452.57	mm
Spacing of Shear rft x=0.87 fy Asv d/ Vus		940	mm
Check for Spacing		OK	
Min Area of Shear rft 0.4 b x /fy		240.00	Sqmm
Check for min Shear rft Area		OK	
DESIGN OF HEELSLAB			
Span Of Heel L2		1.65	M
Upward Soil Reaction		96.51	KN/m2
Upward Soil Reaction Acting At		0.56	M from heel stem base
Wt Of Soil Over Heel W5		162.94	KN/m
Wt Of Heel W6		22.69	KN/m
Max Bm At Heel Mh		148.83	KN-m
Shear Force Vmax		133.68	KN
Downward Weight Of Soil			
DESIGN OF HEEL SLAB TO RESIST BENDING MOMENT			
Grade of Concrete M		25	
Grade of Steel Fe		500	
Base width		1.0	Mtr
Max BM Mx		148.83	KN-M
BM = (Const*fck) bd^2		3.318	bd^2
Calculated Eff Depth of Slab		212	mm
Adopt Effective Depth d		470	mm
Use Dia of Slab rft		16	mm
Adopt Cover for Slab		75	mm
Over all Depth of Base Slab D		555	mm
Width of Slab considered for Cal		1000	mm
Grade of Concrete M		25	
Grade of Steel Fe		500	
a=	0.87 *(fy^2/fck)	8700	
b=	-0.87 fy	-435	

c=	m= Mu/(bd^2)	0.67	
m= Mu/(bd^2)		0.67	
p %= (-b- sqrt(b^2-4ac))/2a		0.160	
At		753	Sqmm
Min area of Tension Steel Ao=0.85*bd/fy		799	Sqmm
Min Area of Steel 0.15 % (Temp Rft)		832.5	Sqmm
Max area of Tensile Steel = 0.04 bD		22200	Sqmm
Provide Area of Tension Steel		833	Sqmm
Area of One Bar		201.14	Sqmm
Spacing of Main Bars		240	mm
Adopted Spacing		240	
Min Area of Steel 0.12 %		666	Sqmm
Check for Min rft		OK	
Temp rft 0.15 % of gross area will be provided in the longitudinal direction		832.5	Sqmm
Use		16	mm Dia bars as distribution Rft
Area of One Bar		201.14	Sqmm
		16	mm Dia
Spacing of Distribution Bars		240	mm
Adopted Spacing		240	
DESIGN/ CHECK FOR HEEL SLAB TO RESIST SHEAR			
Grade of Concrete M		25	
Effective Depth		470	mm
Over allDepth of Slab		555	mm
Dia of Shear rft		16	mm
Area of One Bar		201.14	Sqmm
Spacing of Bars		240	mm
Max Shear Force wL/2		133.68	KN
Percentage of Tensile Steel 100At/2bd =		0.18	%
(at the end, alternate bar are bent up)			
Design Shear Strength		0.31	N/ Sqmm
Calculated k Value		1.00	
For 555mm thk slab, k=		1.00	
Permissible Max Shear Stress		0.31	N/ Sqmm
Nominal Shear stress Vu/bd		0.28	N/ Sqmm
Shear Check		Safe	
Design of shear reinforcement			
Grade of Concrete M		25	
Grade of Steel Fe		500	
Effective Depth of Beam		470	mm
Over all Depth of Beam		555	mm
Width of Beam		1000	mm
Max Shear Force wl/2 Vu		133.68	KN
Strength of Shear rft Vus=Vu-Tc bd		0	N
Dia of Shear rft		12	mm
Area of One Bar		113.14	Sqmm
No of legged vertical stirrups		4	Nos
Area of Vertical Stirrup Rft Asv		452.57	mm
Spacing of Shear rft x=0.87 fy Asv d/ Vus		0	mm
Check for Spacing		NOT OK	
Min Area of Shear rft 0.4 b x /fy		0.00	Sqmm
Check for min Shear rft Area		NOT OK	
DESIGN OF STEM WALL			
Span Of Wall L3		4.95	M
Max Bm At Bottom Of Wall Mw		293.51	KN-m
Shear Force Vmax		107.00	KN
DESIGN OF STEM WALL TO RESIST BENDING MOMENT			
Grade of Concrete M		25	
Grade of Steel Fe		500	
Base width		1.0	Mtr
Max BM Mx		293.51	KN-M
BM = (Const*fck) bd^2		3.318	bd^2
Calculated Eff Depth of Slab		297	mm

Adopt Effective Depth d		560	mm
Use Dia of Stem Wall rft		16	mm
Adopt Cover for Stem wall		75	mm
Over all Depth of Base Slab D		645	mm
Width of Slab considered for Cal		1000	mm
Grade of Concrete M		25	
Grade of Steel Fe		500	
a=	0.87 *(fy^2/fck)	8700	-243600
b=	-0.87 fy	-435	8.7
c=	m= Mu/(bd^2)	0.94	293512914
m= Mu/(bd^2)		0.94	
p %= (-b- sqrt(b^2-4ac))/2a		1261.755	
At		1262	Sqmm
Min area of Tension Steel Ao=0.85*bd/fy		952	Sqmm
Min Area of Steel 0.15 % (Temp Rft)		967.5	Sqmm
Max area of Tensile Steel = 0.04 bD		25800	Sqmm
Provide Area of Tension Steel		1262	Sqmm
Area of One Bar		201.14	Sqmm
Spacing of Main Bars		150	mm
Adopted		150	
Min Area of Steel 0.12 %		774	Sqmm
Check for Min rft		OK	
Temp rft 0.15 % of gross area will be provided in the longitudinal direction		967.5	Sqmm
Use		16	mm Dia bars as distribution Rft
Area of One Bar		201.14	Sqmm
		16	mm Dia
Temperature reinforcement		200	mm
Adopted Spacing		200	
CURTAILMENT REINFORCEMENT			
Depth From Top Of Embankment		2.75	Mtr
Area Of Reinforcement		630.88	Sqmm
Use Dia Of Stem Wall Rft		16	mm
Area Of One Bar		201.14	Sqmm
Spacing Of Distribution Bars		300	mm
Adopted Spacing		300	
DISTRIBUTION STEEL			
Depth From Top Of Embankment		2.75	Mtr
Area Of Reinforcement		967.5	Sqmm
Use Dia Of Stem Wall Rft		16	mm
Area Of One Bar		201.14	Sqmm
Spacing Of Distribution Bars		200	mm
Adopted Spacing		200	
DESIGN/ CHECK FOR STEM WALL TO RESIST SHEAR			
Grade of Concrete M		25	
Effective Depth		560	mm
Over allDepth of Slab		645	mm
Dia of Shear rft		16	mm
Area of One Bar		201.14	Sqmm
Spacing of Bars		150	mm
Max Shear Force wL/2		107.00	KN
Percentage of Tensile Steel 100At/2bd =		0.24	%
(at the end, alternate bar are bent up)			
Design Shear Strength		0.358	N/ Sqmm
Calculated k Value		1.00	
For 645 mm thick slab, k=		1.00	
Permissible Max Shear Stress		0.358	N/ Sqmm
Nominal Shear stress Vu/bd		0.19	N/ Sqmm
Shear Check		Safe	

10.1.3 Concrete Check Dams

There are 9 check dams proposed for construction. They are located across the stream, primarily with the consideration of change in slope as well as accessibility of the dredging rams. The total sediment control is around 1,470 m^3 of debris.

Centerline Chainage	Name	Height (m)	Backflow length (m)	Volume (m3)
310	Check Dam 1	2	150	336.13
510	Check Dam 2	1.5	50	80.01
760	Check Dam 3	2	30	115.65
850	Check Dam 4	2	50	94.62
1200	Check Dam 5	2	80	153.89
1390	Check Dam 6	2	110	166.71
1780	Check Dam 7	2	100	199.18
2280	Check Dam 8	2	60	149.80
3290	Check Dam 9	2	50	174.86
			Total (m^3)	1,470.87

The design consideration has been provided as Annex 5. The location of the check dams is provided in Figure below. At the end of the check dam (downstream) would be the apron to reduce the impact of water velocity.

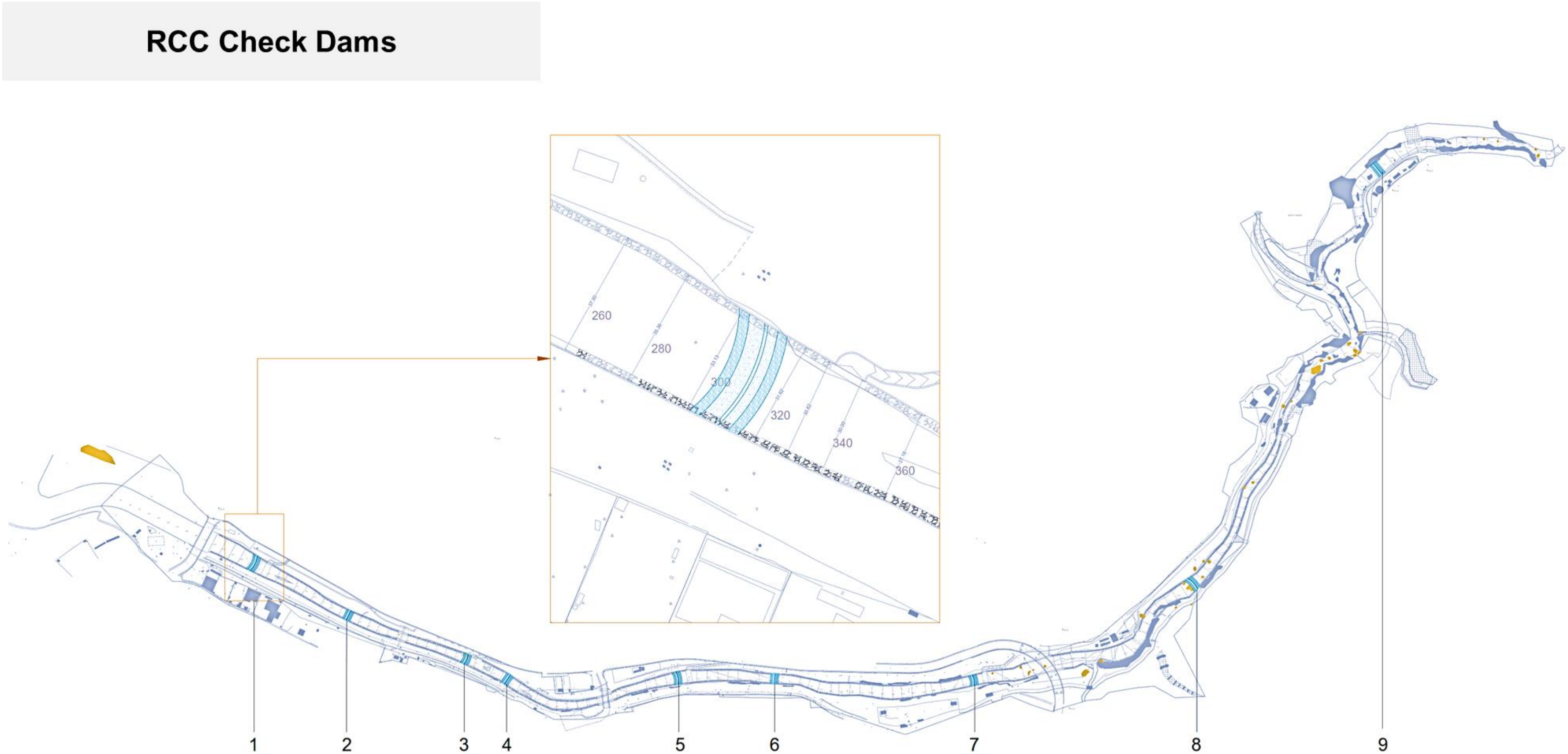
Data	Value	Units	Check
Design Discharge, Q	660	m^3/s	
Location of dam (Chainage)	3290.00	m	
RL of River bed level	271.00	m	
Average RL of Bank top level	276.5	m	
Bed slope	3.4	%	
Flow velocity	7.45	m/s	
Scour Depth, R	2.7	m	
Length of dam, L	27.83	m	
SBC	200.00	kN/m^2	
Geometric properties			
Depth of flow over crest, dc	0.90	m	
Head or Height of check dam from u/s, H	2	m	
RL of Dam top	273.00	m	
Top width of dam, B_t	1	m	
U/s face slope	0:01		
D/s face slope	0.6:1		
Bottom width of dam, B	**2.40**	m	
Stability check			
Unit weight of water, y_w	10	kN/m^3	
Unit weight of soil, y_s	18	kN/m^3	
Unit weight of water, y_c	25	kN/m^3	
Weight of the check dam per unit length			
W1=	50	kN/m	
W2=	40	kN/m	

Total weight, W=	**90**	kN/m	
Stabilizing moment, Mst per unit legnth about toe			
Mst1 by W1	95	kNm/m	
Mst2 by W2	37.33	kNm/m	
Total moment, Mst=	**132.33**	kNm/m	
Overturning moment, Mo per unit legnth about toe			
Water pressure, P_1=	**20.00**	kNm/m	
Sedimentary pressure, P_2=	**10.89**	kNm/m	
Mo=	**17.33**	kNm/m	
FOS against overturning, η_o=	7.64	OK	>1.5
Sliding check			
Friction coefficeint,µ	0.65		
Reisting force, Fres=W*µ	58.5	kN/m	
Driving force, Fdiv=P	30.89	kN/m	
FOS against sliding, η_s=	1.89	OK	>1.5
Resultant force line			
Total moment=	115.01	kNm	
Total weight above GL=	90	kN	
Resultant from toe, x=	1.28	m	OK
Eccentricity, e=B/2-x	0.02	m	
Pressure below base=∑W/B*(1 ±6e/B)			
Max. pressssure, Pmax	161.70	kN/m^2	Safe
Min. pressure, Pmin	154.72	kN/m^2	OK
Design of apron			
Bligh's coefficient, C for gravel sub-soil	5		
Total Length of appron required, L_T=C*H	**10**	m	
Length of D/s impervious appron, L_2			
L_2=0.696*C*Sqrt (H)	**4.92**	m	
Unit discharge, q=1.84(d_c)^3/2	1.57	m^3/s	
Length of D/s impervious Apron including boulder pitching or Talus, L_3			
L_3=1.2*C*sqrt (H*q)	**10.64**	m	
Length of D/s boulder pitching or Talus, $L_4=L_3-L_2$	**5.40**		
Length of U/s impervious Apron, $L_1=L_T-(L_2-B)$	**3.00**	m	
Total length od apron provided=L_1+B+L_2	10.00	m	
Regime width (Larcey's), P			
P=Sqrt(4.83*Q)	56.46	m	
Looseness Factor (LF)=Overall length of check dam, L/Regime width, P			
LF=L/P	0.49		
Scour Depth			
Larcey's scour depth			
Nominal U/s Scour depth, R=1.35*(q^2/f)^1/3			
Take Larcey's silt factor, f	1		
R=	1.82	m	
Maximum scour =1.5R	**2.7**	m	
Based on field data			
Depth of U/s cut off =1.2R	3	m	
Depth of D/s cut off =1.5R	4	m	

Assumed Data			
Provide cut off thickness, t	0.35	m	
Provide floor thickness	0.35	m	
Note:			

1) Since the length of the Check dam is at par with the actual river flow width from the feld, the discharge and velocity data obtained from field is not applied in deteming the legnth of caheck dam.
2) The check dam spacing is also maintained as provided in layout plan.
3) Top width of check dam is taken as 1m (Bligh suggests rang of 0.9 to 1.2m)
4) Botom width is calculated from assumed D/s face slope of 0.6:1.
5) Chedck dam designed as gravity dam to full hydrostatic pressure plus sedimentary soil or mud flow pressure.
6) Water pressure from D/s is neglected as the amount is negligble or very small.
7) Uplift is also neglected as it small as compared to the weight of the dam and diminishes on sides towards top.
8) Foundation depth considered at to the depth of scour.

Figure 121: Location of RCC Check Dams



10.1.4 Culvert Design

10.1.4.1 Introduction

For the project titled, "Revision of Detailed Project Report for OMCHU", under Phuentsholing Thromde, there are two slab bridges of span and carriageway width as shown in Table 1 below;

Table 46: Bridge span and carriageway width

SN.	Span (m)	Carriageway width (m)	Remarks
1	10.00	5.50	1 No.
2	8.00	5.50	1 No.

The detail geometry for two bridges, i.e., span of 10.00 m and 8.00 m are shown in Table below respectively;

Table 47: Detail geometry of 10.00 m span slab bridge

SN	Description	Dimension (m)	Remarks
1	Center to Center span, L	10.00	
2	Bridge length	10.50	End to end
3	Depth of slab (edge)	0.52	
4	Depth of slab (Centre)	0.60	Future resurfacing work
5	Wearing course thickness	0.10	
6	Kerb beam height	0.35	Both edge
7	Depth of slab with kerb beam	0.87	At the two edges
8	Width of kerb beam	0.35	Both edge
9	Carriageway width	5.50	
10	Total width of bridge	6.20	

Table 48: Detail geometry of 8.00 m span slab bridge

SN	Description	Dimension (m)	Remarks
1	Center to Center span, L	8.00	
2	Bridge length	8.50	End to end
3	Depth of slab (edge)	0.44	
4	Depth of slab (Centre)	0.52	Future resurfacing work
5	Wearing course thickness	0.10	
6	Kerb beam height	0.35	Both edge
7	Depth of slab with kerb beam	0.79	At the two edges
8	Width of kerb beam	0.35	Both edge
9	Carriageway width	5.50	
10	Total width of bridge	6.20	

For the two slab bridges as mentioned above, the height of abutments is determined as shown in Table below;

Table 49: Abutment Details

SN.	Span (m)	Footing dimension (m)			Abutment stem dimension (m)			
		Length	Width	Thickness	Thickness (bottom)	Thickness (top)	Width	Height
1	10.00	3.60	6.20	0.80	0.80	0.60	6.20	5.20

2	8.00	4.00	6.20	0.80	1.00	0.60	6.20	5.70

10.1.4.2 Bridge Load of Superstructure (slab)

Dead Load

The primary load (dead load) on the bridge of spans 10.00 m and 8.00 m are shown in Table below;

Table 50: Dead load for 10.00 m span bridge

SN.	Load Description	Load (KN/m)	Remarks
1	Self-weight of deck	89.99	D: Dead load
2	Wearing course load	12.65	DW: Dead load of wearing surface and utilities
3	Barrier/railing	7.20	DC: Dead load of component and attachments

Table 51: Dead load for 8.00 m span bridge

SN.	Load Description	Load (KN/m)	Remarks
1	Self-weight of deck	78.08	D: Dead load
2	Wearing course load	12.65	DW: Dead load of wearing surface and utilities
3	Barrier/railing	7.20	DC: Dead load of component and attachments

10.1.4.3 Live Load (Vehicle Load/Moving Load)

For the carriageway width of bridge between 5.3 m and 6.1 m, the bridge can be designed for vehicular load of IRC-6 Class A (equivalent to 40R) of two lanes or single lane of IRC-6 Class 70R (wheeled). However, the two bridges under the project are designed for two lanes of class 40R as per the requirement of client.

10.1.4.4 Bridge load of Substructure (abutment)

In addition to self-weight of abutment (depend on geometry) and earth pressure load (vertical direction and horizontal direction), following load from superstructure are considered in analysis and for the design as shown in Table below for bridge span of 10.00 m and 8.00 m respectively;

Table 52: Superstructure load of 10.00 m span on its abutment design

SN.	Load case/description	Load value (KN)	Remarks
1	Self-weight	449.93	Vertically downward
2	Wearing course	63.25	
3	Barrier/railing	36.00	
4	Live load/moving load	516.39	

Table 53: Superstructure load of 8.00 m span on its abutment design

SN.	Load case/description	Load value (KN)	Remarks
1	Self-weight	312.33	Vertically downward
2	Wearing course	50.60	
3	Barrier/railing	28.80	
4	Live load/moving load	437.10	

10.1.4.5 Analysis Tool

The relevant bridge design software's were used for analysis of both superstructure and substructure. In addition to the software's used as stated above, MS Excel worksheets are extensively used for specific calculation. The Figure as shown below are model prepared for analysis of the superstructure and substructure respectively using Midas CIVIL.

Figure 122: Model of 8.00 m span superstructure (slab)

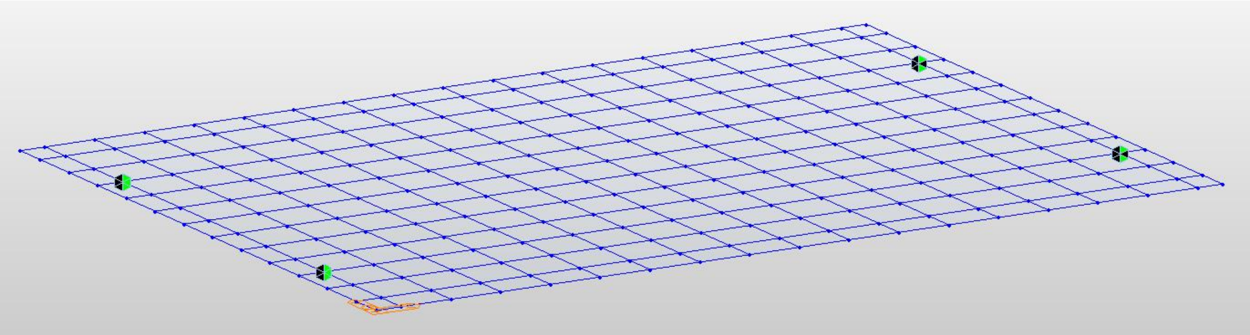
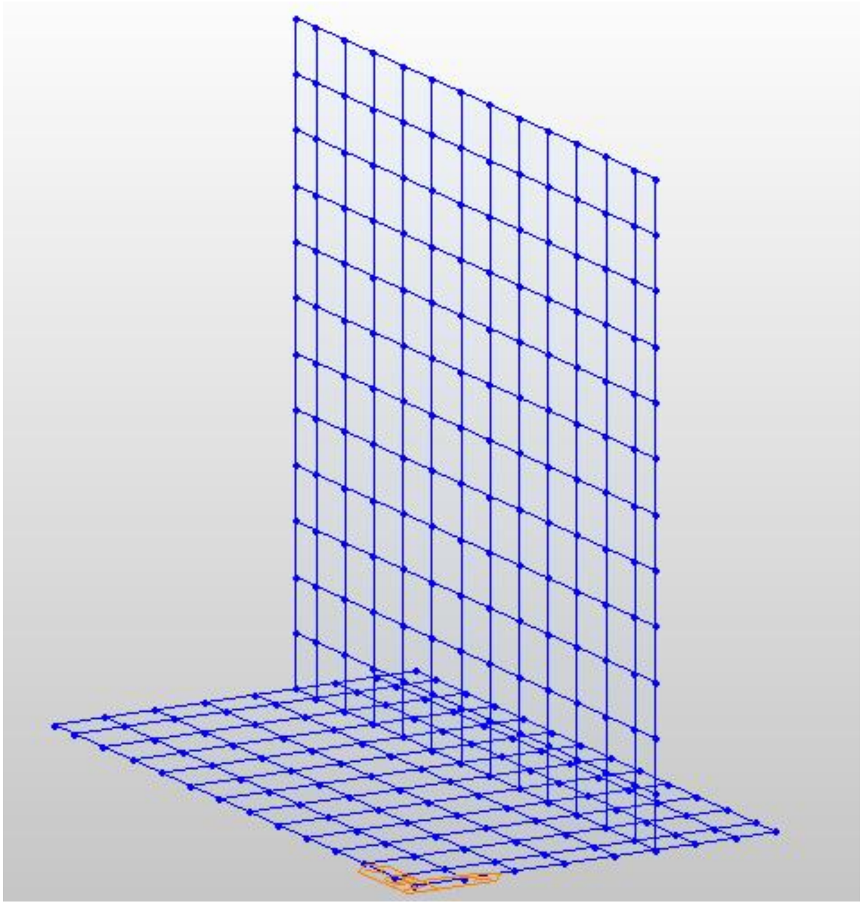


Figure 123: Model of 6.00 m height substructure (abutment)



For the bridge span of 10.00 m and abutment height of 6.5 m also, the models were as same as those shown in Figure above. Following the provision of software, the models for both superstructure and substructure are prepared as plate models.

In the model prepared for analysis, the material properties and unit weight of different materials used are as shown in Table below.

Table 54: Material properties and unit weights:

SN.	Bridge Component	Material properties	Remarks
1	Superstructure (slab)	M30	Concrete grade
2	Substructure (abutment)	M25	Concrete grade
3	Unit weight of concrete	24.00 KN/m3	
4	Unit weight of asphalt (wearing course)	23.00 KN/m3	
5	Unit weight of soil	18.00 KN/m3	
6	Angle of internal friction of soil (Φ)	30O	
7	Allowable bearing capacity (assumed)	300 KN/m2	

10.1.4.6 Analysis Results or the Design Data.

The Table below shows the analysis results of both the span of bridge which are maximum moment for both in strength and in serviceability.

Table 55: Maximum moment for bridge superstructure

SN.	Span (m)	Carriageway width (m)	Moment (KN-m)	
			Strength	Service
1	8	5.5	2310.20	1753.30
2	10	5.5	3790.10	2884.60

The Table below are analysis results for abutment with heights 6.00 m and 6.50 m.

Table 56: Maximum moment for 6.00 m height abutment

SN.	Component	Length (m)	Width (m)	Moment (KN-m)	Position
1	Footing slab	3.60	6.20	82.43	Longitudinal
				32.71	Transverse
2	Abutment stem	0.80	6.2	993.68	Vertical
				198.74	Horizontal

Table 57: Maximum moment for 6.50 m height abutment

SN.	Component	Length (m)	Width (m)	Moment (KN-m)	Position
1	Footing slab	4.00	6.20	90.63	Longitudinal
				31.13	Transverse
2	Abutment stem	1.00	6.2	913.92	Vertical
				182.78	Horizontal

10.1.4.7 Reinforcement Design

Based on the analysis results determined as shown from Table above, the reinforcement for different components were designed. The amount of reinforcement depends on analysis results of different geometries of the bridge components. The reinforcement designs of bridge superstructure or slab of both the spans are shown in Table below;

Table 58: Reinforcement design of bridge superstructure (slab)

SN.	Span (m)	Carriageway width (m)	Reinforcement	Position
1	8	5.5	T20@100	Bottom longitudinal
			T12@200	Top longitudinal
			T12@100	Bottom transverse
			T12@200	Top transverse
2	10	5.5	T25@125	Bottom longitudinal
			T12@200	Top longitudinal
			T16@125	Bottom transverse
			T12@200	Top transverse

Similarly for substructure (abutment) of the bridges, the Table below shows the reinforcement designs of abutment height with 6.00 m and 6.50 m respectively.

Table 59: Reinforcement design of abutment (height = 6.00 m)

SN.	Component	Rebar	Position
1	Footing slab	T16@200	Longitudinal direction (top & bottom)
		T12@110	Transverse direction (top & bottom)
2	Abutment stem	T25@140	Main bar (backside)
		T16@140	Main bar (riverside)
		T12@150	Distribution bar (backside & river side)

Table 60: Reinforcement design of abutment (height = 6.50 m)

SN.	Component	Rebar	Position
1	Footing slab	T16@200	Longitudinal direction (top & bottom)
		T12@110	Transverse direction (top & bottom)
2	Abutment stem	T20@120	Main bar (backside)
		T16@120	Main bar (riverside)
		T16@150	Distribution bar (backside)
		T12@150	Distribution bar river side)

10.1.4.8 Stability check of abutments

The abutment should be stable against all the load and forces acting on it from superstructure and earth pressure. For the design of abutment, safety check against sliding failure and base failure are mandatory. If these two checks are passed, safety against overturning is determined safe always.

10.1.4.9 Sliding failure

For the stability check against sliding failure, the horizontal and vertical forces acting in footing slab of abutment are required and safety is checked using following formula;

$$Factor\ of safety\ = \frac{\mu F_V}{F_H}$$

Where

μ	:	Coefficient of friction (0.55 considering coarse grained)
FV	:	Total vertical force
FH	:	Total horizontal force

When the factor of safety is determined greater than 1.5 as per IRC: 112 – 2011 and IS:456, then the structure is considered safe against sliding. The Table below shows forces and safety factors for two abutments.

Table 61: Safety factor check against sliding

SN.	Abutment Height (m)	Forces (KN)		Factor of safety	Remarks
1	6.00	Horizontal force (FH)	1173.96	2.34	Safe against sliding
		Vertical force (FV)	4995.83		
2	6.50	Horizontal force (FH)	1147.26	2.47	Safe against sliding
		Vertical force (FV)	5144.35		

10.1.4.10 Safety against Base Failure

The safety against base failure is checked by comparing allowable bearing capacity assumed and maximum reaction in the mesh of the plate element of the footing slab. When maximum reaction is divided by mesh size (area of mesh) and if the value is less than allowable bearing capacity of ground, the structure can be safely constructed. The Table below show safety check against base failure of the abutments.

Table 62: Safety check against base failure

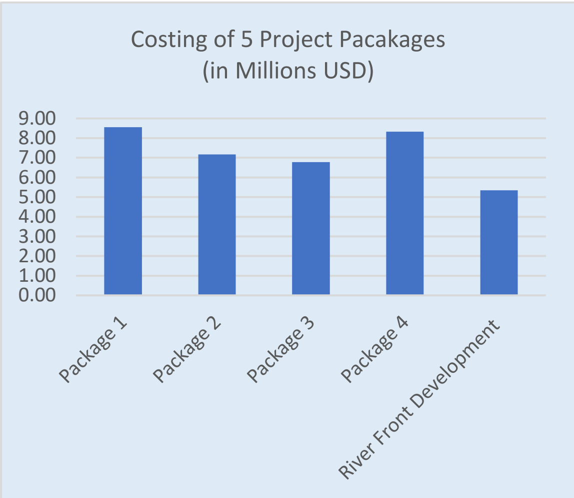
SN.	Abutment Height (m)	Mesh Size of footing	Maximum reaction of mesh	Bearing capacity	Remarks
1	6.00	0.5 x 0.5	39.99	159.96	Less than 300 KN/m2; Hence SAFE
2	6.50	0.5 x 0.5	41.19	164.76	Less than 300 KN/m2; Hence SAFE

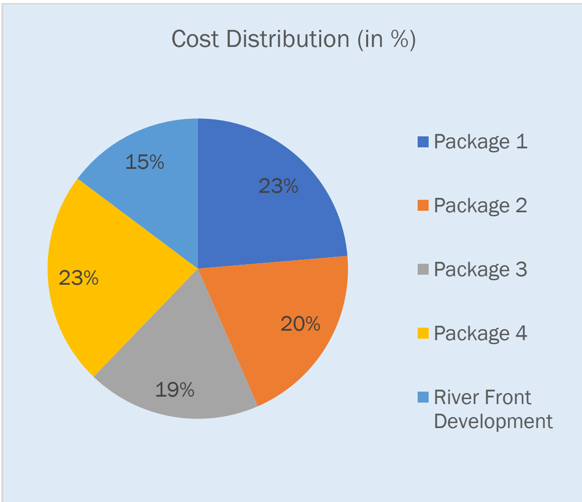
11 Project Cost

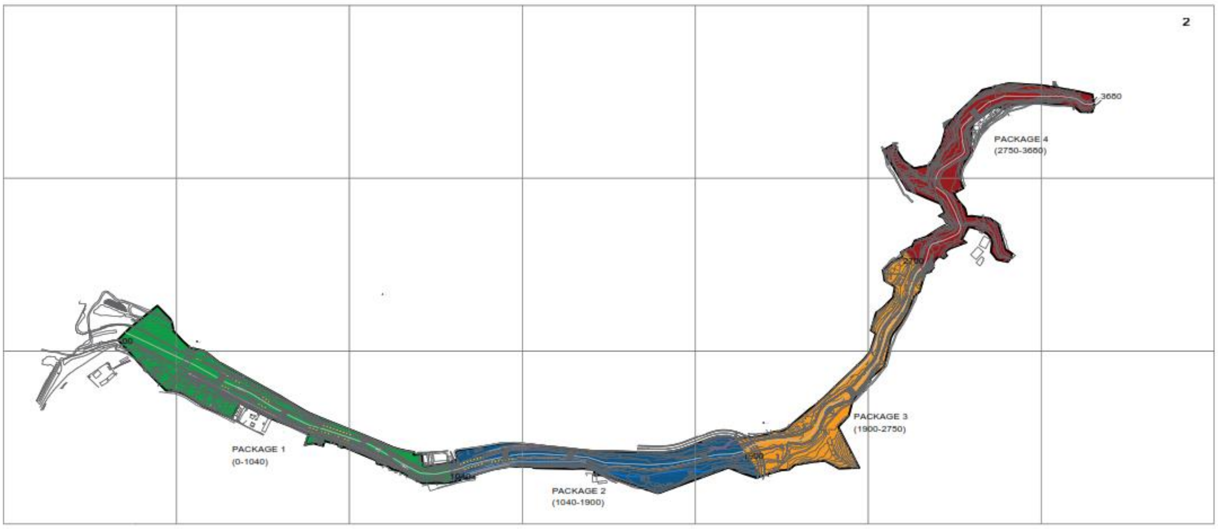
The Project Cost has been prepared based on the Bhutan Schedule of Rates 2022, and where necessary based on the analyzed rates from the market. The rate of conversion for Ngultrums to US Dollars is 1 USD= 80 Nu. based on the current rates. There are four packages of construction, and envisaged to be completed in a phased manner in the event of budget constraints. The packages are divided as follows;

6. Package 1: Omchhu Bridge 2 To RSTA Bridge
7. Package 2: RSTA Bridge to Curvilinear Bridge
8. Package 3: Curvilinear Bridge to Upstream of Bailey Bridge (Ch-2750)
9. Package 4: Upstream of Bailey Bridge (Ch-2750) To Water Treatment Plant

The packages are of similar financial value and of the distance coverage. It must be realized that the execution of the entire stretch at one go may not be practical since ideal construction period would only be from October till April (6-7 months). Therefore, even the packages themselves might have to be sub-contracted and such provisions should be thought about by the client. The landscape development has been considered an independent package, and while the overall masterplan shall become the guiding principle, it is subject to change based on the final land profile and availability after construction of the flood defense mechanism.





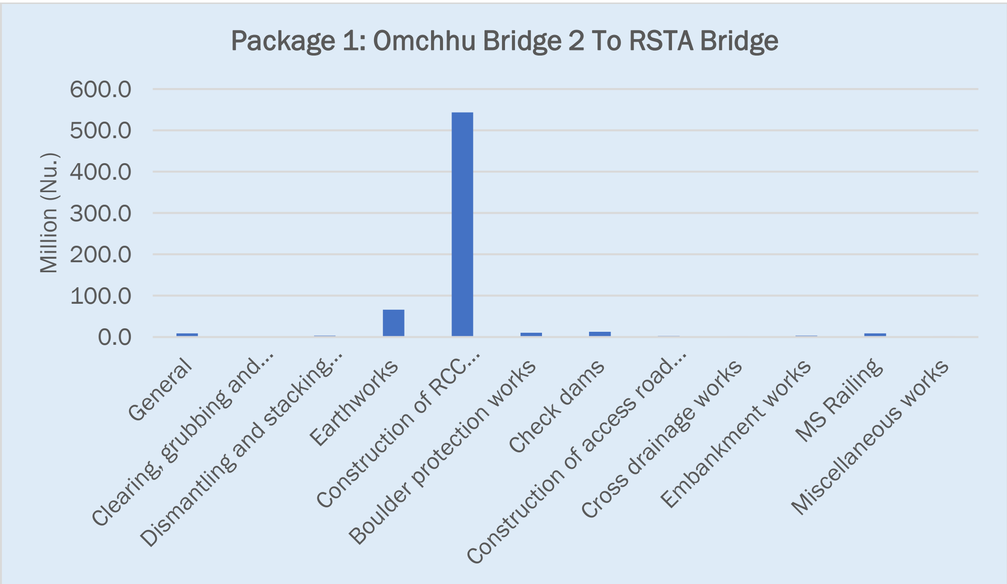


The -costing is presented in the format of 5 components which includes the 4 packages and the Landscape component. This amounts to a total of USD 36.17 Million. The 4 packages alone constitute USD 30.83 Million. The major cost across all packages is for the reinforced wall construction which accounts to flood defense structures which accounts to USD 24.8 Million, while the earthwork component in these packages constitute another USD 2.69 Million.

While the Project Cost Estimate Document (Standalone document) contains the specifics of the cost estimates, this section briefly describes distribution package wise.

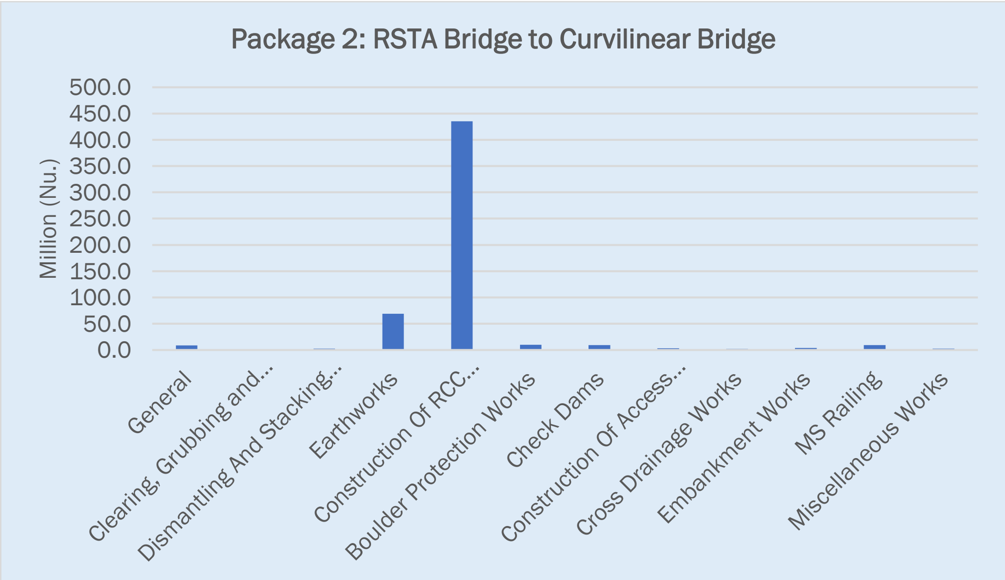
Package 1: Omchhu Bridge 2 To RSTA Bridge

Section	Item	Amount (Nu. Million)
Section 1	General	8.6
Section 2	Clearing, grubbing and removals	0.1
Section 3	Dismantling and stacking for reuse	3.2
Section 4	Earthworks	66.2
Section 5	Construction of RCC Counterfort walls	543.8
Section 6	Boulder protection works	10.7
Section 7	Check dams	13.0
Section 8	Construction of access road and ramps	2.3
Section 9	Cross drainage works	2.2
Section 10	Embankment works	3.5
Section 11	MS Railing	9.0
Section 12	Miscellaneous works	2.0
TOTAL (Nu)		664.55
CONTINGENCIES 3%		19.94
GRAND TOTAL - MITIGATION INTERVENTION (Nu)		684.49
GRAND TOTAL - MITIGATION INTERVENTION (USD)		**8.56**



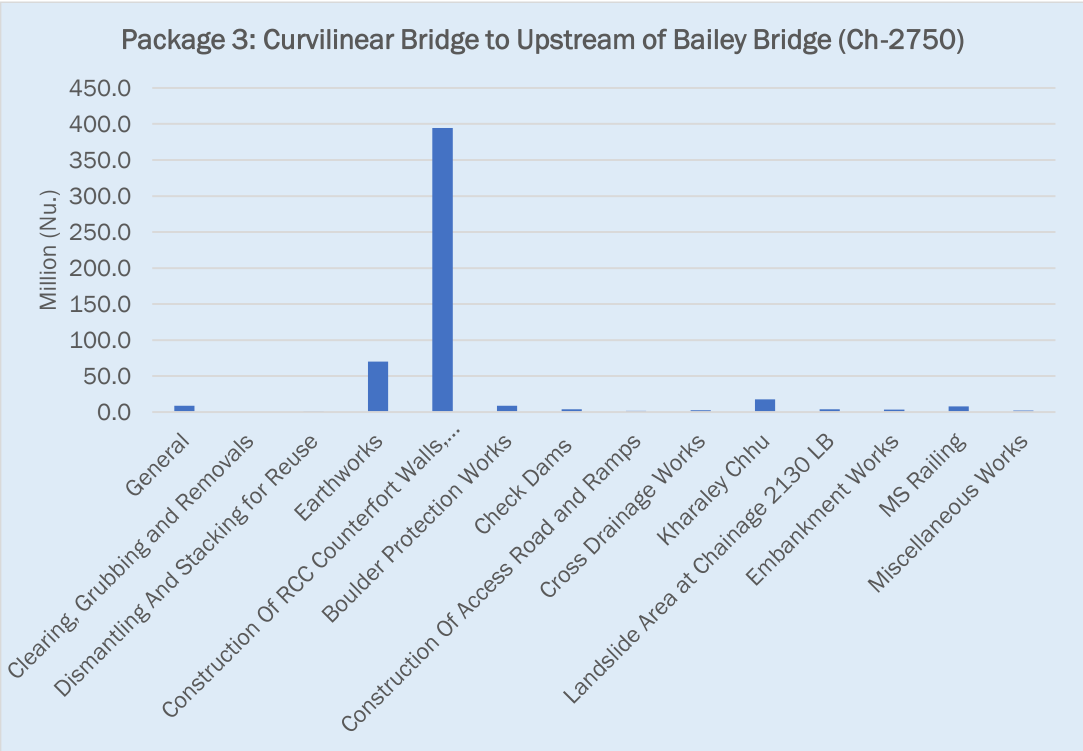
PACKAGE 2: RSTA Bridge to Curvilinear Bridge

Section	Item	Amount (Nu. Million)
Section 1	General	8.9
Section 2	Clearing, Grubbing and Removals	0.1
Section 3	Dismantling And Stacking for Reuse	2.7
Section 4	Earthworks	69.2
Section 5	Construction Of RCC Counterfort Walls	435.3
Section 6	Boulder Protection Works	10.2
Section 7	Check Dams	9.3
Section 8	Construction Of Access Road and Rams	3.1
Section 9	Cross Drainage Works	2.1
Section 10	Embankment Works	3.5
Section 11	MS Railing	9.2
Section 12	Miscellaneous Works	2.8
TOTAL (Nu)		556.32
CONTINGENCIES 3%		16.69
GRAND TOTAL - MITIGATION INTERVENTION (Nu)		573.01
GRAND TOTAL - MITIGATION INTERVENTION (USD)		7.16



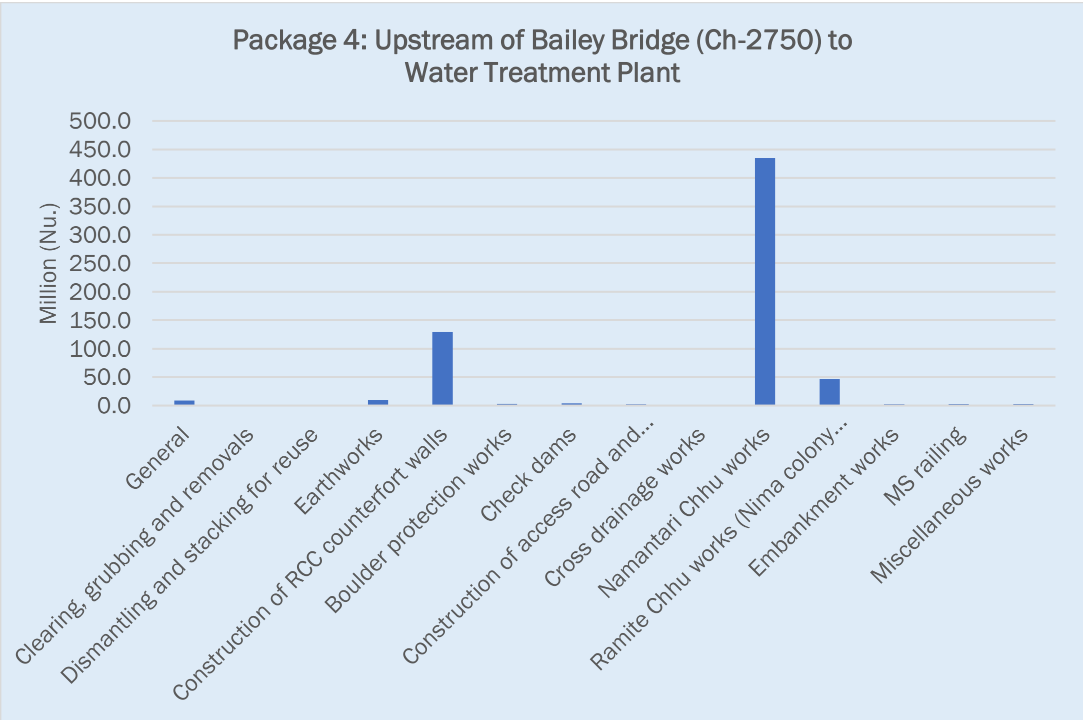
PACKAGE 3: Curvilinear Bridge to Upstream of Bailey Bridge (Ch-2750)

Section	Item	Amount (Nu. Million)
Section 1	General	8.9
Section 2	Clearing, Grubbing and Removals	0.1
Section 3	Dismantling And Stacking for Reuse	1.0
Section 4	Earthworks	69.9
Section 5	Construction Of RCC Counterfort Walls, Package 3	394.3
Section 6	Boulder Protection Works	8.9
Section 7	Check Dams	3.9
Section 8	Construction Of Access Road and Ramps	1.9
Section 9	Cross Drainage Works	2.5
Section 10	Kharaley Chhu	17.5
Section 11	Landslide Area at Chainage 2130 LB	4.0
Section 12	Embankment Works	3.5
Section 13	MS Railing	8.0
Section 14	Miscellaneous Works	2.3
TOTAL (Nu)		526.73
CONTINGENCIES 3%		15.80
GRAND TOTAL - MITIGATION INTERVENTION (Nu)		542.53
GRAND TOTAL - MITIGATION INTERVENTION (USD)		6.78



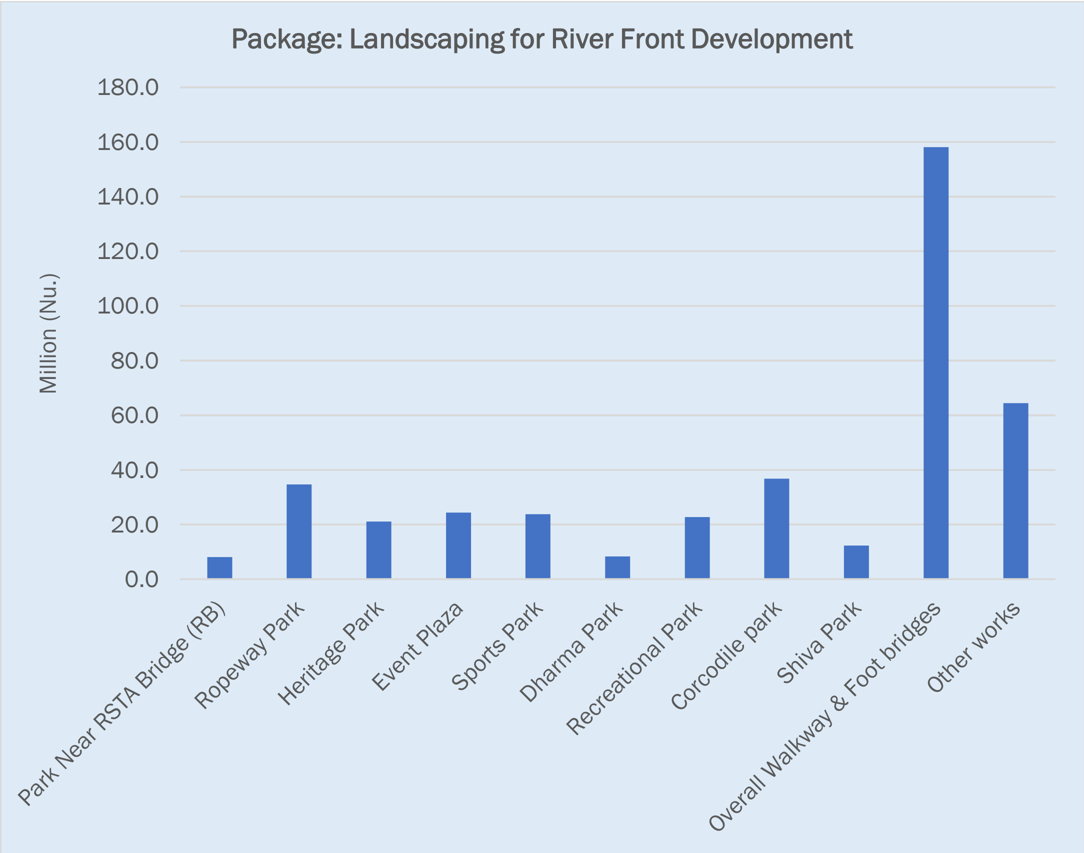
Package 4: Upstream of Bailey Bridge (Ch-2750) To Water Treatment Plant

Section	Item	Amount (Nu. Million)
Section 1	General	8.9
Section 2	Clearing, grubbing and removals	0.0
Section 3	Dismantling and stacking for reuse	0.5
Section 4	Earthworks	9.8
Section 5	Construction of RCC counterfort walls	129.3
Section 6	Boulder protection works	2.9
Section 7	Check dams	4.0
Section 8	Construction of access road and rams	1.9
Section 9	Cross drainage works	1.1
Section 10	Namantari Chhu works	435.0
Section 11	Ramite Chhu works (Nima colony area)	46.4
Section 12	Embankment works	2.1
Section 13	MS railing	2.5
Section 14	Miscellaneous works	2.9
TOTAL (Nu)		647.20
CONTINGENCIES 3%		19.42
GRAND TOTAL - MITIGATION INTERVENTION (Nu)		666.62
GRAND TOTAL - MITIGATION INTERVENTION (USD)		8.33



Package 5: Landscaping

Section	Item	Amount (Nu. Million)
Section 1	Park Near RSTA Bridge (RB)	8.1
Section 2	Ropeway Park	34.7
Section 3	Heritage Park	21.1
Section 4	Event Plaza	24.3
Section 5	Sports Park	23.8
Section 6	Dharma Park	8.4
Section 7	Recreational Park	22.7
Section 8	Corcodile park	36.8
Section 9	Shiva Park	12.3
Section 10	Overall Walkway & Foot bridges	158.1
Section 11	Other works	64.5
TOTAL (Nu)		414.7
CONTINGENCIES 3%		12.4
GRAND TOTAL - MITIGATION INTERVENTION (Nu)		427.1
GRAND TOTAL - MITIGATION INTERVENTION (USD)		5.339



12 Landscape Development: Omchhu Riverfront Development

The complete landscape designs are provided as an independent document. This section contains the overall approach, literature review, site analysis, and snapshots of the proposed plans only. Therefore, it is not exhaustive in terms of the contents, for which the main Landscape Design and Drawings document shall be referred.

12.1 The Riverfront as Socio Interactive Breathing Space: An Overview

"A riverfront is a significant resource and a challenging opportunity for a city; a chance to be an escape valve for the pressure-cooker of crowded city life, a chance to be a bright, breathing edge of city living and can be achieved by providing meaningful interactive spaces." Arthur Cotton Moor

Riverfronts are public spaces. The residents of the city and visitors alike can use them for a variety of activities, like a festive gathering place, a variety of recreation activity (active and passive) or for some mixed use, be it residential, retail, a city landmark etc. They could be used as for cycling, jogging or other activities that require open areas. It can also be utilized for some passive activities like fishing and boating. These activities will serve a threefold purpose – provide recreation, create a healthier environment that promotes people to stay fit and healthy and boost the economic opportunities of the city. The rejuvenated and re-energized riverfronts will attract more visitors both local and otherwise. Cities throughout the world have developed them as destinations to expand their tourism industry.

Riverfronts in Bhutan have never been actively exploited for their enormous potential. The residents of Phuentsholing have few public open spaces. The Omchhu in Phuentsholing has the potential to be of greater utility and value. The Omchhu riverfront has sufficient physical, visual and emotional appeal and the development of greenways, gardens and play areas can contribute to the quality of daily life in all of all aspects – social, economic, cultural and ecological. The objective of this project is to transform the Omchhu riverfront into a socio interactive space equipped with certain facilities for entertainment and recreation to serve the city, its residents and all visitors.

12.2 Methodology

The methodology applied for this design assignment is principally a four tiered approach viz. a) Initial library/literature review, b) a detailed analysis of the site and the overall context of the site with regard to Phuentsholing city c) Concept designs and options presented for discussions and development; feedback from stakeholders d) Approval of final design and preparation of final drawings and report.

The first stage in the assignment relied on library and in-depth case studies. A multitude of case studies were reviewed and three in particular have been presented in detail for their overall relevance. Isolated good examples have always been referenced, incorporated and modified wherever relevant.

The second stage in the work methodology was to conduct a detailed analysis of the site and the overall context of the site with regard to the City vis-a-vis implications of existing conditions like traffic and proposed development activities. Such conditions have a bearing on the Project and the Project in turn will have a bearing on existing conditions.

The third stage concerned itself mainly with concept proposals presented to consultation groups consisting of the public, elected local government officials, technical personnel of the Thromde and senior management of the Thromde with the Thrompon at its head. These concepts are inextricably linked to the site analysis and overall citywide impact analysis as well as keen reference to global good practices. All proposals have been scrutinized, modified and collectively modified over a series of meetings and presentations satisfactorily.

The final stage as an output of the preceding stages is the final Landscape Masterplan. This masterplan proposes to develop a series of public open spaces interconnected by a continuous walkway and dedicated path for cyclists that contribute to the establishment of socio interactive spaces along the Omchhu. The vision is to create meaningful spaces that not only serve the residents but the city by the process of "place making" and establishing a "place identity" through the design of recreational spaces. These proposals will rely on riverfront developments and good practices from around the world. Different case studies and their analysis will provide the strategies of a successful riverfront with meaningful socio interactive spaces. From the synthesis of findings, a matrix outlying a set of design attributes and guidelines have been framed as in the table below.

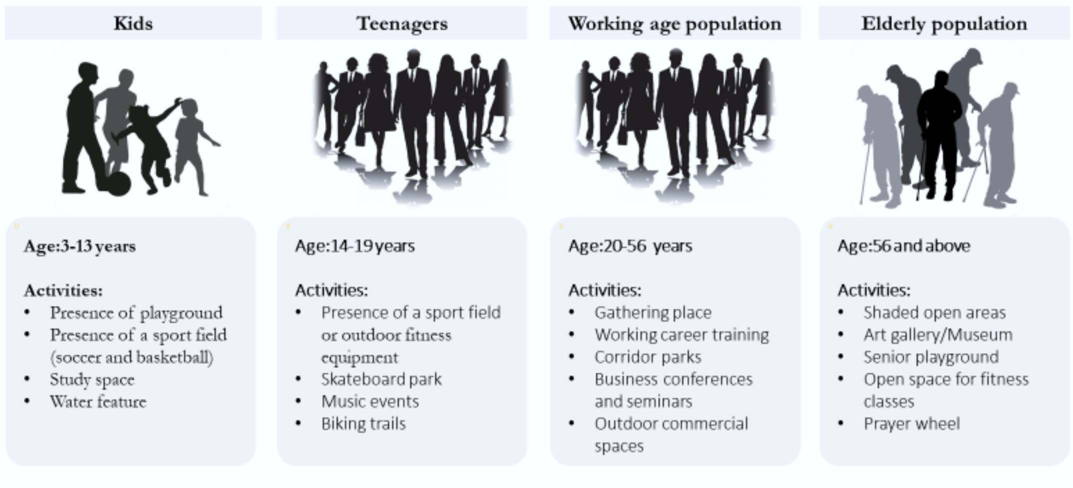
Table 63: The Design Matrix

PLANNING SOCIO INTERACTIVE RIVERFRONT SPACES THE DESIGN MATRIX GUIDELINE			
DESIGN ASPECT		STRATEGIES	DESIGN GUIDELINES
ACCESSIBILITY AND CONNECTIVITY	Physical Access	Physical access to the riverfront improved via key access points	Dedicated, safe and interesting pedestrian and bicycle routes
	Spatial and visual access	Reconfiguration of nodes and connections where possible to improve the connectivity within the city	Continue and strengthen existing routes to the riverfront and control building setbacks and height in proximity of the river where possible
	Vertical access	Vertical access points required to integrate the physical experience with the river and viewing the river from elevated walkways	Ramps to be featured as far as possible for universal access. Steps to be provided only where ramps are not feasible
	Connectivity	Connectivity to, along and across the river	Connectivity may be lateral, longitudinal or vertical achieved by paths, bridges and types of gradients
	Corridor continuity	Ensure path continuity along the river to achieve an ideal situation	Promenades along the river must be publicly accessible at all times and structures along the river must be setback sufficiently to accommodate the flow of people
	Vistas and Greenways	Enhancing existing routes to the riverfront and strategic planting of trees and green open spaces	Planning of new boulevards and green open spaces to improve the recreational network. However covered walkways and shelters from the sun and rain must also be considered
OTHER ASPECTS	Aesthetics	Research and apply	Sensitive and innovative
	Practicality	Achievable within reasonable cost	Life cycle of materials to be considered over materials that require constant maintenance

	Diversity	Multifunction amenities that are inclusive for all. Diversity of surfaces, tree and plantation along the riverfront	Suggest complementary amenities considering adjacent land use where applicable and suggest amenities that are best suited to particular locations. Trees and plant variety to add to seasonal variety and overall aesthetics, utility and practicality
	Character and Identity	Maintain and enhance national and local character where applicable	National identity and local identity could be in the form of architectural and sculptural details
	Safety	Maximize the safety of all users; especially for children and persons with disabilities	Railings of pedestrian/ cycling paths and bridges have to take into consideration child proof barriers.

Figure 124: User Needs' Analysis

Figure 125: User profile guide



12.3 Literature Review

The following case studies were selected for detailed review and presentation during the early stages of the Project. Presented here is a summary.

- Case study 1: The Green revolution: Land and river reclamation for recreational purposes: Riverfront Development Choenggyecheon, Seoul, South Korea.
- Case study 2: Reclaiming the polluted Sabarmati, Ahmedabad, Gujarat, India.
- Case study 3: The Ecological restoration of the Millennium Parkland, Sydney Olympic Park, Sydney, NSW, Australia

12.3.1 CASE STUDY 1: Riverfront Development Choenggyecheon, Seoul, South Korea

The Cheonggyecheon was once a small river that flowed in the historic part of Seoul and joined Hangang the main river of Seoul. The Cheonggyecheon was used by the local residents as a place to wash their laundry and fish. After the Korean war the demand for more roads turned the river into an elevated highway that serviced more than a hundred thousand vehicles daily. The Choenggyecheon river was lost. Growing environmental and historic consciousness initiated this Mega Project despite stiff political and local opposition to the plan considering the complexity, scale and prohibitive cost of the Project.

Figure 126: A map of Seoul showing the location of the Cheonggyecheon river and some key facts

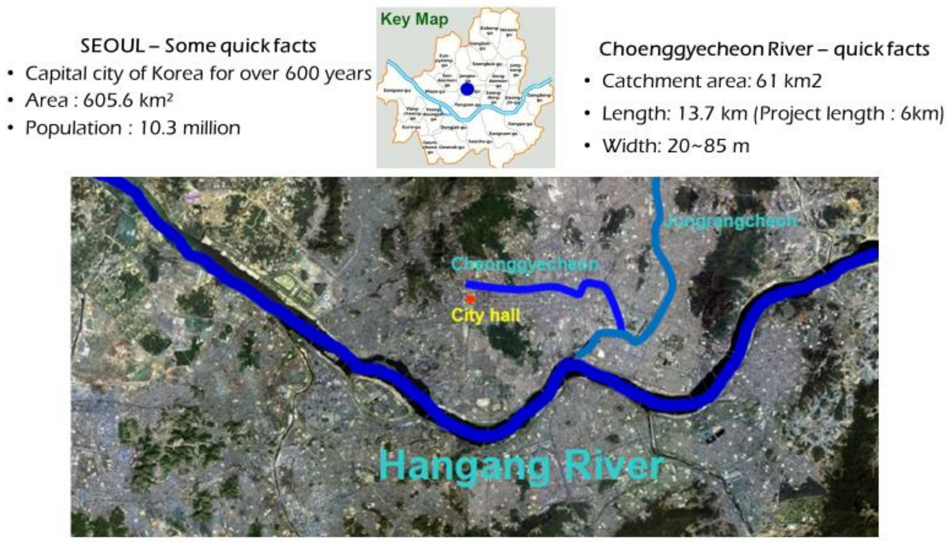


Figure 127: Map showing proposed bridges, ramps and stairs to improve the accessibility and connectivity to the Choenggyecheon river

Figure 128: Images showing proposed Thematic spots along the Choenggyecheon

Figure 129: Historical vignettes of the Choenggyecheon

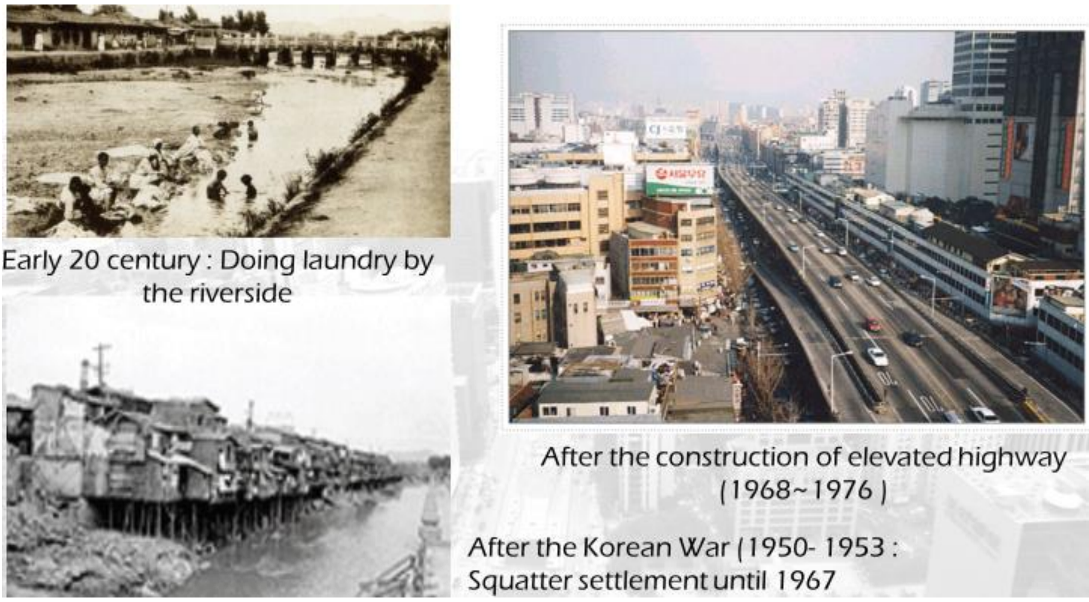


Figure 130: Images showing proposed work sequence on the Choenggyecheon reclamation Project

Figure 131: Section showing proposed attention to drainage and sewerage infrastructure

Figure 132: Images showing the success story of the Choenggyecheon reclamation Project

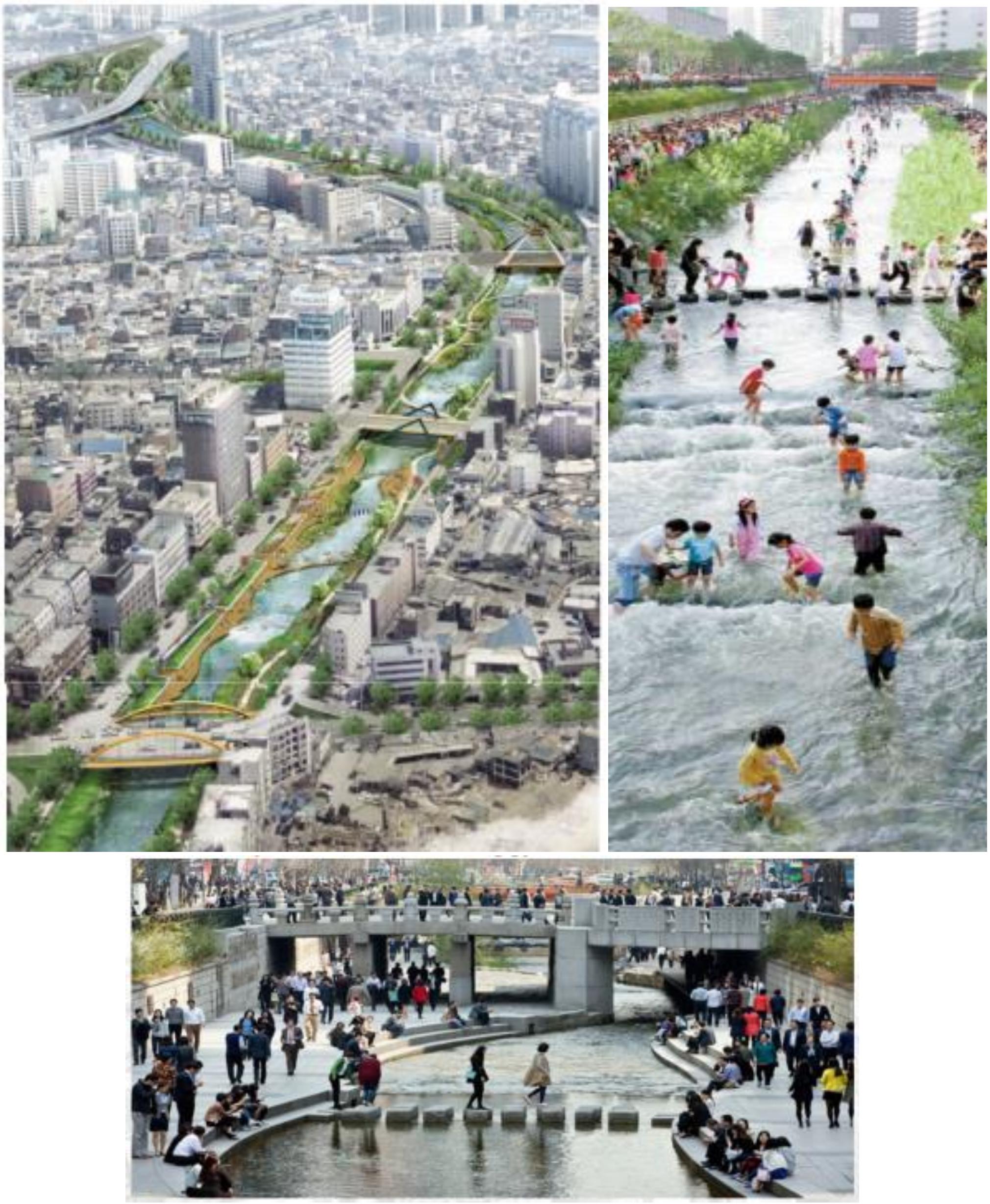


Figure 133: Images showing the attention to lighting details; appropriate degree of illumination for human, fish and insect life and 3 dimensional CRI (color rendering Index) considerations)

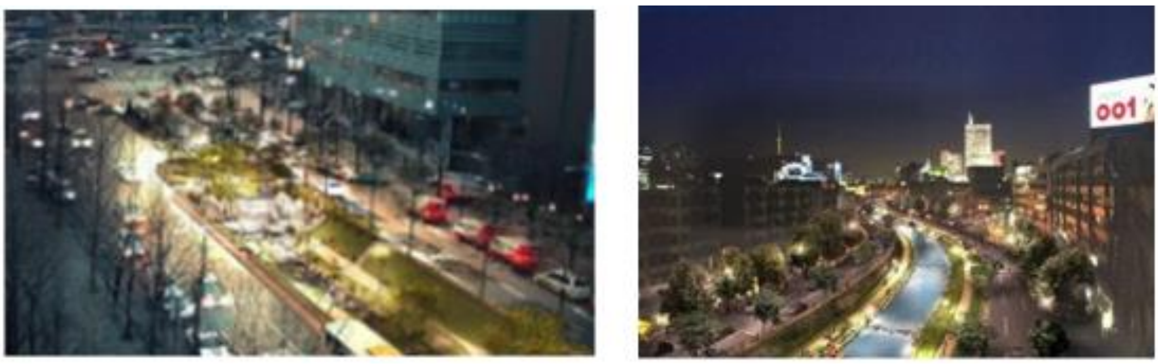


Figure 134: Images showing the river of traffic before and the Choenggyecheon river reclaimed

12.3.1.1 Key take-aways

The Choenggyecheon was once a river that was developed into a highway. Environmental and historic consciousness prompted a Restoration Project a green revolution to reclaim land for recreational purposes. The result:

- A river of traffic replaced by a natural lost river
- Created an identity for Seoul; an iconic public riverfront recreational area; Since opening in 2005 until Feb 2016, more than 200 million visitors reportedly visit the riverfront.
- Revitalized a historic area of Seoul; Increased investment opportunities
- Strengthened the city's pedestrian network by the multiplicity of bridges
- Flood control measures in place to account for a 200-year rainfall high design capacity
- Restored the river and the codependent ecosystem
- Improved the quality of life for citizens

12.3.2 CASE STUDY 2: Riverfront Development Sabarmati, Ahmedabad, Gujarat, India)

While the area along the river Sabarmati had been inhabited well before the 11th century the walled city of Ahmedabad is said to have been founded in 1411 by Sultan Ahmed Shah. Agruculture and rural settlments has actively been seen on both sides of the river Sabarmati since then.

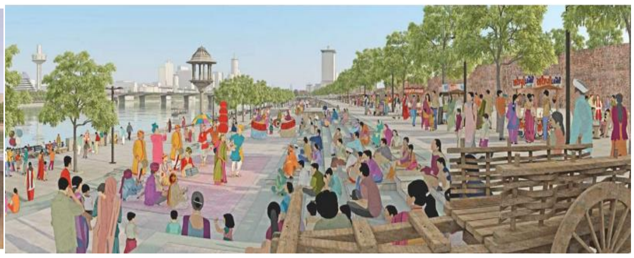
Figure 135: Locational map of Ahmedabad and map showing the Sabarmati at the Centre of the city

Ahmedabad is the largest and most populous city in the Indian state of Gujarat. Its population at about 8.25 million (2021 census) makes it the fifth most populous city in India. The Sabarmati River was the lifeline and literally the backbone of the city and the river divides the city into the eastern and western sections. However the polluted river and the type of squatter settlements compounded by sewerage as well as residential and industrial waste discharging directly into the river polluted the river heavily and rendered the riverbanks unaesthetic and underutilised as far as its economic and recreactional potential was concerned. These conditions kept the river from the inhabitants of the city.

Figure 136: The previous state of affairs of the Sabarmati River and river bank

Figure 137: The design proposals for flood protection and infrastructure development

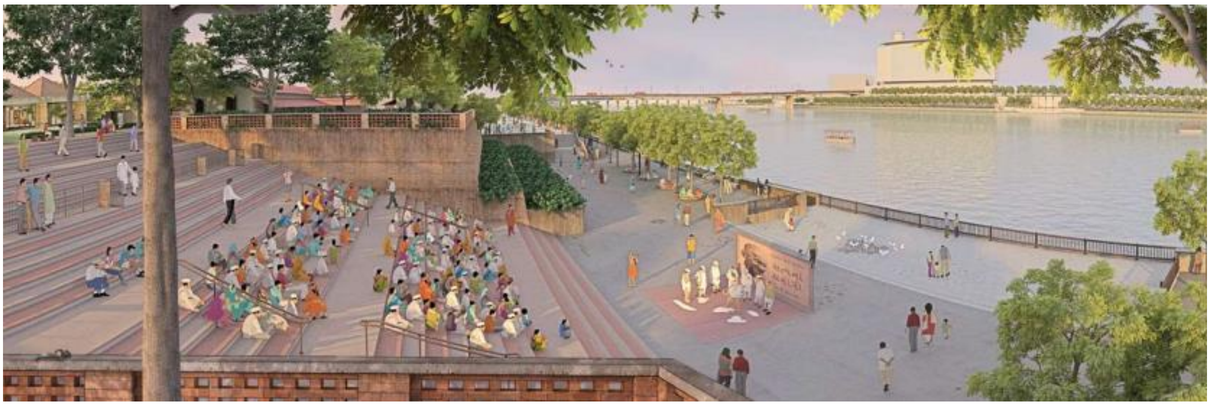
Figure 138: The proposal for the Vallabhsadan Plaza and the Heritage Plaza



Sabarmati river was abused, neglected and difficult to access with 12,000 hutments on both banks of the river and had the unorganized Gujari bazaar of more than 1200 vendors on the eastern bank facing court litigation and about 200 dhobi families using both banks of the river along with more than 40 storm water outlets and drains polluting and contaminating the river. Besides being an issue of critical environmental degradation, it was a political time bomb in itself.

As a national heritage site, this plaza alone opens directly onto the Sabarmati via a series of wide steps that create an amphitheatre connecting the Ashram to the lower level river promenade.

Figure 139: The proposal for the Gandhi Ashram Plaza



The riverfront as a multi faceted recreational area for all ages includes sports area complexes like the Paldi, Pirana and Shahpur sports grounds. The Laundry campus created to provide state of the art laundry facilities to improve the livelihoods of the community that traditionally washed laundry on the river banks

Figure 140: The 50,000 m² Flower Garden (amongst many others) designed as the venue for Ahmedabad's annual flower show though it serves a neighborhood park for the rest of the year.

Figure 141: The network of recreational areas improved by increased river crossings enhancing connectivity and the livability for the citizens of the city

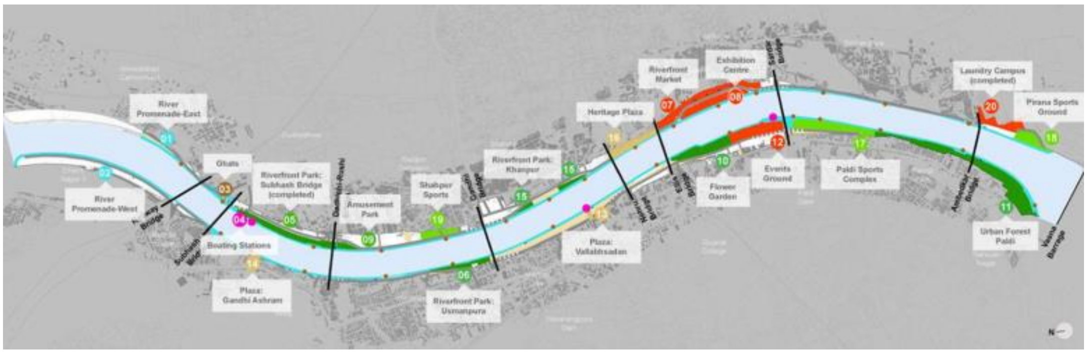


Figure 142: The Sabarmati redeveloped to become an iconic riverfront development model

12.3.2.1 Key take-aways

The Sabarmati was once literally the dump yard for Ahmedabad. Environmental awareness and the potential for great economic benefits transformed the polluted river and riverfront into an iconic riverfront development project. The key take-aways are :

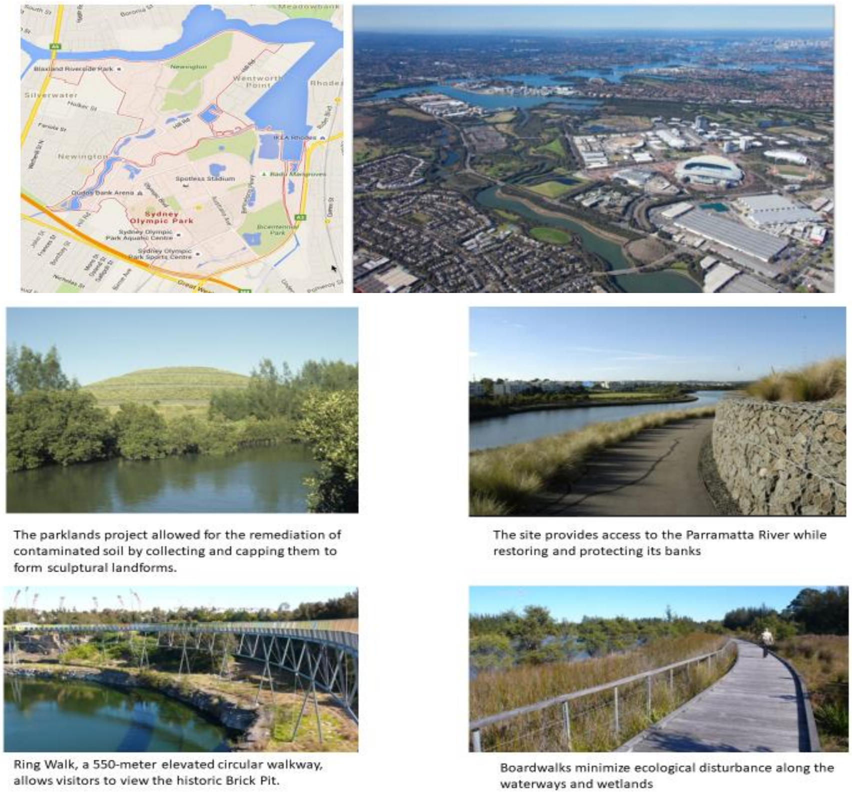
- Revitalised Ahmedabad as a whole as the Sabarmati is central to the city
- Flood control measures in place to account for a 100 year high rainfall design capacity
- Restored the river and rejuvenated the badly depleted and eroded ecosystem
- Added about 1,000,000 sq mts of parks/multifaceted public open spaces along the Sabarmati
- Increased investment opportunities as 15% of land is now leased for commercial, retail and residential development
- Revamped the city's transportation and pedestrian network by the multiplicity of bridges, enhanced and streamlined roads
- Slum upgradation and poverty alleviation through social housing schemes.
- Improved the quality of life for citizens

12.3.3 CASE STUDY 3: Millennium Parkland, Sydney Olympic Park, Sydney, NSW, Australia

The Millennium Parkland at the Sydney Olympic Park occupying 1000 acres (4.046 million sq mtr) was completed in 2015 and is considered a success story for the ecological restoration of the wetlands of the Parramatta river and Homebush bay.

It provides venues for a variety of recreation and leisure pursuits for 2.5 million people annually. Visitation grew from 750,000 in 2002 to 2.3 million in 2007. It also provides educational opportunities for nearly 20,000 children annually, with 18,600 students participating in curriculum-based environmental education programs in the parklands in since 2006-07.

Figure 143: Map and photographs showing the Sydney Olympic Park and the Millennium Parklands



The part has a central theme of 'Sustainability'. Sustainable features of the Sydney Millennium Parklands include;

- Contaminated soil was collected and capped to create a series of positive landforms, ranging from 20 to 60 meters high.
- An existing central surface parking lot was replanted and partially developed as a park arrival and service village to provide information and a starting point for train, jitney, bicycle, and pedestrian ways.
- A system of separate paths for walking, bicycling, and jogging stretches the full length of the river and bay frontage and includes a continuous lighted boardwalk and bicycle promenade.
- The Ring Walk, a 550-meter elevated circular walkway, allows visitors to view the historic Brick Pit, which provides habitat for the Green and Golden Bell Frog.
- Linear forested buffers were created around each of the parkland parcels.
- The lawn and buildings of the former Royal Australian Navy Armaments Depot and a 124-acre aboriginal forest were preserved and restored.

Figure 144: Masterplan and 3D of the park

12.3.3.1 Key take-aways

- Restored and protected more than 15 miles of continuous waterfront along the Parramatta River and Homebush Bay, including a 124-acre Aboriginal forest.
- Treated contaminated soils. Roughly 35 megaliters of leachate were collected and transferred to a waste treatment facility. Groundwater contaminated with 750 kg of hydrocarbons, including 430 kg of benzene, has been successfully degraded by microorganisms in the Wilson Park bioremediation ponds.
- Recycled over 4,600 megaliters of water over 7 years, providing irrigation and greywater for on-site use. Of total water consumption during this period, only 2% was sourced from Sydney's water supply despite one of the worst droughts in Australia's history.
- Provides habitat for more than 180 native species of birds, including those in decline in other areas. The once-endangered Green and Golden Bell Frog population in the parklands is now one of the largest populations in New South Wales.

12.4 Site And Context Analysis

The section presents in a photographic fashion, the site and context analysis for the formulation of masterplan.

Figure 145: Map of Bhutan and the Project Area

Figure 146: Key neighborhood context

Figure 147: Map and photographs indicating the detailed Site context near the PHSS/NPPF colony

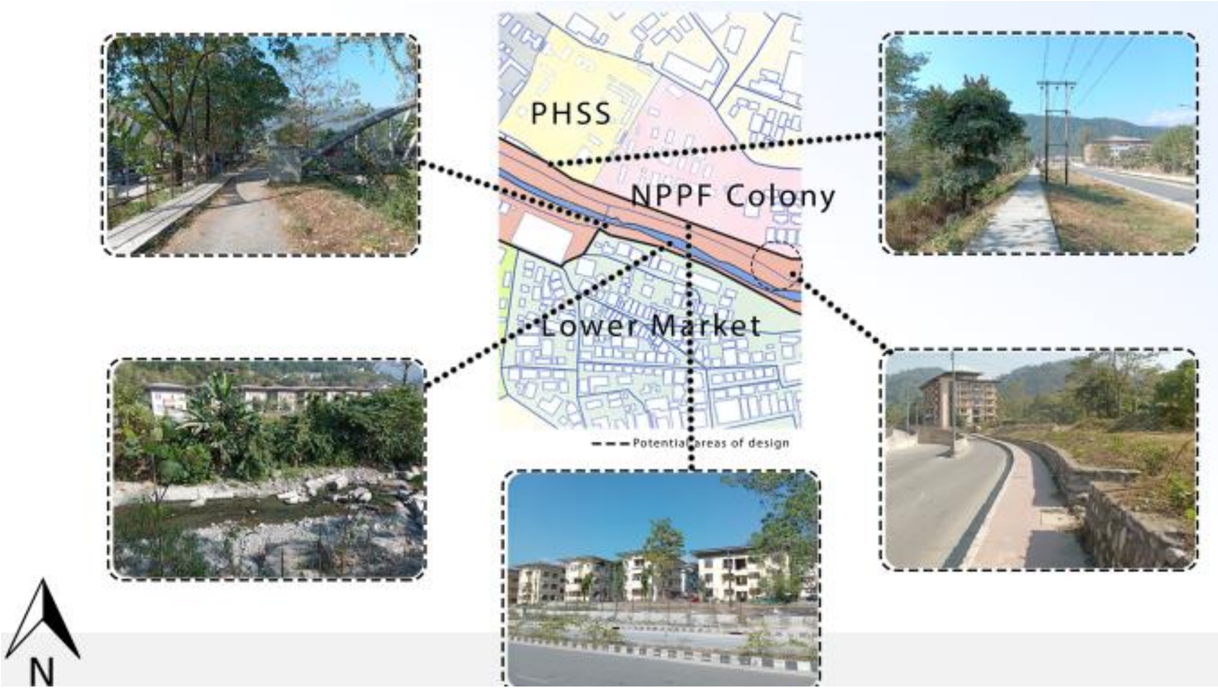


Figure 148: Map and photographs indicating the detailed Site context at lower Omchhu

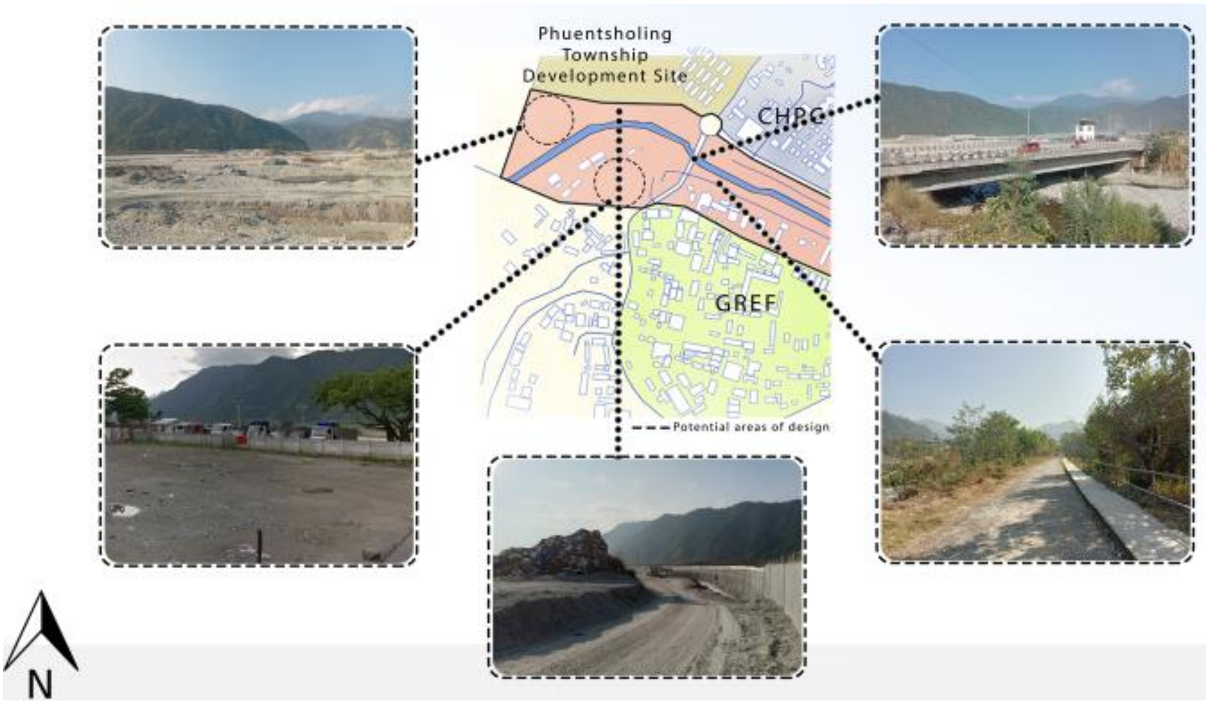


Figure 149: Map and photographs indicating the detailed Site context near the curvilinear bridge

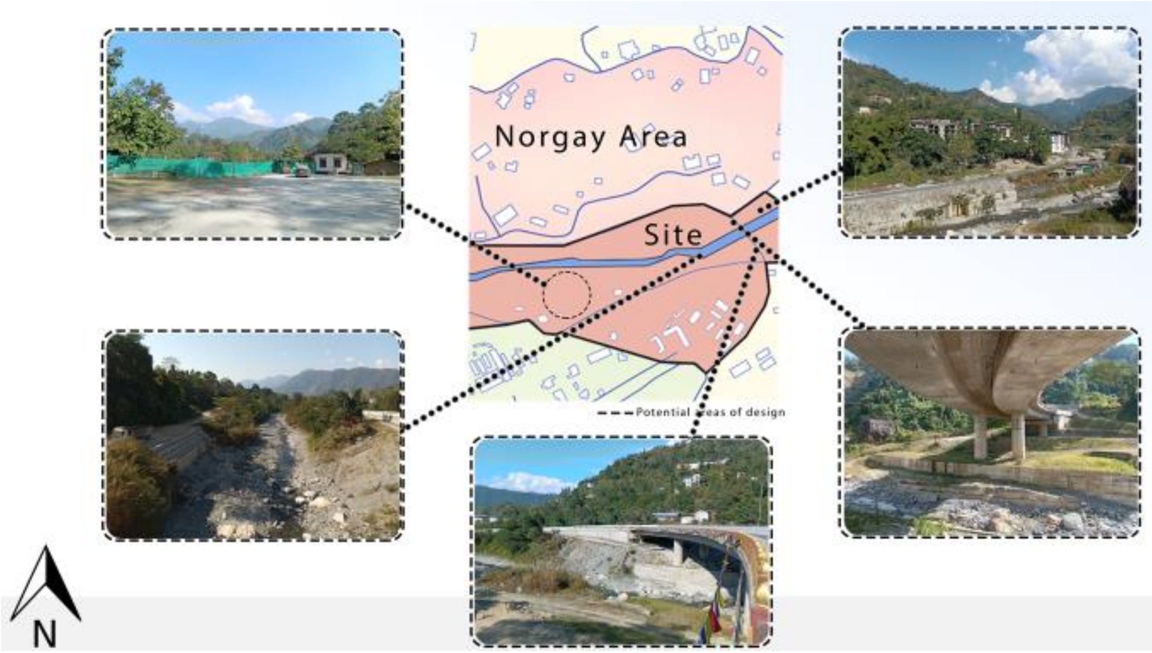


Figure 150: Built area, and flora and fauna

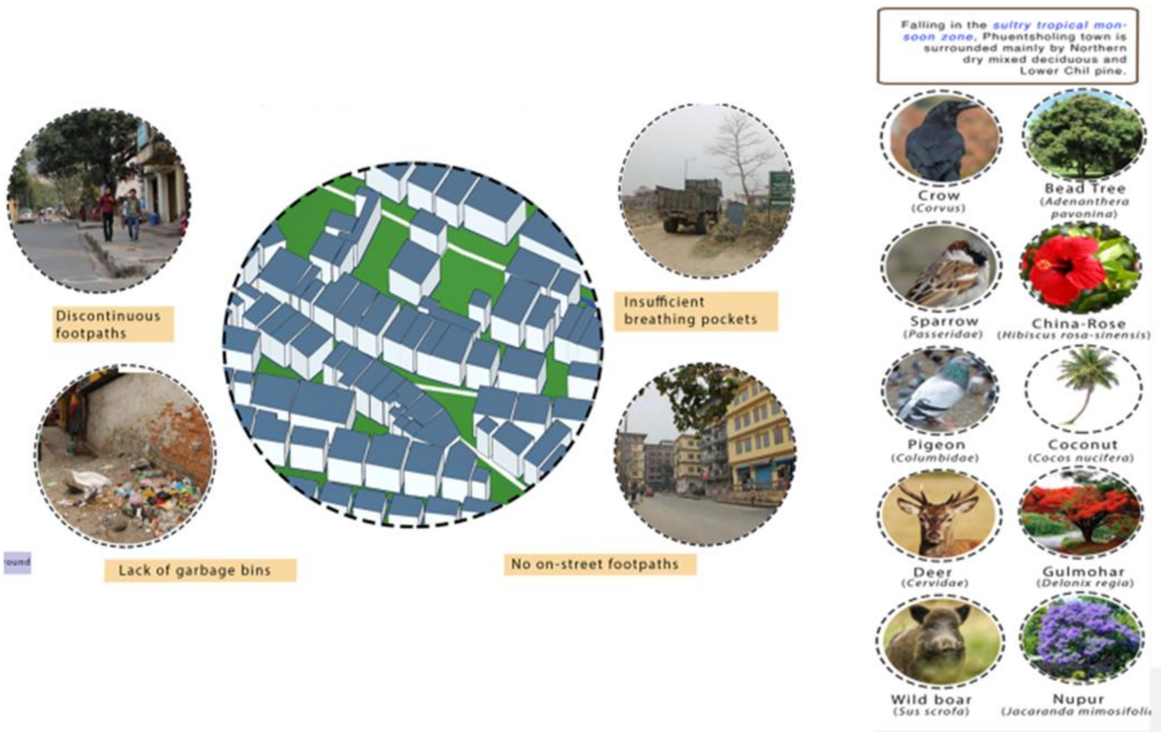


Figure 151: Sun path analysis for the Omchhu river basin area

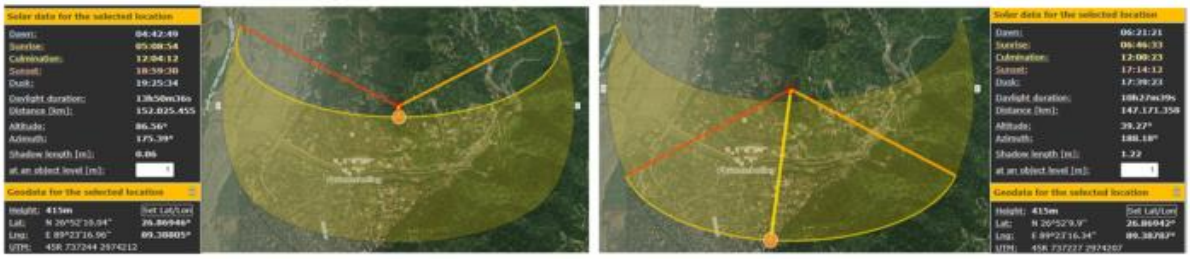


Figure 152: Longitudinal section through the Omchhu

12.4.1 Specific Sections AS-IS

Figure 153: Map of the Omchhu indicating the location of sections

Figure 154: Section 1 near the bridge closest to the PTTD township/ YDF bridge

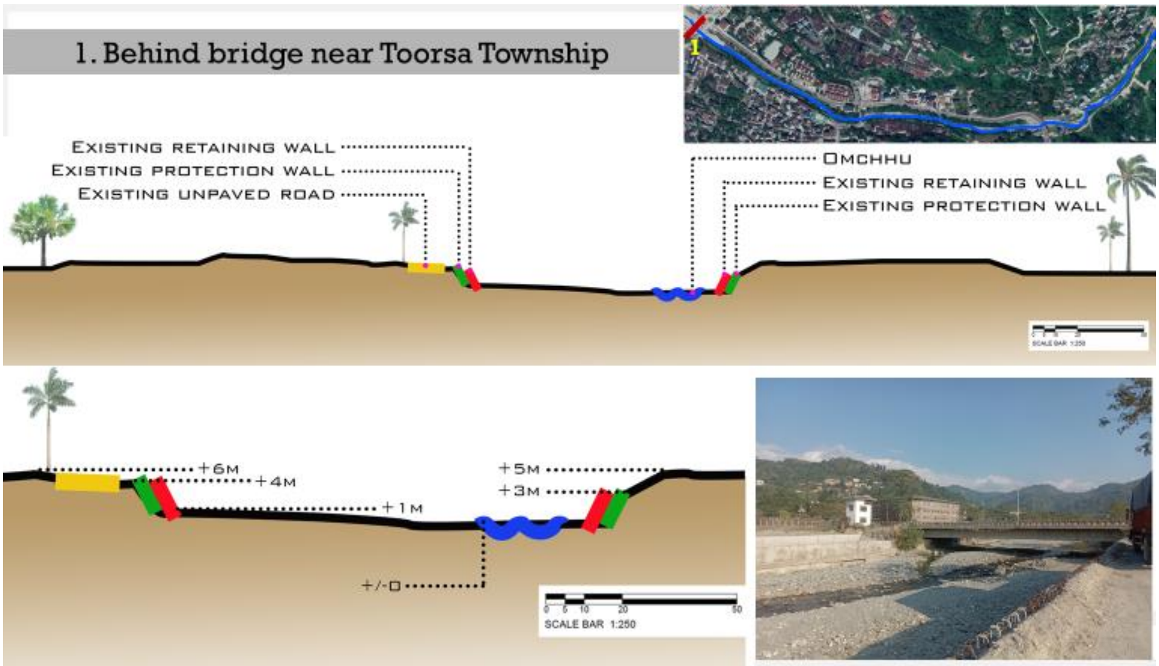


Figure 155: Section 2 between the YDF bridge and the pedestrian bridge near the Vegetable market

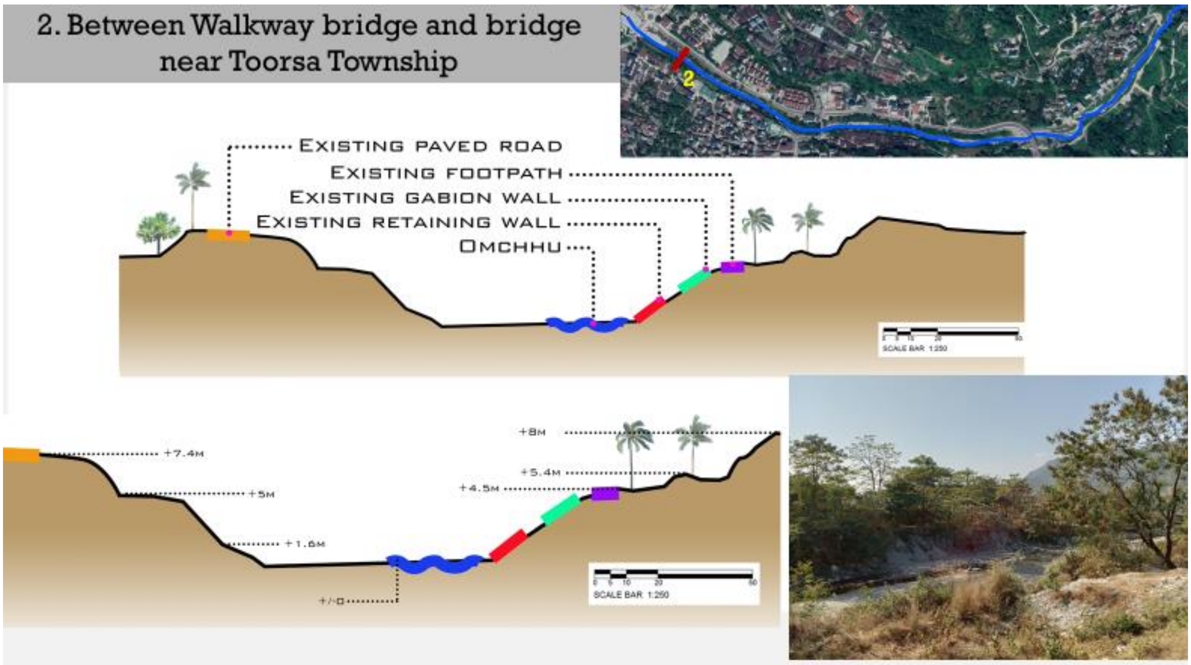


Figure 156: Section 3 near the pedestrian bridge near the vegetable market

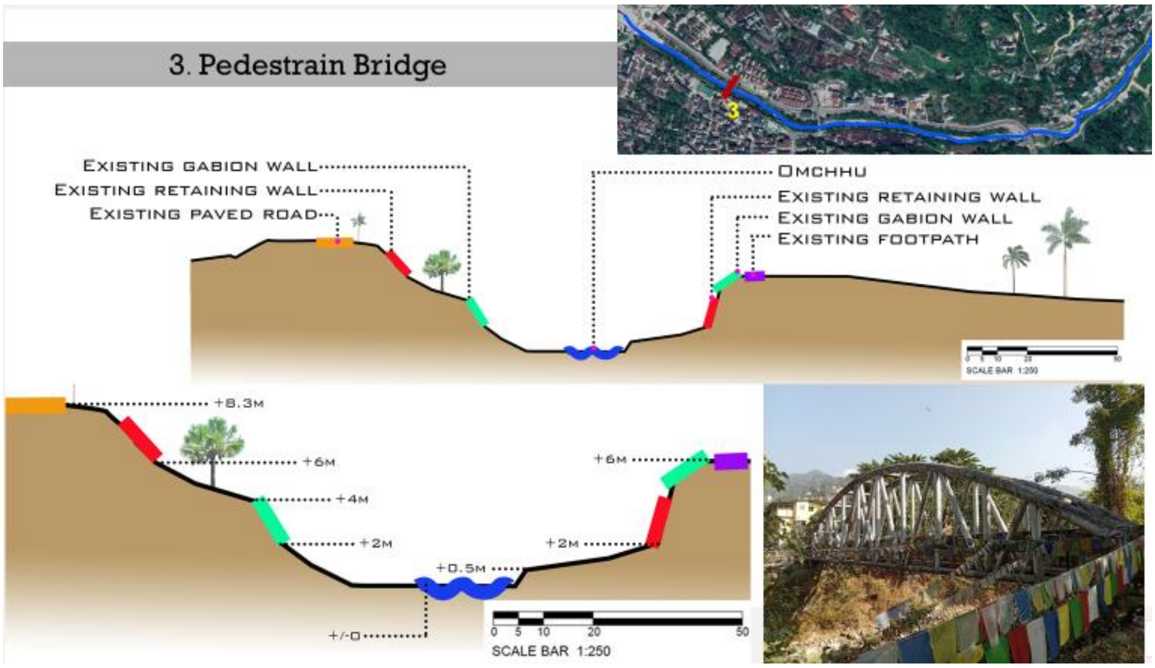


Figure 157: Section 4 near Hotel Phuentsholing/Norgay bridge

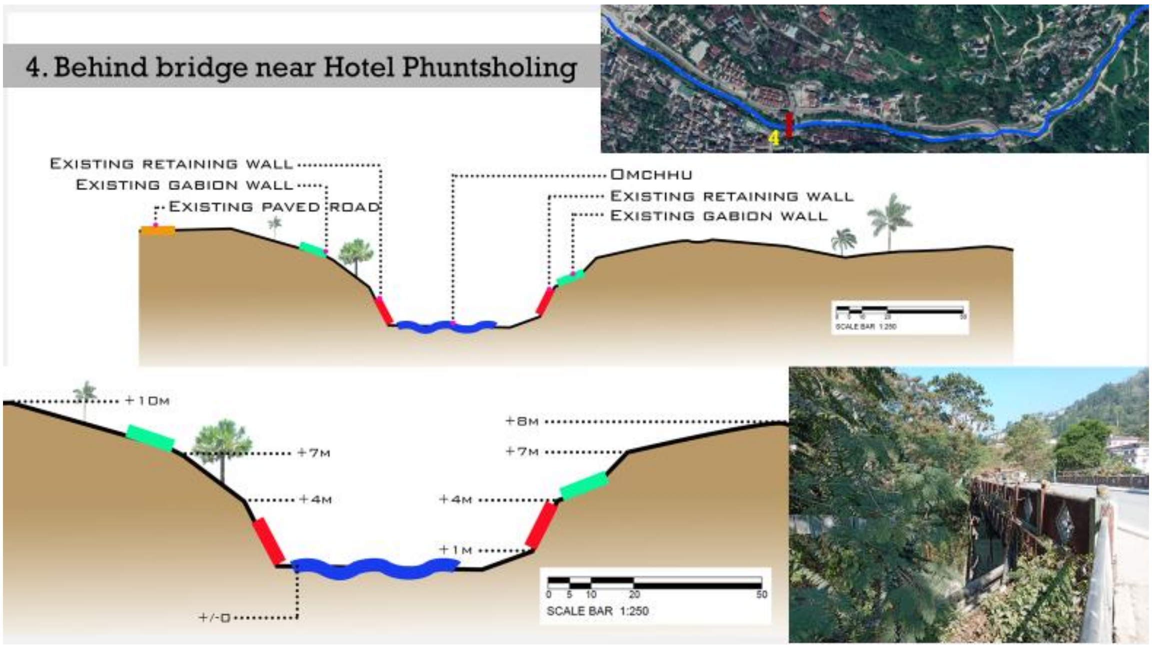


Figure 158: Section 5 near the RSTA bus terminal, Norgay area

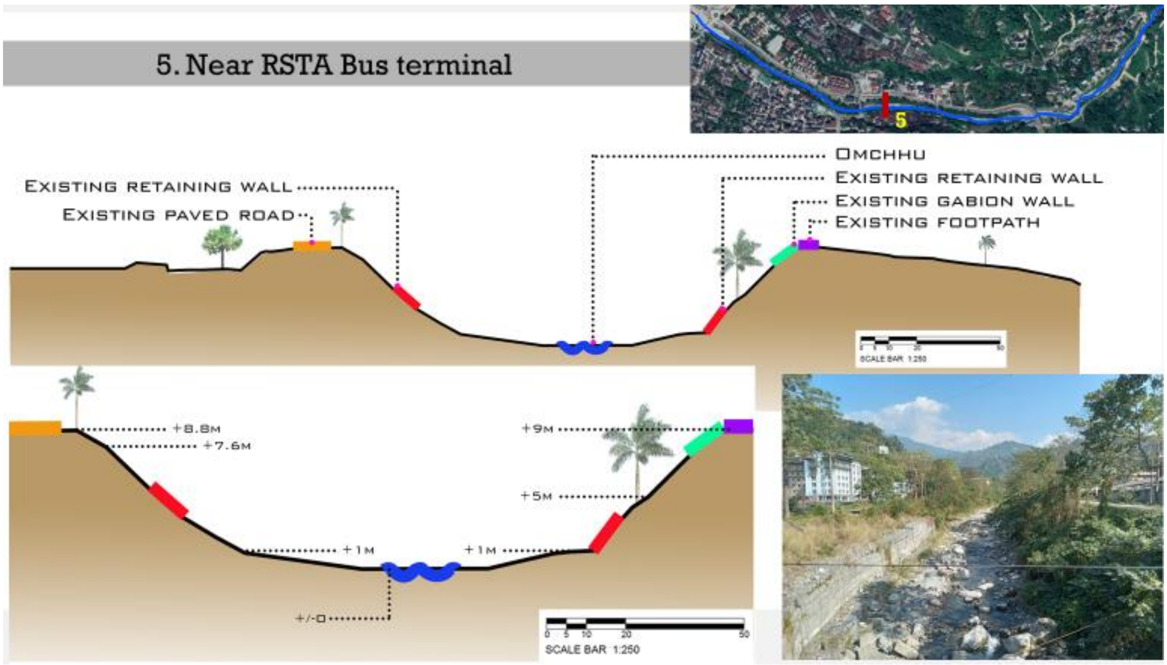


Figure 159: Section 6 opposite the Archery ground

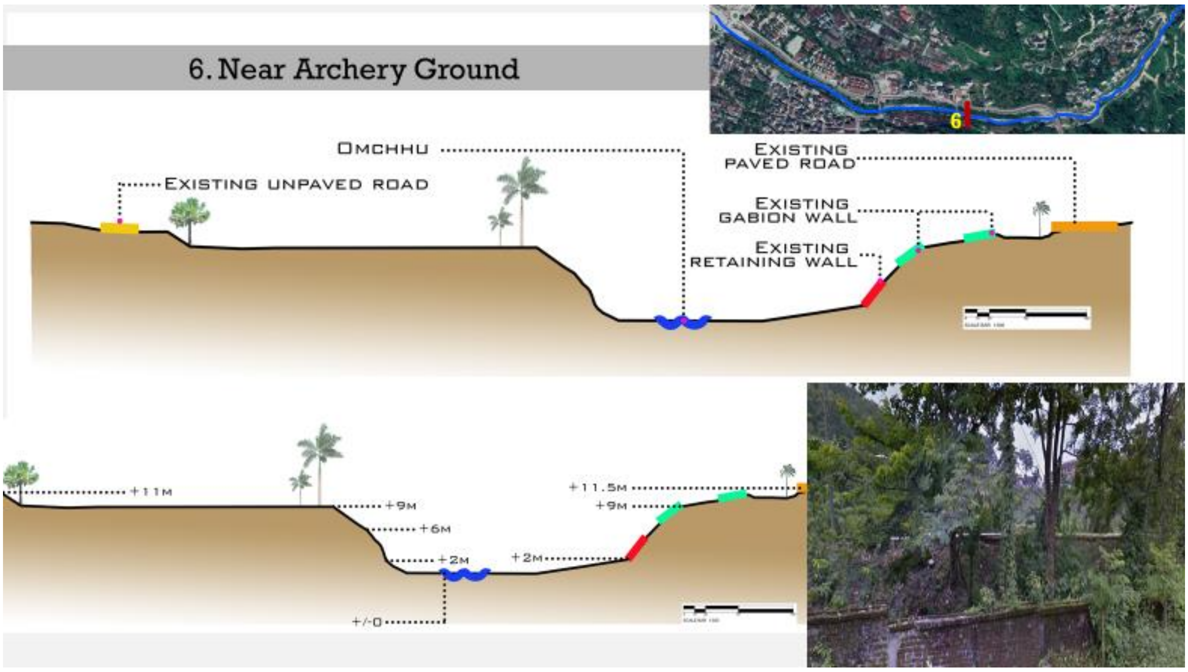


Figure 160: Section 7 near the curvilinear bridge

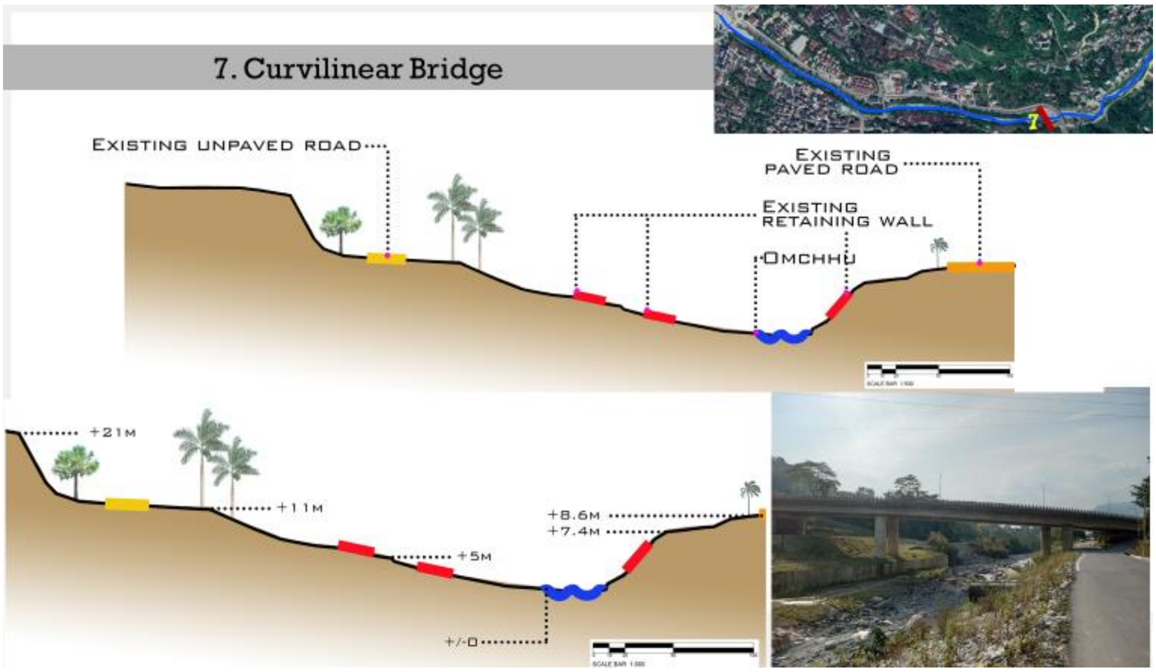
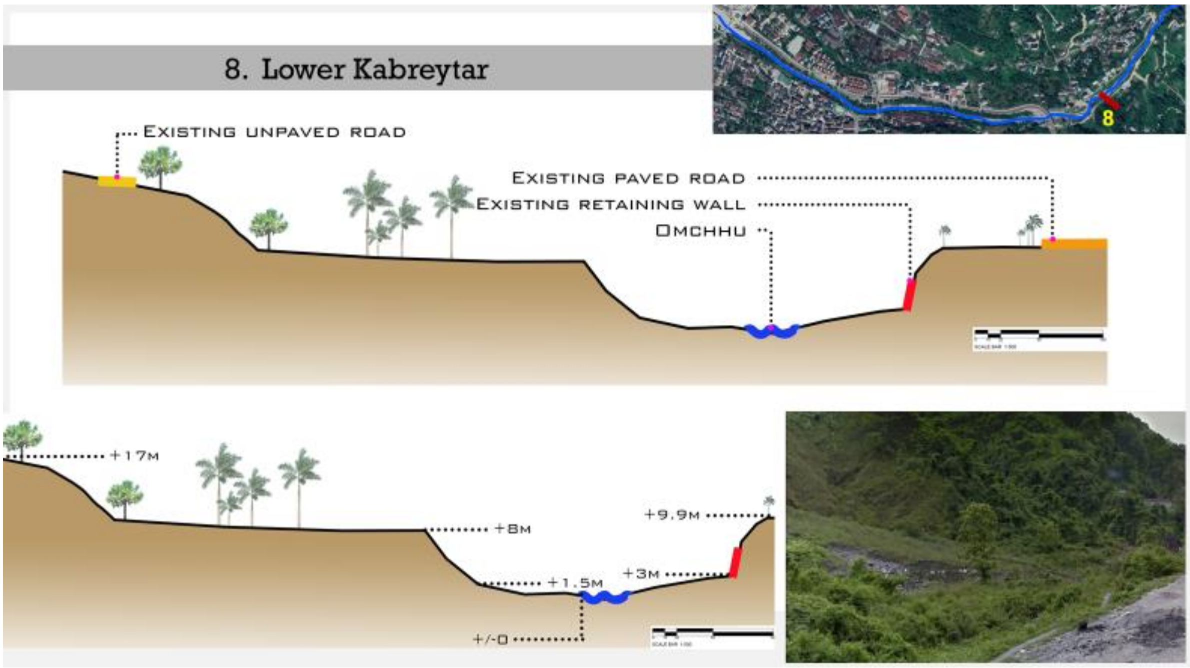


Figure 161: Section 8 lower Kabraytar



12.5 The Proposal

Key issues and considerations of proposed Riverfront Development

The land reclaimed after the river training works will be used as sites for riverfront development works. A comprehensive study of the entire area and nearby settlements along the area of interest has been done to assess the viability of the proposals. The landscape proposals have been designed considering the existing and anticipated topography after the river training works are completed.

History and Location – Each riverfront comes with a different context and history. The Omchhu has a history of being a very turbulent and destructive river during the monsoons. The key issue for the Omchhu rehabilitation is the risk of flood and damage to property, infrastructure and human lives. The need to provide a robust engineering design overrides all other concerns. Landscaping and garden provisioning is to be exercised only after the flood water mitigation is considered. It is for this reason the discharge volume for a 100-year flood risk (600 m3) determines the minimum height of the RCC walls. This has in effect changed the proposal from a two-level footpath and cycling path to a single level as the river width and height cannot be compromised. During the winter months the river is barely a stream and the river basin is an expanse of boulders, rocks and undefined gravel paths and vegetation that is barren, uninviting and therefor underutilized.

Liability- Infrastructure near water always pose a major risk especially for children. Certain liabilities like falling in the river while using walkways that have no railing, falling in water bodies etc. have to be taken into consideration.

Education / Interpretation. Riverfront redevelopment is a great way of educating the residents and visitors about the region's biological/ecological diversity and the city's cultural heritage where applicable.

12.6 Salient Design features

The Omchhu Riverfront proposal now host a vibrant and robust combination of many amenities, like events plaza, sports arena, parks, gardens, open spaces, footpaths and cycle trails and several other community assets and infrastructure like cafeterias, vendor stalls, public toilets, cycle sheds etc. as it is agreed that the riverfront area should be an environment for diverse activity and expanded recreational opportunities besides being flexible and can respond to market conditions and economic opportunities. A mix of open and covered spaces has been considered and proposed. These provide opportunities for year-round activities. The Omchhu riverfront has been envisaged as a well-known destination that draws people of all ages and the region to the riverfront and should offer places and opportunities for celebration throughout the year; from small informal gatherings to large festivals and educational activities. Recreational facilities include playground facilities for children and parks which can enhance learning of local Bhutanese culture, flora and fauna as well as of the nation.

Open sports facilities for people like open air gyms for yoga and exercise, volleyball, badminton, futsal grounds, spread along the Omchhu banks have been proposed. A more pedestrian oriented waterfront, with walking and cycling trails inviting intimate contact with the landscape and free from vehicular traffic. Barrier free or universal access design principles have been integrated in the design of all recreational facilities to make them accessible to all people, regardless of age, disability or other factors.

Diverse trees, both ornamental and fruit bearing, evergreen and deciduous trees, plants and flowers have been intentionally chosen. Suitable evergreen species at appropriate locations for shade and aesthetics have been selected. Besides common trees like the Gulmohar, Jacaranda, Palm lesser-known tropical ornamental trees like Flame of the Forest, Himalayan ash, Hong Kong Orchid, Javanese Cassia, Mexican lilac, Purple Glory, Sandpaper Vine, Tabebuia, Yellow Mai tree, Yellow Elder etc. are being proposed to introduce a variety that is both refreshing as well as educational. Lawn grass has been proposed over the entire stretch of the Project area to reduce dust pollution and promote greenery. Grass that is flood and drought resistant need's introduction. Vetiver or khus is one such grass. Vetiver is most closely related to Sorghum but shares many morphological characteristics with other fragrant grasses such as lemongrass, citronella and palmarosa. The vetiver bunch grass in tufts and its root system is finely structured, is very strong and can grow about 3 m deep within the first year. It is thus frost, wildfire, drought and even flood resistant. Under clear water the plant can reportedly survive up to two months. Its strong deep fibrous root system can help to protect soil against heavy grazing pressure, and soil erosion.

The riverfront is designed with walkways, recreational parks, religious and cultural attributes, exercise and play areas for children and adults, fountains etc. Creating multiple destinations, connecting destinations and optimizing public access for interaction purpose. Connecting public open spaces with a continuous riverfront trail to link destinations and serve as a destination for walking, jogging and other related purposes. Facilities such as drinking water taps, public toilets and eateries have been provided as per the requirement worked out. Adequate artificial lighting with innovative ideas such as using eco-friendly lights have been proposed.

Interaction with the river has become important for planning of sustainable development. This can be tackled by selecting the heights, materials used for building, native as well as exotic plants for landscaping; reusing disturbed areas and building within the context. Public accessibility must be enhanced. People are drawn to water. Human interaction with water is innate and instinctive especially when complimented with sensitive design that is both inviting and innovative. Once introduced landscaping and water features inculcate the desire to be near it, physically or at least in its visual proximity. It therefore becomes critical that the Omchhu should have water in it especially during the non-monsoon seasons. Omchhu river front landscape development would be significantly impaired if the Omchhu didn't have water in it. To achieve this check dams have been introduced. These check dams perform the following critical functions

- Checking the flow of the river in flood
- Storing water for aesthetic and practical purposes like gardening and rainwater storage

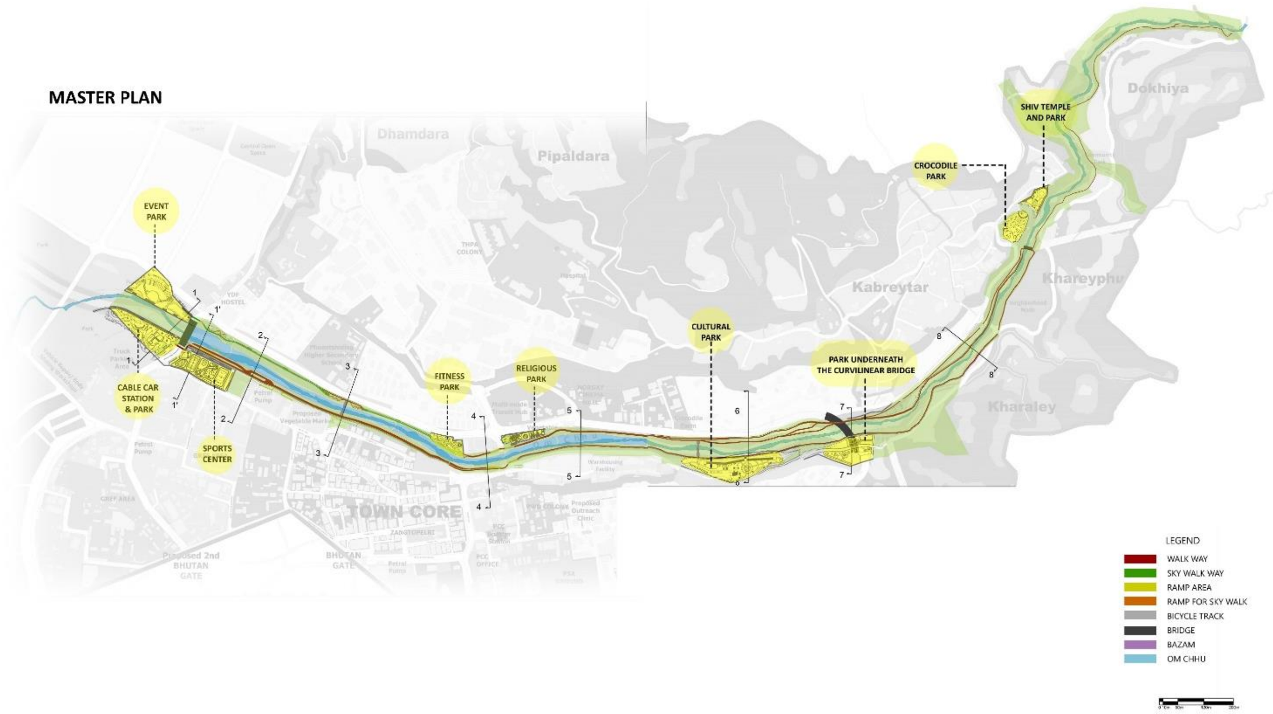
The check dams also offer the opportunity to generate electricity through mini hydro-electric plants to provide electricity for the electrification of the riverfront and paying for its own upkeep. It must be mentioned here that the ferocity of the Omchhu during the monsoons will in all probability cause damage to these dams. Yearly maintenance to the dams themselves as well as annual dredging have to be done periodically. Nine of these check dams have been provided- mainly in the middle stretch of the Omchhu. These dams have also been strategically located to enhance the overall ambience of the landscaped gardens and parks along the Omchhu.

To achieve these objective walkways, trails and benches are provided as they give people an opportunity to be either in the river or near it. An effective or fruitful riverfront having active use can be achieved if multiple entry points to the river are available. Walkways along with bridges are important as they define as well as provide variety in movement patterns on the site. They are also physical pedestrian linkages between different parts of the site. A traditional looking Baazam constructed of RCC has been proposed to add a Bhutanese architectural element to the entire ensemble. In a similar vein a suspension bridge has been introduced. Besides being a cost-effective way to provide pedestrian, there is still the novelty of walking on a swaying suspension bridge.

12.7 Synthesis of planning

Riverfront as vital breathing space with meaningful socio interactive spaces that enrich the lives of residents and visitors besides providing economic opportunities for both the Thromde and locals residents. The following pockets along the Omchhu have been designed and the impact that public spaces on a riverfront have can be seen in an area's development in economy, people's health, recreational & tourism activities.

Figure 162: Masterplan of the Riverfront Development Project



Apart from river protection structures, we have parks along the river all interconnected by walkways, cycling lanes and even bridges. All the parks are designed and placed based on our studies of the site and the surrounding context. Each park is proposed in a certain location to activate the surrounding area, creating opportunities for social and economic growth. Kids, teenagers, adults and elders all have the opportunity to come together and enjoy these spaces.

12.7.1 Ropeway Park (Zone 1)

The Ropeway Park is designed as per the Thromde's suggestion for a future provision to connect different areas of Phuentsholing. The park is located right next to the existing truck parking which later has the potential to be a bus parking. The ropeway is also placed close to the event park. This is done intentionally so that it eases commute for the general public during shows and events. This area can also be the starting or the end point for all the people who come to enjoy the cycling or the jogging trails along the Omchhu River.

Apart from the provision for the Ropeway, the park is also integrated with programs like parking space, cafeterias, toilets, small parks, open air gazeboes, open and covered walk ways.

ZONE 1: CABLE CAR STATION & PARK

ZONE 1: CABLE CAR STATION & PARK

BIRD EYE VIEW

THE CAFE, POND AND THE CABLE CAR STATION

BIRD EYE VIEW FROM PLAZA AND HIGHWAY SIDE

TRIANGULAR THREE SIDE ELEVATED WALKWAY WITH GREEN AND POND BYSIDE

THE AMPITHEATER AND ELEVATED WALKWAY

APPROACH FROM THE HIGHWAY PARKING

OMCHHU LANDSCAPE

12.7.2 Event Park (Zone 2)

The Event Park is located in the center of the current town and the future town expansion in Toorsa. This is area not too far away like the rest of the parks and has easy accessibility from the road and enough parking for a big crowd. The event plaza provides the opportunity for all the people to host concerts, cultural shows, food festivals, exhibitions and other programs. The space has an outdoor stage and an amphitheater which can accommodate up to 700 plus people.

The park also accommodates other programs like parking space, cafeterias, toilets, small parks, open air gazeboes, open, prayer wheels and covered walk ways.

ZONE 2: EVENT PARK

ZONE 2: EVENT PARK

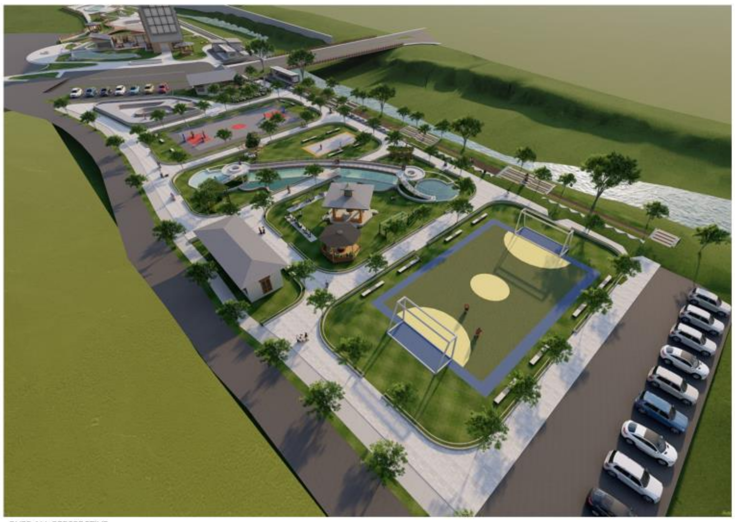
OMCHHU LANDSCAPE

12.7.3 Sports Park (Zone 3)

The sports center is located close to the school, the rope way park and the event plaza. The sports center consists of various outdoor sports like futsal courts, basketball courts, skate park, badminton court and bicycle stand. The area also has cafes, public restrooms, outdoor water bodies and outdoor seating areas.

ZONE 3: SPORT'S CENTER

ZONE 3: THE SPORT'S CENTER


OVER ALL PERSPECTIVE


SKATE APRK


ELEVATED WALKWAY


THE ELEVATED WALKWAY


THE STEPPED SEATING AREA ALONG THE RIVER


SEATING AREA WITH NET

OMCHHU LANDSCAPE

12.7.4 Fitness Park (Zone 4)

The Fitness Park is located on the other side of the bank located in the areas close to the NPPF colony. The park consists of outdoor play areas for kids and exercise areas for adults with outdoor gym equipment. The park is also connected to the Sky walk structure which has bicycle lanes on the lower level and walkway area on the top. The idea of the sky walk is to provide the public elevated walkways with a nice view of the river and the surrounding area. The elevated area also consists of covered zones where we have proposed solar roofs. These solar roofs will provide lighting to the sky walk and the parks nearby. The Sky walks are designed with ramps so that wheel's chairs can also access these areas.

FITNESS PARK

FITNESS PARK

BIRDS EYE VIEW

VIEW OF RAMP

VIEW OF PARK FROM RAMP

VIEW SHOWING KIDS PLAY AREA

VIEW SHOWING ADULT EXERCISE GYM

VIEW SHOWING OUTDOOR NOOK AND YOGA AREA

OMCHHU LANDSCAPE

12.7.5 Dharma Garden (Zone 5)

The Dharma Park is located next to the RSTA office/the bus stop and the residential areas. The park consists of religious stupas, prayer wheels and other amenities like public restrooms etc. This park is provided in order to promote the cultural heritage and promote more outdoor spaces for the elderly people and people waiting at the bus stop.

THE RELIGIOUS PARK

THE RELIGIOUS PARK

PERSPECTIVE FROM ENTRANCE

BIRDS EYE VIEW

CHORTEN VIEW FROM SEATING AREA

LEFT ENTRANCE APPROACH

MANI DHUNGKHOR VIEW FROM ENTRANCE

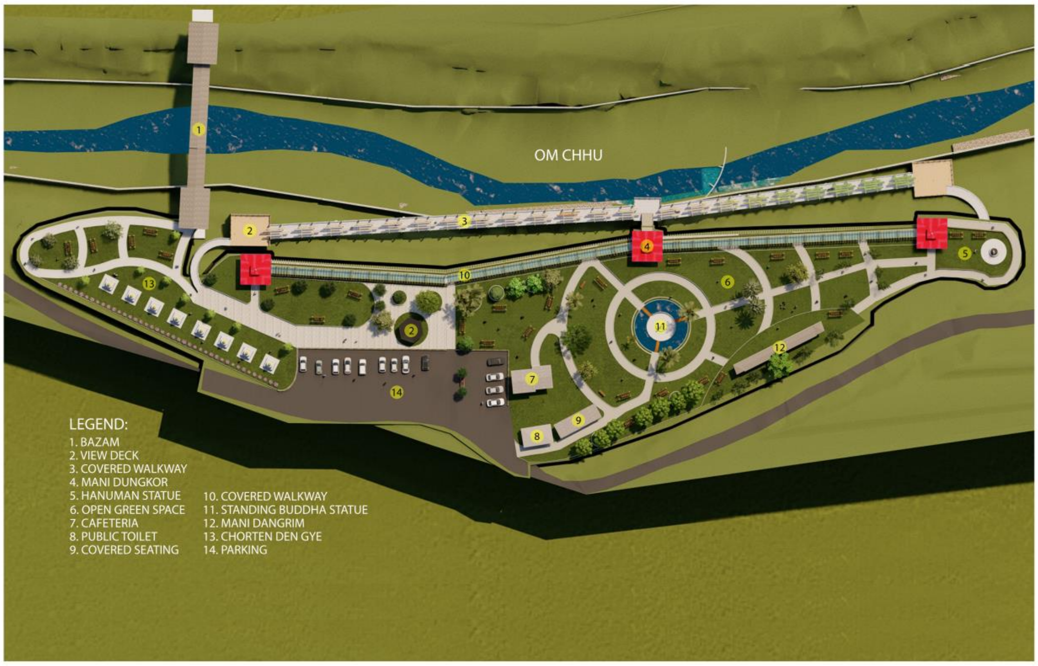
RIGHT ENTRANCE APPROACH

OMCHHU LANDSCAPE

12.7.6 Heritage Park (Zone 6)

The Heritage Park is located a little further away from the main town and the busy streets Phuentsholing, which makes it an ideal location as a picnic spot for the people. The park consists of green areas, gazeboes, water bodies, cafes, public restrooms, religious stupas and covered walkways along the Omchhu river. A Bazam (traditional bridge) is also proposed in this area to connect this zone to the other side making the parks more integrated.

ZONE 4: CULTURAL PARK



ZONE 4: CULTURAL PARK

OMCHHU LANDSCAPE

12.7.7 Recreational Park (Zone 7)

The outdoor amphitheater is located right below the curvilinear bridge. During our site study it was found that the area below this bridge had the potential for outdoor activities. This zone will have outdoor exercise areas and an open amphitheater and since its located right below the bridge you have shelter from the rain. This area will promote outdoor activities for kids and adults residing in these areas which is quite far away from the main town.

12.7.8 Lord Shiva and the Crocodile Park (Zone 8)

In the overall design we have also proposed a Lord Shiva Park dedicated the Hindu community in the region. The crocodile park has also been relocated upon the suggestion by Phuentsholing Thromde. However, the crocodile park has been redesigned to make it friendlier for both the visitors and the crocodile. These two parks will also promote tourism in the far regions where of Phuentsholing Thromde where the tourists usually do not go.

THE CROCODILE AND SHIV PARK SITE PLAN

SHIV TEMPLE & PARK

BIRD EYE VIEW

LEFT SIDE PERSEPCTIVE FROM PARKING

APPROACH FROM THE ENTRANCE

THE ENTRANCE GATE

THE SHIVA STATUE WITH ELEVATED PLATFORM

THE NANDI AND THE SHIVA STATUE IN AXIS

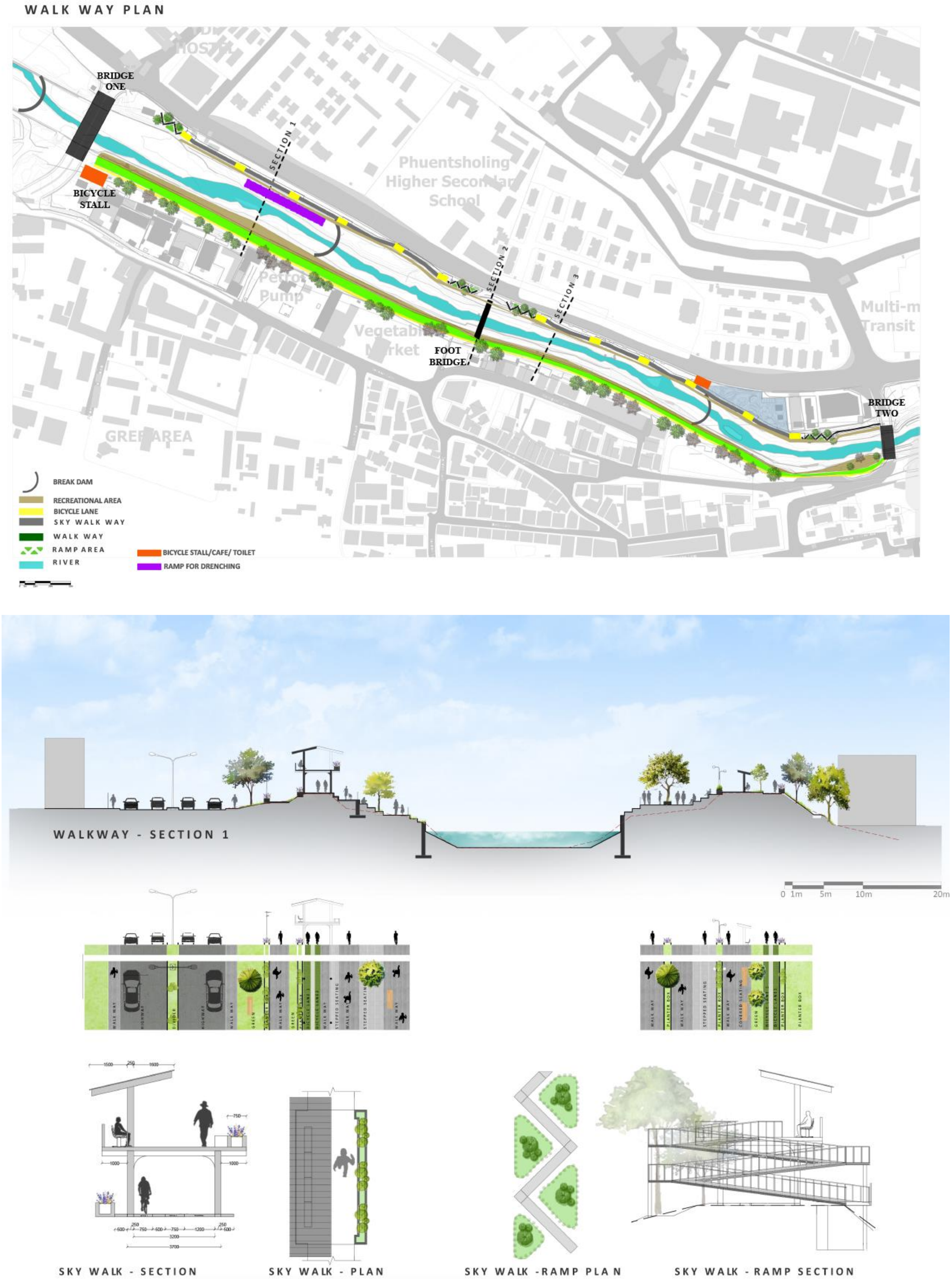
OMCHHU LANDSCAPE

12.7.9 Connectivity in the overall Landscape Design

Connectivity in the overall landscape design is very important to activate all the areas. Hence, we have features like walkways, bicycle lanes, skywalks, trekking and jogging trails along the Omchhu river.

Figure 163: Overall connectivity plan

Figure 164: Walkway and typical section



13 Considerations and Disclaimer

The project of this scale and magnitude requires a through planning, and a robust mechanism to handle the changing profile of river bed, the hydrological conditions, and land use. It also requires the acceptance that changes are inevitable, but execution must be completed in a record speed while at the same time ensuring that technical standards are fully met.

The following considerations (but not limited to) are necessary when implementing the project;

- The planning is based on the current scenarios and data availability. In particular the designs are based on the hydraulic study carried in 2020 by Egis International. The hydraulic report is based on the validation of the hydrological study of 2018 Gyeltshen Consultancy.
- It is likely that the designs may change based on varying ground conditions, therefore, for the current designs to be appropriate and relevant, it is recommended to implement them at the earliest possible.
- The project shall be executed from the upstream to downstream, and to the extent possible at a single stretch. If such a funding mechanism is not possible, then it shall be executed based on package wise from upstream.
- During procurement the client shall consider the procurement method and advance contracting. Other considerations of material stocking, labour camps, and the resources required to execute the construction must be well planned, and ensured that the contractor utilizes the available construction period of 6-7 months in the most efficient delivery method.
- The client may also consider retaining a consulting team as Design and Monitoring Supervision Consultant to ensure that designs are revised and supervision is carried out thoroughly.
- During implementation, certain critical areas need to be dealt with caution, therefore, adequate provisions in the contract shall be made to enable to construction firm to forecast the requirement of safety measures (including construction of scaffolding to retain the existing infrastructure). Such locations have been highlighted in the Infrastructure Design and Drawings.

14 References

National Statistics Bureau (2019) Economic Census of Bhutan 2018-2019

National Center for Hydrology and Meteorology---- (2019). Analysis of Historical Climate and Climate Projection for Bhutan. Available at: https://www.nchm.gov.bt/attachment/ckfinder/userfiles/files/Analysis%20of%20Historical%20Climate%20and%20Climate%20Change%20Projection.pdf

National Environment Commission (2021)/ Deltares- Assessment of climate risks on water resources for the National Adaptation Plan (NAP) in Bhutan. Available at: http://www.nec.gov.bt/publications/climate-change

Japan International Cooperation Agency (2014). Data Collection Survey on Urban Development and Environment in the Kingdom of Bhutan. Available at: https://openjicareport.jica.go.jp/pdf/12148565_01.pdf

Flood Engineering Management Division (nd) Pasakha Flood Mitigation Proposal-Technical Report. Available at: https://procurement-notices.undp.org/view_file.cfm?doc_id=41850

Fonder, N.L., & Xanthoulis, S. (2007). Roman aqueduct and hydraulic engineering: case of Nîmes aqueduct and its Pont du Gard bridge. Water Science & Technology: Water Supply, 7, 121-129.

Chanson, H. (2000). Hydraulics of Roman Aqueducts: Steep Chutes, Cascades, and Dropshafts. American Journal of Archaeology, 104, 47- 72.

Osuagwu, Joachim & Okoro, Bon & Agunwamba, Jonah. (2013). The Use Of Sandbags As Check-Dam For Erosion Control. Conference: International Engineering Conference/Exhibition, Nigerian Society of Engineers.At: International Conference Center Abuja. Available from: https://www.researchgate.net/publication/332767210_THE_USE_OF_SANDBAGS_AS_CHECK-DAM_FOR_EROSION_CONTROL [accessed Jul 29 2022].

Tardio, G., Mickovski, S., Stokes, A., Devkota, S. (2017). Bamboo structures as a resilient erosion control measure. ICE Proceedings. 170. 10.1680/jfoen.16.00033.

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Paz, F.E., Paz, P.F., Almanzor, N., Marcos, M.C.M. (2018). Performance evaluation of soil erosion control geotextile materials on road cut slope. Advances in Civil, Environmental, & Materials Research, Incheon, Korea, August 27- 31, 2018. Available at: http://www.i-asem.org/publication_conf/acem18/2.ICGE18/XH3B.1.GE1168_4750F1.pdf

Gross National Happiness Commission (2013), Eleventh Five Year Plan (July 2013 – June 2018) Local Government Plan – Volume III. Available at: https://www.gnhc.gov.bt/en/wp-content/uploads/2017/05/Eleventh-Five-Year-Plan-phuentsholing-Thromde.pdf

15 Appendix

Appendix 1: Results from hydraulic simulations

The table below shows results of simulation from HEC-RAS for P1, PF2 and PF3 using the geometric section of lowered river bed level with PTDP confluence as most likely situation in the field. It may be noted that the work in the field to dredge upto this level is already on the way by PTDP.

River Sta	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
3480.295	503.2	280.43	285.15	285.15	286.64	0.018384	5.41	92.98	30.96	1
3480.295	582	280.43	285.5	285.5	287.09	0.018098	5.59	104.15	32.61	1
3480.295	660.4	280.43	285.82	285.82	287.5	0.017792	5.75	114.78	33.89	1
3364.436	503.2	275.39	279.8	280.38	282.01	0.028242	7.12	80.06	31.37	1.22
3364.436	582	275.39	280.08	280.7	282.46	0.028362	7.41	88.9	32.51	1.23
3364.436	660.4	275.39	280.3	280.97	282.89	0.029085	7.72	96.33	33.03	1.25
3245.844	503.2	272.74	276.02	276.78	278.49	0.030523	6.96	72.28	42.75	1.71
3245.844	582	272.74	276.21	277.03	278.87	0.029472	7.22	80.59	43.88	1.7
3245.844	660.4	272.74	276.39	277.26	279.22	0.028545	7.45	88.67	44.96	1.69
3145.845	503.2	269.38	273.37	273.64	274.92	0.019893	6.59	98.04	41.61	1.11
3145.845	582	269.38	273.67	273.92	275.28	0.018843	6.76	110.78	43.28	1.09
3145.845	660.4	269.38	273.93	274.17	275.62	0.018208	6.94	122.35	44.45	1.08
3048.008	503.2	266.49	270.91	270.63	271.75	0.008765	4.52	140.99	72.12	0.74
3048.008	582	266.49	270.85	270.84	272.04	0.012585	5.35	136.45	69.91	0.88
3048.008	660.4	266.49	270.83	271.29	272.39	0.016552	6.12	135.13	69.29	1.01
2971.77	503.2	264.22	268.09	268.39	269.95	0.026423	6.24	84.24	32.23	1.2
2971.77	582	264.22	268.39	268.83	270.4	0.024983	6.5	94.06	32.99	1.18
2971.77	660.4	264.22	268.77	269.4	270.79	0.021662	6.54	107.24	43.61	1.12
2864.746	503.2	259.38	262.45	263.44	265.77	0.058387	8.24	64.57	34.43	1.75
2864.746	582	259.38	262.67	263.85	266.25	0.056374	8.6	72.06	35.45	1.75
2864.746	660.4	259.38	262.86	264.11	266.71	0.054901	8.94	79.21	36.51	1.75
2773.891	503.2	254.53	257.28	258.55	261.64	0.093408	9.25	54.39	27.85	2.11
2773.891	582	254.53	257.53	258.88	262.08	0.084988	9.45	61.61	28.27	2.04
2773.891	660.4	254.53	257.78	259.19	262.49	0.077589	9.61	68.71	28.63	1.98
2658.75	503.2	247.84	252.01	253.08	255.63	0.035767	8.94	63.52	21.01	1.48
2658.75	582	247.84	252.36	253.42	256.28	0.034721	9.35	70.88	21.68	1.48
2658.75	660.4	247.84	252.69	254.04	256.87	0.033676	9.7	78.15	22.35	1.48
2522.035	503.2	243.09	247.49	248.08	250.02	0.032782	7.05	71.58	24.34	1.3
2522.035	582	243.09	247.78	248.45	250.58	0.03262	7.42	78.81	24.8	1.31
2522.035	660.4	243.09	248.06	248.79	251.12	0.032459	7.75	85.73	25.16	1.31
2340.688	503.2	238.35	242.65	242.65	243.99	0.020774	5.13	98.79	37.97	1
2340.688	582	238.35	242.94	242.94	244.39	0.019853	5.34	110.08	38.95	0.99
2340.688	660.4	238.35	243.22	243.22	244.77	0.019146	5.53	120.86	39.58	0.99
2308.976	503.2	237.17	240.34	240.96	242.58	0.043113	6.62	76.23	37.81	1.47
2308.976	582	237.17	240.54	241.24	243.02	0.042902	6.98	83.8	38.39	1.48
2308.976	660.4	237.17	240.73	241.51	243.44	0.042739	7.31	91	38.9	1.5
2210.136	503.2	232.95	238.7	238.07	240.14	0.013317	5.32	94.51	21.38	0.81
2210.136	582	232.95	239.15	238.49	240.74	0.013473	5.57	104.41	21.86	0.81
2210.136	660.4	232.95	239.54	238.89	241.28	0.013967	5.85	112.87	22.26	0.83

River Sta	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
2136.716	503.2	231.27	234.91	235.83	238.17	0.044616	8.08	64.01	23.37	1.55
2136.716	582	231.27	235.17	236.19	238.8	0.045324	8.54	70.19	24.16	1.58
2136.716	660.4	231.27	235.41	236.55	239.41	0.044969	8.96	76.05	24.16	1.6
2054.632	503.2	228.53	232.37	233.24	235.4	0.046244	7.71	65.28	24.05	1.49
2054.632	582	228.53	232.64	233.6	235.98	0.046916	8.1	71.87	24.63	1.51
2054.632	660.4	228.53	232.9	233.94	236.54	0.046292	8.45	78.2	25.15	1.52
1956.252	503.2	225.97	233.14	230.25	233.43	0.000935	2.4	210.08	41.79	0.34
1956.252	582	225.97	233.55	230.56	233.88	0.00099	2.56	227.45	42.35	0.35
1956.252	660.4	225.97	233.93	230.85	234.31	0.001042	2.71	243.72	42.86	0.36
1954	Bridge									
1927.11	503.2	225.65	231.67	231.67	233.13	0.019767	5.35	94.1	32.21	1
1927.11	582	225.65	232	232	233.57	0.019267	5.55	104.91	33.28	1
1927.11	660.4	225.65	232.3	232.3	233.98	0.018914	5.73	115.22	34.26	1
1902.756	503.2	224.67	227.38	228.71	231.89	0.102771	9.4	53.52	29.26	2.22
1902.756	582	224.67	227.61	229	232.35	0.096273	9.64	60.36	30.16	2.18
1902.756	660.4	224.67	227.83	229.27	232.78	0.091012	9.85	67.03	31.01	2.14
1870.652	503.2	223.91	227.34	227.84	229.44	0.03305	6.43	78.3	31.99	1.31
1870.652	582	223.91	227.55	228.13	229.94	0.033865	6.84	85.1	32.51	1.34
1870.652	660.4	223.91	227.75	228.44	230.4	0.0343	7.21	91.84	33.02	1.36
1766.37	503.2	221.62	225.35	225.43	226.89	0.018371	5.52	92.83	33.91	1.01
1766.37	582	221.62	225.64	225.75	227.34	0.018056	5.81	102.56	34.8	1.02
1766.37	660.4	221.62	225.9	226.04	227.75	0.017937	6.08	111.62	35.6	1.03
1626.068	503.2	218.85	222.56	222.66	224.08	0.020965	5.47	92.29	34.45	1.05
1626.068	582	218.85	222.82	222.97	224.52	0.020966	5.77	101.4	35.11	1.07
1626.068	660.4	218.85	223.08	223.26	224.92	0.020818	6.02	110.48	35.86	1.07
1522.635	503.2	216.72	220.95	220.86	222.26	0.016246	5.09	99.98	35.47	0.95
1522.635	582	216.72	221.26	221.16	222.69	0.015616	5.31	111.24	36.06	0.94
1522.635	660.4	216.72	221.53	221.43	223.09	0.015654	5.56	120.74	36.68	0.95
1419.299	503.2	214.5	218.99	218.99	220.45	0.017165	5.44	95.67	32.94	0.98
1419.299	582	214.5	219.3	219.3	220.9	0.016851	5.69	105.96	33.44	0.98
1419.299	660.4	214.5	219.58	219.58	221.32	0.016748	5.94	115.51	34	0.98
1313.862	503.2	212.09	216.14	216.45	218.02	0.024394	6.2	84.44	30.79	1.14
1313.862	582	212.09	216.43	216.76	218.48	0.023986	6.49	93.48	31.21	1.14
1313.862	660.4	212.09	216.72	217.08	218.92	0.023399	6.72	102.49	31.64	1.14
1214.642	503.2	210.08	215.76	214.99	216.73	0.00985	4.36	115.7	32.85	0.73
1214.642	582	210.08	216.15	215.34	217.2	0.009508	4.54	128.79	33.92	0.73
1214.642	660.4	210.08	216.52	215.66	217.65	0.009205	4.7	141.56	34.52	0.72
1192.355	503.2	209.67	215.39	214.82	216.48	0.011721	4.62	109.02	30.33	0.78
1192.355	582	209.67	215.75	215.14	216.95	0.011727	4.85	120.03	30.59	0.78
1192.355	660.4	209.67	216.1	215.44	217.4	0.011807	5.05	130.78	31.22	0.79
1083.497	503.2	207.86	213.58	213.23	214.94	0.007012	5.17	97.39	27.82	0.88
1083.497	582	207.86	214.07	213.63	215.46	0.006532	5.22	111.55	29.73	0.86
1083.497	660.4	207.86	214.53	214	215.94	0.006027	5.25	125.75	31.52	0.84
1073	Bridge									

River Sta	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
1005.801	503.2	206.75	211.1	211.48	213.31	0.025896	6.61	77.21	24.59	1.15
1005.801	582	206.75	211.42	211.86	213.86	0.025818	6.95	85.18	25.05	1.16
1005.801	660.4	206.75	211.72	212.22	214.38	0.025766	7.26	92.8	25.48	1.17
985.4417	503.2	206.41	211.02	211.02	212.81	0.017934	6.01	86.66	24.54	0.98
985.4417	582	206.41	211.42	211.42	213.35	0.017426	6.26	96.59	25.09	0.98
985.4417	660.4	206.41	211.81	211.81	213.86	0.017185	6.46	106.58	26.38	0.98
903.8136	503.2	204.88	209.38	209.13	210.93	0.015702	5.71	92.31	24.91	0.94
903.8136	582	204.88	209.84	209.67	211.46	0.016296	5.83	104.48	28.38	0.96
903.8136	660.4	204.88	210.23	210.02	211.93	0.015808	5.96	115.85	29.66	0.96
835.3117	503.2	203.5	208.4	207.97	209.75	0.012673	5.15	98.63	26.58	0.83
835.3117	582	203.5	208.9	208.34	210.31	0.011505	5.27	112.11	27.61	0.8
835.3117	660.4	203.5	209.35	208.69	210.82	0.010627	5.4	125.04	29.91	0.79
769.2536	503.2	202.21	208.14	206.75	209.02	0.005919	4.19	124.12	29.2	0.6
769.2536	582	202.21	208.69	207.15	209.62	0.005498	4.33	140.64	31.29	0.59
769.2536	660.4	202.21	209.19	207.52	210.17	0.0052	4.46	156.89	33.41	0.58
705.7698	503.2	200.92	206.04	206.04	208.1	0.018248	6.4	80.4	19.71	1
705.7698	582	200.92	206.51	206.51	208.73	0.017913	6.63	89.93	20.49	1
705.7698	660.4	200.92	206.9	206.9	209.31	0.017899	6.92	97.86	20.49	1
684.9354	503.2	200.51	204.09	205.05	207.41	0.043965	8.11	63.08	22.87	1.53
684.9354	582	200.51	204.38	205.44	208.03	0.043296	8.51	69.73	23.14	1.53
684.9354	660.4	200.51	204.65	205.84	208.61	0.042714	8.86	76.12	23.41	1.53
592.3548	503.2	198.99	203.3	203.3	204.85	0.019167	5.5	91.42	29.42	1
592.3548	582	198.99	203.64	203.64	205.32	0.018981	5.74	101.42	30.34	1
592.3548	660.4	198.99	203.95	203.95	205.76	0.018465	5.96	110.89	31.17	0.99
557.4137	503.2	198.47	202.93	201.62	203.44	0.004961	3.17	158.97	43.94	0.53
557.4137	582	198.47	203.25	201.88	203.83	0.005116	3.36	173.18	44.57	0.54
557.4137	660.4	198.47	203.55	202.12	204.19	0.005277	3.54	186.61	45.32	0.56
480.1176	503.2	197.34	201.39	201.33	202.61	0.01802	4.91	102.58	39.09	0.97
480.1176	582	197.34	201.63	201.6	203	0.018186	5.19	112.21	39.28	0.98
480.1176	660.4	197.34	201.96	201.95	203.35	0.018783	5.21	126.72	45.09	0.99
450.977	503.2	196.88	201.14	200.84	202.08	0.013114	4.29	117.72	45.08	0.84
450.977	582	196.88	201.47	201.09	202.46	0.012262	4.42	132.5	46.43	0.82
450.977	660.4	196.88	201.8	201.34	202.82	0.011353	4.49	147.99	47.54	0.8
398.9657	503.2	196.14	200.56	200.09	201.45	0.01081	4.2	121.56	41.22	0.77
398.9657	582	196.14	200.89	200.35	201.85	0.010398	4.37	135.5	41.97	0.77
398.9657	660.4	196.14	201.23	200.6	202.24	0.010771	4.47	150.16	45.87	0.78
344.7923	503.2	195.39	199.95	199.39	200.86	0.010442	4.23	120.55	39.22	0.77
344.7923	582	195.39	200.29	199.68	201.27	0.010335	4.4	134.1	40.62	0.77
344.7923	660.4	195.39	200.58	199.97	201.65	0.010165	4.59	145.88	40.62	0.77
323.4169	503.2	195.09	199.17	199.17	200.52	0.018424	5.17	98.38	36.37	1
323.4169	582	195.09	199.47	199.47	200.93	0.018029	5.37	109.6	37.57	1
323.4169	660.4	195.09	199.76	199.76	201.32	0.017693	5.54	120.63	38.95	1
304	503.2	194.98	198.66	198.87	200.23	0.011264	5.54	91.01	36.36	1.12
304	582	194.98	198.92	199.16	200.64	0.011133	5.8	100.46	37.05	1.13

River Sta	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
304	660.4	194.98	199.17	199.41	201.02	0.010936	6.02	109.85	37.96	1.13
289	503.2	194.75	198.29	198.64	200.03	0.013723	5.84	86.82	37.16	1.22
289	582	194.75	198.53	198.91	200.44	0.013539	6.12	95.88	37.82	1.22
289	660.4	194.75	198.77	199.17	200.82	0.013264	6.36	104.74	38.31	1.23
274.0391	503.2	194	197.62	198.21	199.74	0.019325	6.45	77.99	34.63	1.37
274.0391	582	194	197.87	198.48	200.16	0.019238	6.7	86.92	36.27	1.38
274.0391	660.4	194	198.1	198.73	200.55	0.019136	6.92	95.37	37.57	1.39
260.5555	503.2	193.5	197.27	197.85	199.47	0.0198	6.58	76.63	35.54	1.43
260.5555	582	193.5	197.51	198.13	199.89	0.018564	6.84	85.3	35.54	1.41
260.5555	660.4	193.5	197.75	198.39	200.29	0.017472	7.06	93.74	35.54	1.39
245.5503	503.2	193	196.68	197.38	199.13	0.022841	6.93	72.63	33.59	1.5
245.5503	582	193	196.91	197.65	199.55	0.02403	7.2	80.81	36.38	1.54
245.5503	660.4	193	197.12	197.92	199.96	0.023252	7.47	88.42	36.38	1.53
224.8703	503.2	192.74	196.28	196.92	198.63	0.022283	6.79	74.11	34.11	1.47
224.8703	582	192.74	196.54	197.16	199.05	0.02063	7.02	83.01	34.11	1.44
224.8703	660.4	192.74	196.77	197.39	199.47	0.019879	7.28	90.78	34.11	1.42
213.1328	503.2	192.5	194.15	195.26	197.99	0.10843	8.68	57.99	35.76	2.18
213.1328	582	192.5	197.23	195.53	197.83	0.004726	3.42	170.28	36.86	0.51
213.1328	660.4	192.5	197.78	195.8	198.39	0.004268	3.47	190.38	36.92	0.49
210	Bridge									
200.0378	503.2	192.5	196.45	195.17	197.03	0.005823	3.38	149.08	38.55	0.55
200.0378	582	192.5	196.82	195.43	197.47	0.00586	3.56	163.43	38.55	0.55
200.0378	660.4	192.5	197.17	195.69	197.88	0.005908	3.73	176.94	38.55	0.56
184.5447	503.2	192.5	196.3	195.2	196.93	0.006295	3.52	142.97	39.12	0.59
184.5447	582	192.5	196.67	195.47	197.37	0.006087	3.7	157.66	39.12	0.59
184.5447	660.4	192.5	197.03	195.72	197.79	0.005931	3.86	171.47	39.12	0.59
163.9418	503.2	192.29	195.95	195.15	196.76	0.008968	4	126.07	36.82	0.69
163.9418	582	192.29	196.32	195.46	197.2	0.008529	4.17	139.74	36.82	0.68
163.9418	660.4	192.29	196.66	195.72	197.62	0.008232	4.34	152.43	36.82	0.68
152.8975	503.2	191.88	195.94	194.85	196.64	0.00717	3.71	135.77	37.79	0.62
152.8975	582	191.88	196.32	195.13	197.08	0.006874	3.89	150.12	37.79	0.62
152.8975	660.4	191.88	196.67	195.42	197.5	0.006679	4.05	163.41	37.79	0.62
142.3226	503.2	191.83	195.96	194.6	196.54	0.005344	3.37	149.32	37.53	0.54
142.3226	582	191.83	196.34	194.88	196.98	0.00527	3.56	163.66	37.53	0.54
142.3226	660.4	191.83	196.69	195.13	197.4	0.005234	3.74	176.94	37.53	0.55
132.7837	503.2	191.88	195.52	194.84	196.43	0.010524	4.24	118.69	34.41	0.73
132.7837	582	191.88	195.88	195.12	196.88	0.010667	4.42	131.53	36	0.74
132.7837	660.4	191.88	196.24	195.41	197.3	0.010598	4.57	144.57	37.79	0.74
117.9283	503.2	191.39	195.76	194.04	196.2	0.003805	2.96	170.14	40.85	0.46
117.9283	582	191.39	196.14	194.31	196.64	0.003845	3.14	185.82	41.05	0.47
117.9283	660.4	191.39	196.5	194.54	197.06	0.003869	3.3	200.77	41.06	0.48
100.8495	503.2	191.2	194.58	194.58	195.98	0.020074	5.24	96.16	34.42	1
100.8495	582	191.2	194.87	194.87	196.41	0.019459	5.5	105.98	34.42	1
100.8495	660.4	191.2	195.14	195.14	196.82	0.018947	5.74	115.32	34.42	1

River Sta	Q Total (m3/s)	Min Ch El (m)	W.S. Elev (m)	Crit W.S. (m)	E.G. Elev (m)	E.G. Slope (m/m)	Vel Chnl (m/s)	Flow Area (m2)	Top Width (m)	Froude # Chl
60	503.2	190.95	194.87	193.86	195.56	0.003601	3.7	136.16	37.92	0.62
60	582	190.95	195.17	194.14	195.96	0.003749	3.94	147.72	38.14	0.64
60	660.4	190.95	195.47	194.4	196.34	0.003959	4.14	159.65	39.9	0.66
1	503.2	190.51	194.07	194.07	195.4	0.009616	5.11	98.62	37.15	1
1	582	190.51	194.37	194.37	195.8	0.009357	5.29	110.28	38.88	1
1	660.4	190.51	194.62	194.62	196.17	0.009088	5.52	120.08	38.88	1
-109	503.2	190.19	192.94	193.39	194.92	0.016143	6.25	81.2	35.11	1.31
-109	582	190.19	193.21	193.67	195.34	0.015017	6.46	90.89	35.11	1.28
-109	660.4	190.19	193.48	193.94	195.73	0.014087	6.64	100.29	35.11	1.25
-215	503.2	190.08	193.71	192.72	194.35	0.00335	3.54	142.05	40.76	0.61
-215	582	190.08	194	192.97	194.73	0.003452	3.78	154.03	40.76	0.62
-215	660.4	190.08	194.27	193.21	195.09	0.003547	4	165.23	40.76	0.63
-285	503.2	189.76	192.95	192.95	194.18	0.009251	4.91	102.89	41.79	1
-285	582	189.76	193.19	193.19	194.55	0.009087	5.17	113.17	41.79	1
-285	660.4	189.76	193.44	193.44	194.91	0.008849	5.38	123.36	41.79	1
-339	503.2	189.32	191.28	192.02	193.8	0.028086	7.03	71.6	37.73	1.63
-339	582	189.32	191.51	192.28	194.19	0.025966	7.24	80.37	37.78	1.59
-339	660.4	189.32	191.74	192.53	194.55	0.024204	7.43	88.94	37.78	1.55
-426	503.2	188.91	190.83	191.43	193	0.024521	6.53	77.01	40.77	1.52
-426	582	188.91	190.99	191.69	193.45	0.024765	6.94	83.93	40.81	1.54
-426	660.4	188.91	191.16	191.93	193.87	0.024912	7.3	90.52	40.85	1.56
-495	503.2	188.7	191.47	191.47	192.69	0.009503	4.9	102.8	42.44	1
-495	582	188.7	191.72	191.72	193.06	0.0091	5.12	113.82	43.13	1
-495	660.4	188.7	191.96	191.96	193.41	0.008786	5.33	124.22	43.13	1
-818	503.2	187.28	187.95	188.3	189.14	0.049101	4.82	104.47	156.44	1.88
-818	582	187.28	187.99	188.4	189.42	0.055716	5.3	109.77	156.46	2.02
-818	660.4	187.28	188.02	188.51	189.71	0.061709	5.75	114.87	156.49	2.14
-971	503.2	187.15	188.66		188.76	0.00148	1.43	358.23	251.51	0.37
-971	582	187.15	188.8		188.92	0.001448	1.51	394.66	251.81	0.38
-971	660.4	187.15	188.94		189.06	0.001423	1.57	429.02	252.1	0.38
-1029	503.2	187.13	188.66		188.73	0.000982	1.18	427.01	281.05	0.31
-1029	582	187.13	188.81		188.88	0.000967	1.24	468.36	281.11	0.31
-1029	660.4	187.13	188.94		189.03	0.000955	1.3	507.3	281.18	0.31
-1079	503.2	187.11	188.63	187.84	188.71	0.001136	1.27	397.53	262.67	0.33
-1079	582	187.11	188.78	187.91	188.87	0.00112	1.34	435.79	262.73	0.33
-1079	660.4	187.11	188.91	187.98	189.01	0.001108	1.4	471.86	262.79	0.33
-1121	503.2	187.01	188.62	187.76	188.7	0.001	1.22	413.93	260.96	0.31
-1121	582	187.01	188.77	187.83	188.85	0.001	1.3	451.96	261.03	0.31
-1121	660.4	187.01	188.9	187.9	189	0.001001	1.36	487.77	261.11	0.32

Appendix 2: Scour depth calculation results

Q Total	River Chainage	River Width (m)	Silt Factor D50	DL f	Scour Depth	LACEY'S	Blench's Zt (m)	Overall Scour Depth Selected
B(m3/s)	(m)		mm	1.76 Sqrt.D50	1.33(q2/f)1/3	1.25 - 1.50 * DL	1.45	
660.00	-709	62.00	4.80	3.86	3.82	5.72	1.98	2.30
660.00	-638	39.00	4.80	3.86	5.20	6.50	1.98	2.30
660.00	-426	39.00	4.80	3.86	5.20	6.50	1.98	2.30
660.00	-109	39.00	4.80	3.86	5.20	6.50	1.98	2.30
660.00	1	39.00	4.80	3.86	5.20	6.50	1.98	2.30
660.00	60	39.00	4.80	3.86	5.20	6.50	1.94	2.30
660.00	200	40.14	4.80	3.86	5.10	6.37	1.72	2.30
660.00	213	48.25	4.80	3.86	4.51	5.64	1.68	2.30
660.00	246	49.71	4.80	3.86	4.42	5.53	1.79	2.30
660.00	296	45.51	4.80	3.86	4.69	5.86	1.84	2.30
660.00	368	43.54	4.80	3.86	4.83	6.04	1.69	2.30
660.00	399	49.55	4.80	3.86	4.43	6.65	1.92	2.30
660.00	480	40.75	4.80	3.86	5.05	7.57	1.84	2.30
660.00	555	43.43	4.80	3.86	4.84	7.26	2.88	2.30
660.00	592	22.26	4.80	3.86	7.55	11.33	2.85	6.34
660.00	657	22.64	4.80	3.86	7.47	11.20	2.65	6.64
660.00	685	25.25	4.80	3.86	6.94	10.42	2.60	6.25
660.00	706	25.85	4.80	3.86	6.84	10.25	2.60	6.52
660.00	732	25.92	4.80	3.86	6.82	10.24	2.21	6.83
660.00	756	33.03	4.80	3.86	5.81	8.71	2.24	4.94
660.00	769	32.35	4.80	3.86	5.89	8.83	2.16	4.86
660.00	790	34.31	4.80	3.86	5.66	8.49	2.20	4.58
660.00	812	33.22	4.80	3.86	5.78	8.68	2.31	2.30
660.00	825	30.92	4.80	3.86	6.07	9.10	2.35	2.30
660.00	835	30.21	4.80	3.86	6.16	9.24	2.39	4.52
660.00	847	29.44	4.80	3.86	6.27	9.40	2.44	5.12
660.00	858	28.53	4.80	3.86	6.40	9.60	2.42	5.16
660.00	869	28.94	4.80	3.86	6.34	9.51	2.38	4.71
660.00	880	29.54	4.80	3.86	6.25	9.38	2.39	4.50
660.00	892	29.48	4.80	3.86	6.26	7.83	2.38	3.30
660.00	904	29.62	4.80	3.86	6.24	7.80	2.40	4.02
660.00	911	29.29	4.80	3.86	6.29	7.86	2.47	4.37
660.00	926	28.04	4.80	3.86	6.48	8.09	2.36	3.55
660.00	943	30.01	4.80	3.86	6.19	7.74	2.39	2.90
660.00	958	29.37	4.80	3.86	6.28	7.85	2.57	3.40
660.00	985	26.40	4.80	3.86	6.74	10.11	2.58	4.22
660.00	1,006	26.21	4.80	3.86	6.77	10.16	2.66	4.83
660.00	1,028	25.02	4.80	3.86	6.99	10.48	2.59	5.78
660.00	1,050	26.13	4.80	3.86	6.79	8.48	2.26	5.30
660.00	1,083	31.90	4.80	3.86	5.94	7.43	2.32	3.78
660.00	1,114	30.66	4.80	3.86	6.10	7.63	2.31	3.84
660.00	1,151	30.94	4.80	3.86	6.06	9.10	2.28	2.60
660.00	1,169	31.55	4.80	3.86	5.99	8.98	2.19	2.30
660.00	1,192	33.58	4.80	3.86	5.74	8.61	2.15	2.30
660.00	1,215	34.52	4.80	3.86	5.64	8.46	2.11	2.30
660.00	1,234	35.53	4.80	3.86	5.53	8.30	2.10	2.30
660.00	1,249	35.60	4.80	3.86	5.52	8.28	2.09	5.31
660.00	1,268	35.89	4.80	3.86	5.49	8.24	2.17	4.59
660.00	1,289	34.02	4.80	3.86	5.69	8.54	2.24	4.89
660.00	1,314	32.46	4.80	3.86	5.87	8.81	2.21	4.97
660.00	1,336	33.13	4.80	3.86	5.79	7.24	2.14	3.22
660.00	1,359	34.77	4.80	3.86	5.61	7.01	2.18	2.75
660.00	1,394	33.67	4.80	3.86	5.73	7.16	2.17	2.80
660.00	1,419	34.00	4.80	3.86	5.69	7.12	2.09	2.50
660.00	1,446	35.89	4.80	3.86	5.49	6.87	2.07	2.30
660.00	1,471	36.53	4.80	3.86	5.43	6.79	2.02	2.30
660.00	1,492	37.77	4.80	3.86	5.31	6.64	2.06	2.30
660.00	1,523	36.67	4.80	3.86	5.41	6.77	2.05	2.30
660.00	1,554	37.03	4.80	3.86	5.38	6.72	1.92	2.30
660.00	1,575	40.96	4.80	3.86	5.03	6.29	2.06	2.30
660.00	1,605	36.74	4.80	3.86	5.41	6.76	2.07	2.91

Q Total	River Chainage	River Width (m)	Silt Factor D50	DL f	Scour Depth	LACEY'S	Blench's Zt (m)	Overall Scour Depth Selected
B(m3/s)	(m)		mm	1.76 Sqrt.D50	1.33(q2/f)1/3	1.25 - 1.50 * DL	1.45	
660.00	1,626	36.50	4.80	3.86	5.43	6.79	2.07	2.69
660.00	1,650	36.51	4.80	3.86	5.43	6.79	2.05	2.30
660.00	1,685	36.89	4.80	3.86	5.39	6.74	2.07	2.30
660.00	1,712	36.56	4.80	3.86	5.43	6.78	2.08	2.30
660.00	1,740	36.23	4.80	3.86	5.46	6.82	2.09	2.30
660.00	1,766	36.05	4.80	3.86	5.48	6.85	2.12	2.30
660.00	1,792	35.13	4.80	3.86	5.57	6.97	2.13	2.30
660.00	1,816	35.00	4.80	3.86	5.59	6.98	2.11	2.30
660.00	1,843	35.40	4.80	3.86	5.54	6.93	2.10	2.30
660.00	1,871	35.73	4.80	3.86	5.51	6.89	2.09	2.30
660.00	1,903	35.98	4.80	3.86	5.48	6.85	2.08	2.30
660.00	1,927	36.14	4.80	3.86	5.47	6.83	2.04	2.30
660.00	1,956	37.19	4.80	3.86	5.36	6.71	2.06	2.92
660.00	1,981	36.64	4.80	3.86	5.42	6.77	2.06	3.06
660.00	1,998	36.80	4.80	3.86	5.40	6.75	2.03	3.12
660.00	2,021	37.49	4.80	3.86	5.34	6.67	2.15	2.30
660.00	2,041	34.56	4.80	3.86	5.63	7.04	2.18	3.17
660.00	2,055	33.71	4.80	3.86	5.73	7.16	2.19	2.51
660.00	2,066	33.59	4.80	3.86	5.74	7.18	2.21	3.60
660.00	2,079	33.05	4.80	3.86	5.80	7.25	2.21	3.69
660.00	2,092	33.07	4.80	3.86	5.80	7.25	2.26	3.33
660.00	2,105	32.03	4.80	3.86	5.93	7.41	2.08	3.01
660.00	2,122	36.15	4.80	3.86	5.47	8.20	2.30	3.09
660.00	2,137	31.10	4.80	3.86	6.04	9.07	2.41	3.02
660.00	2,151	28.97	4.80	3.86	6.34	9.50	2.10	2.91
660.00	2,159	35.61	4.80	3.86	5.52	8.28	2.72	2.30
660.00	2,168	24.22	4.80	3.86	7.14	8.92	2.75	2.30
660.00	2,177	23.81	4.80	3.86	7.22	9.03	2.54	3.01
660.00	2,187	26.91	4.80	3.86	6.66	8.32	2.86	3.56
660.00	2,198	22.49	4.80	3.86	7.50	9.38	2.57	4.52
660.00	2,210	26.42	4.80	3.86	6.74	8.42	1.68	3.68
660.00	2,228	49.80	4.80	3.86	4.42	5.52	1.55	2.30
660.00	2,242	56.32	4.80	3.86	4.07	5.08	1.67	2.30
660.00	2,257	50.17	4.80	3.86	4.39	5.49	1.77	2.30
660.00	2,275	46.15	4.80	3.86	4.65	6.97	2.01	3.06
660.00	2,289	38.21	4.80	3.86	5.27	6.59	1.91	2.30
660.00	2,297	40.85	5.00	3.94	4.97	6.21	1.91	2.30
660.00	2,309	40.79	5.00	3.94	4.98	6.22	1.95	2.30
660.00	2,319	39.54	5.00	3.94	5.08	6.35	2.16	2.30
660.00	2,328	33.94	5.00	3.94	5.62	7.03	1.93	2.30
660.00	2,337	40.39	5.00	3.94	5.01	6.26	1.95	2.30
660.00	2,341	39.58	5.00	3.94	5.08	6.35	2.01	2.30
660.00	2,349	37.74	5.00	3.94	5.24	6.55	2.10	2.32
660.00	2,355	35.57	5.00	3.94	5.45	6.81	2.21	2.49
660.00	2,362	32.92	5.00	3.94	5.74	7.18	2.30	2.87
660.00	2,377	30.94	5.00	3.94	5.98	7.48	2.43	3.25
660.00	2,390	28.54	5.00	3.94	6.31	7.89	2.52	3.72
660.00	2,410	26.97	5.00	3.94	6.56	8.19	2.28	4.05
660.00	2,431	31.32	5.00	3.94	5.93	7.42	2.58	3.37
660.00	2,448	26.02	5.00	3.94	6.71	8.39	2.50	4.39
660.00	2,470	27.27	5.00	3.94	6.51	8.13	2.57	3.71
660.00	2,498	26.22	5.00	3.94	6.68	8.35	2.58	4.08
660.00	2,510	25.98	5.00	3.94	6.72	8.40	2.58	4.06
660.00	2,522	26.08	5.00	3.94	6.70	8.38	2.57	4.22
660.00	2,534	26.25	5.00	3.94	6.68	8.34	2.57	3.96
660.00	2,545	26.15	5.00	3.94	6.69	8.37	2.65	4.25
660.00	2,562	25.08	5.00	3.94	6.88	8.60	2.40	4.17
660.00	2,578	29.00	5.00	3.94	6.25	7.81	2.50	3.60
660.00	2,590	27.29	5.00	3.94	6.50	8.13	2.73	3.26
660.00	2,603	23.91	5.00	3.94	7.10	8.88	2.65	4.16
660.00	2,618	25.04	5.00	3.94	6.89	8.61	2.68	3.80
660.00	2,630	24.66	5.00	3.94	6.96	8.70	2.56	3.62
660.00	2,644	26.38	5.00	3.94	6.65	8.32	2.51	3.68
660.00	2,659	27.13	5.00	3.94	6.53	8.16	2.47	3.06

Q Total	River Chainage	River Width (m)	Silt Factor D50	DL f	Scour Depth	LACEY'S	Blench's Zt (m)	Overall Scour Depth Selected
B(m3/s)	(m)		mm	1.76 Sqrt.D50	1.33(q2/f)1/3	1.25 - 1.50 * DL	1.45	
660.00	2,677	27.75	5.00	3.94	6.43	8.04	2.38	3.34
60.00	2,692	29.31	5.00	3.94	6.20	7.75	2.10	3.08
660.00	2,705	35.59	5.00	3.94	5.45	6.81	1.94	2.30
660.00	2,712	39.86	5.00	3.94	5.05	6.32	1.53	2.30
660.00	2,720	56.80	5.00	3.94	3.99	4.99	1.72	2.30
660.00	2,733	47.75	5.00	3.94	4.48	5.60	1.80	2.30
660.00	2,747	44.62	5.00	3.94	4.69	5.86	1.74	2.30
660.00	2,761	46.91	5.00	3.94	4.53	5.67	1.98	2.30
660.00	2,774	38.71	5.00	3.94	5.15	6.44	2.82	2.30
660.00	2,789	22.75	5.00	3.94	7.34	9.18	1.89	2.67
660.00	2,801	41.43	5.00	3.94	4.92	6.16	1.95	2.30
660.00	2,815	39.59	5.00	3.94	5.08	6.34	1.96	2.30
660.00	2,826	39.32	5.00	3.94	5.10	6.37	1.75	2.30
660.00	2,836	46.69	5.00	3.94	4.55	5.68	1.64	2.30
660.00	2,844	51.25	5.00	3.94	4.27	5.34	1.61	2.30
660.00	2,854	52.89	5.00	3.94	4.18	5.23	1.79	2.30
660.00	2,865	45.04	5.00	3.94	4.66	5.82	1.86	2.30
660.00	2,876	42.63	5.00	3.94	4.83	6.04	1.59	2.30
660.00	2,889	53.83	5.00	3.94	4.14	5.17	1.83	2.30
660.00	2,899	43.73	5.00	3.94	4.75	5.94	1.67	2.30
660.00	2,904	49.96	5.00	3.94	4.35	5.43	1.86	2.30
660.00	2,910	42.45	5.00	3.94	4.85	6.06	1.78	2.30
660.00	2,926	45.27	5.00	3.94	4.64	5.80	1.73	2.30
660.00	2,934	47.28	5.00	3.94	4.51	5.64	1.62	2.30
660.00	2,944	52.16	5.00	3.94	4.22	5.28	2.03	2.30
660.00	2,957	37.35	5.00	3.94	5.28	6.60	1.38	1.09
660.00	2,972	66.53	5.00	3.94	3.59	4.49	1.47	2.30
660.00	2,982	60.34	5.00	3.94	3.83	4.79	1.49	2.30
660.00	2,991	59.53	5.00	3.94	3.87	4.83	1.27	2.30
660.00	3,001	75.79	5.00	3.94	3.29	4.12	1.60	2.30
660.00	3,013	53.24	5.00	3.94	4.17	5.21	1.58	2.30
660.00	3,025	54.47	5.00	3.94	4.10	5.13	1.30	2.30
660.00	3,037	72.70	5.00	3.94	3.38	4.23	1.27	2.30
660.00	3,048	75.62	5.00	3.94	3.30	4.12	1.23	2.30
660.00	3,068	79.57	5.00	3.94	3.19	3.98	1.41	2.30
660.00	3,078	64.42	5.00	3.94	3.67	4.59	1.50	2.30
660.00	3,089	58.86	5.00	3.94	3.90	4.87	1.60	2.30
660.00	3,102	53.17	5.00	3.94	4.17	5.21	1.69	2.30
660.00	3,120	49.27	5.00	3.94	4.39	5.48	1.73	2.30
660.00	3,139	47.47	5.00	3.94	4.50	5.62	1.77	2.30
660.00	3,146	45.82	5.00	3.94	4.60	5.76	1.77	2.30
660.00	3,152	45.88	5.00	3.94	4.60	5.75	1.66	2.30
660.00	3,158	50.50	5.00	3.94	4.32	5.39	1.62	2.30
660.00	3,173	52.15	5.00	3.94	4.22	5.28	1.62	2.30
660.00	3,184	52.23	5.00	3.94	4.22	5.27	1.44	2.30
660.00	3,210	62.37	5.00	3.94	3.75	4.69	1.68	2.30
660.00	3,246	49.77	5.00	3.94	4.36	5.45	1.91	2.30
660.00	3,278	41.05	5.00	3.94	4.95	6.19	1.96	2.30
660.00	3,312	39.27	5.00	3.94	5.10	6.38	2.03	2.30
660.00	3,341	37.38	5.00	3.94	5.27	6.59	2.14	2.30
660.00	3,364	34.51	5.00	3.94	5.56	6.95	2.21	2.30
660.00	3,382	32.83	5.00	3.94	5.75	7.19	2.15	2.86
660.00	3,405	34.25	5.00	3.94	5.59	6.99	2.38	2.94
660.00	3,422	29.43	5.00	3.94	6.19	7.73	2.44	3.35
660.00	3,439	28.31	5.00	3.94	6.35	7.93	2.38	3.71
660.00	3,461	29.49	5.00	3.94	6.18	7.72	2.30	3.53
660.00	3,480	30.95	5.00	3.94	5.98	7.48	2.3	2.99

Appendix 3: Design Considerations and Calculations

Centerline Chainge Revision DPR	Chainage as per Rev DPR, RB	Chainage as per Rev DPR, LB	Scour Depth	Design Scour Depth	Design discharge	Velocity (m)	Velocity for Design (m)	Channel Width (2022)	Height Required	Height Required with FB (1.5m)	Height Required with FB+Backflow	Final Height Required	Design Height	Bed Level	RB Top Level	LB Top Level	Wall Length Right Bank	Wall Length Left Bank
200	200-220	200-220	2.30	2.30	660.40	7.28	9.10	43.64	1.66	3.16		3.16	4.50	193.17	199.00	199.18	No Wall	No Wall
220	220-240	220-240	2.30	2.30	660.40	7.28	9.10	38.31	1.89	3.39		3.39	4.50	193.17	199.00	199.54	21	19
240	240-260	240-260	2.30	2.30	660.40	7.47	9.34	37.31	1.90	3.40		3.40	4.50	193.28	199.00	199.89	20.0	20.0
260	260-280	260-280	2.30	2.30	660.40	6.92	8.65	37.30	2.05	3.55		3.55	4.50	193.67	199.00	200.25	20.1	20.0
280	280-300	280-300	2.30	2.30	660.40	6.36	7.95	35.36	2.35	3.85		3.85	4.50	193.73	200.00	200.61	20.0	20.0
300	300-330	300-320	2.30	2.30	660.40	6.02	7.53	33.13	2.65	4.15	2.00	6.15	6.50	193.88	200.40	200.97	28.0	20.0
310	310	310		0.00				33.13						193.88			Check Dam	
330	330-340	320-340	2.30	2.30	660.40	5.54	6.93	31.62	3.02	4.52	1.60	6.11	6.50	194.28			Ramp	21.2
340	340-360	340-360	2.30	2.30	660.40	4.59	5.74	30.00	3.84	5.34	1.60	6.94	7.50	194.27	201.00	201.69	20.1	20.0
360	360-380	360-380	2.30	2.30	660.40	4.47	5.59	27.13	4.36	5.86	1.37	7.22	7.50	194.51	201.40	202.04	20.1	20.0
380	380-400	380-400	2.30	2.30	660.40	4.47	5.59	26.10	4.53	6.03	1.32	7.35	7.50	194.56	202.00	202.40	20.1	20.0
400	400-420	400-420	2.30	2.30	660.40	4.47	5.59	25.24	4.68	6.18	1.40	7.58	7.50	194.48	203.00	202.76	21.7	17.4
420	420-440	420-440	2.30	2.30	660.40	4.49	5.61	24.76	4.75	6.25	1.48	7.73	7.50	194.39	203.50	203.12	19.7	20.5
440	440-460	440-460	2.30	2.30	660.40	4.49	5.61	23.31	5.05	6.55	1.20	7.75	7.50	194.68	204.00	203.70	20.1	20.0
460	460-480	460-480	2.30	2.30	660.40	5.21	6.51	20.59	4.92	6.42	-0.12	6.30	7.50	196.00	204.40	204.28	19.4	20.9
480	480-500	480-500	2.30	2.30	660.40	5.21	6.51	21.14	4.80	6.30	0.27	6.56	7.50	195.61	204.70	204.86	20.0	20.0
500	500-520	500-520	2.30	2.30	660.40	5.21	6.51	21.29	4.76	6.26	0.06	6.32	7.50	195.82	205.00	205.29	19.8	20.3
510	510	510		0.00				21.29						195.82			Check Dam	
520	520-540	520-540	2.30	2.30	660.40	3.54	6.51	21.59	4.70	6.20	1.16	7.35	7.50	196.16	205.50	205.73	20.0	20.0
540	540-560	540-560	2.30	2.30	660.40	3.54	6.51	22.43	4.52	6.02	0.95	6.97	7.50	196.36	206.00	206.16	20.0	20.0
560	560-580	560-580	2.30	2.30	660.40	3.54	6.51	22.67	4.47	5.97	0.08	6.05	7.50	197.24	206.30	206.60	20.0	20.1
580	580-600	580-600	2.30	2.30	660.40	5.96	7.45	21.30	4.16	5.66		5.66	7.50	197.78	206.60	207.03	17.5	21.5
600	600-620	600-620	6.90	4.50	660.40	5.96	7.15	18.94	4.88	6.38		6.38	7.50	198.16	206.80	207.03	20.1	20.0
620	620-637	620-637	6.90	4.50	660.40	5.96	7.15	18.13	5.09	6.59		6.59	7.50	197.98	207.00	207.64	Bridge	Bridge
637	637-660	637-660	6.90	4.50	660.40	8.86	10.63	17.60	3.53	5.03		5.03	6.50	199.08			23.7	23.5
660	660-680	660-680	6.90	4.50	660.40	8.86	10.63	17.67	3.52	5.02		5.02	6.50	199.29	207.30	208.95	20.0	20.0
680	680-700	680-700	6.90	4.50	660.40	8.86	10.63	17.67	3.52	5.02		5.02	6.50	199.14	207.50	209.20	20.0	20.0
700	700-720	700-720	6.90	4.50	660.40	6.92	8.65	17.49	4.37	5.87		5.87	6.50	199.78	207.40	209.50	20.0	20.0
720	720-740	720-740	6.90	4.50	660.40	6.92	8.65	18.60	4.10	5.60		5.60	6.50	200.03	207.70	209.79	20.0	20.0
740	740-760	740-760	6.90	4.50	660.40	4.46	6.75	19.53	5.01	6.51		6.51	6.50	200.32	207.90	210.37	20.0	20.1
760	760	760		0.00				21.63						200.29	208.00	210.96	Check Dam	
760	760-780	760-780	5.80	4.50	660.40	4.46	6.75	21.63	4.52	6.02	2.00	8.02	7.50	200.29	208.00	210.96	20.0	20.1
780	780-790	780-800	5.80	4.50	660.40	4.46	6.75	22.55	4.34	5.84	1.29	7.13	7.50	201.00	208.00	211.54	Ramp	19.8
790	790-820	800-820	5.80	4.50	660.40	5.40	6.75	22.97	4.26	5.76		5.76	6.50	202.44			32.1	17.2
820	820-840	820-840	5.80	4.50	660.40	5.40	6.75	22.02	4.44	5.94		5.94	6.50	203.23	209.00	212.18	20.1	20.1
840	840-860	840-860	5.80	4.50	660.40	5.40	6.75	21.53	4.54	6.04		6.04	6.50	203.72	210.00	212.71	20.1	20.1
850	850	850		0.00				21.53						203.72			Check Dam	

860	860-880	860-880	5.80	4.50	660.40	5.96	7.45	21.77	4.07	5.57	2.00	7.57	7.50	204.18	210.00	212.90	20.0	20.0
880	880-900	880-900	5.80	4.50	660.40	5.96	7.45	21.22	4.18	5.68	1.77	7.45	7.50	204.87	211.00	213.30	20.0	20.0
900	900-920	900-920	5.80	4.50	660.40	5.96	7.45	19.58	4.53	6.03	0.98	7.01	7.50	205.64	212.00	213.54	20.0	20.0
920	920-940	920-940	5.80	4.50	660.40	5.96	7.45	19.20	4.62	6.12	0.66	6.78	7.50	206.00	213.00	214.00	20.1	20.1
940	940-960	940-960	5.80	4.50	660.40	6.46	8.08	18.56	4.41	5.91	0.40	6.31	6.50	205.91	213.00	215.00	14.7	24.6
960	960-980	960-980	5.80	4.50	660.40	6.46	8.08	20.28	4.03	5.53	0.59	6.13	6.50	206.13	214.00	216.00	20.0	20.1
980	980-1000	980-1000	5.80	4.50	660.40	7.26	9.08	18.93	3.84	5.34	0.59	5.94	6.50	206.92	214.00	217.00	17.1	22.4
1000	1000-1020	1000-1020	5.80	4.50	660.40	7.26	9.08	18.37	3.96	5.46	0.08	5.54	6.50	207.24	215.00	218.00	20.0	20.0
1020	1020-1040	1020-1040	5.80	4.50	660.40	7.26	8.71	16.76	4.52	6.02		6.02	6.50	207.51	215.00	218.00	20.0	20.2
1040	1040-1055	1040-1055	5.80	4.50	660.40	5.25	6.30	18.37	5.71	7.21		7.21	7.50	207.31	0.00	0.00	Bridge	Bridge
1055	1055-1060	1055-1060	5.80	4.50	660.40	5.25	6.30	17.62	5.95	7.45		7.45	7.50	207.31	0.00	0.00	Bridge	Bridge
1060	1060-1070	1060-1070	5.80	4.50	660.40	5.25	6.30	18.37	5.71	7.21		7.21	7.50	207.82	0.00	0.00	Bridge	Bridge
1070	1070-1100	1070-1100	3.85	3.85	660.40	5.25	6.30	18.26	5.74	7.24		7.24	7.50	207.94			31.2	28.9
1100	1100-1120	1100-1120	3.85	3.85	660.40	5.25	6.30	18.36	5.71	7.21		7.21	7.50	208.78	217.00	219.40	20.0	20.1
1120	1120-1140	1120-1140	3.85	3.85	660.40	5.05	6.31	18.23	5.74	7.24		7.24	7.50	208.72	217.00	219.70	20.1	20.0
1140	1140-1160	1140-1160	2.70	2.70	660.40	5.05	6.31	19.27	5.43	6.93		6.93	7.00	209.32	217.00	220.00	20.4	20.0
1160	1160-1180	1160-1180	2.70	2.70	660.40	5.05	6.31	21.39	4.89	6.39		6.39	6.50	210.00	219.00	220.50	20.0	20.0
1180	1180-1200	1180-1200	2.70	2.70	660.40	5.05	6.31	23.48	4.46	5.96		5.96	6.50	211.00	220.50	221.00	20.7	19.7
1200	1200-1220	1200-1220	2.70	2.70	660.40	4.70	8.40	25.59	3.07	4.57	2.00	6.57	6.00	211.06	220.00	221.40	20.0	20.0
1200	1200	1200		0.00				25.59						211.06	220.00	221.40	Check Dam	
1220	1220-1230	1220-1240	2.70	2.70	660.40	4.70	8.40	27.09	2.90	4.40	1.79	6.19	6.50	211.27	221.00	221.70	Ramp	18.18
1230	1230-1260	1240-260	5.50	4.50	660.40	0.00	8.40	25.57	3.07	4.57	1.30	5.87	6.50	211.77			30.1	20.0
1260	1260-1280	1260-1280	5.50	4.50	660.40	6.72	8.40	24.74	3.18	4.68	0.78	5.46	5.50	212.28	222.00	222.50	20.0	20.0
1280	1280-1300	1280-1300	5.50	4.50	660.40	6.72	8.40	23.83	3.30	4.80	0.06	4.86	5.50	213.00	222.00	223.00	21.1	19.0
1300	1300-1320	1300-1320	5.50	4.50	660.40	6.72	8.40	22.96	3.42	4.92	0.06	4.99	5.50	213.00	223.00	223.40	20.0	20.0
1320	1320-1340	1320-1340	5.50	4.50	660.40	6.72	8.40	23.42	3.36	4.86	0.06	4.92	5.50	213.00	224.00	223.70	20.1	20.1
1340	1340-1360	1340-1360	2.80	2.80	660.40	6.72	8.40	23.03	3.41	4.91	0.37	5.29	6.00	212.69	224.00	224.00	20.0	20.1
1360	1360-1380	1360-1380	2.80	2.80	660.40	5.94	7.43	19.59	4.54	6.04		6.04	6.00	213.50	224.00	224.50	20.3	19.7
1390	1390	1390		0.00				20.54						214.48			Check Dam	
1380	1380-1400	1380-1400	2.80	2.80	660.40	5.94	7.43	20.54	4.33	5.83	2.00	7.83	7.50	214.48	224.30	225.00	20.0	20.0
1400	1400-1420	1400-1420	2.80	2.80	660.40	5.94	7.43	20.10	4.43	5.93	1.48	7.40	7.50	215.00	225.00	225.50	20.0	20.0
1420	1420-1440	1420-1440	2.80	2.80	660.40	5.94	7.43	19.57	4.54	6.04	0.48	6.52	7.50	216.00	226.00	226.00	21.5	18.6
1440	1440-1460	1440-1460	2.30	2.30	660.40	5.94	7.43	17.31	5.14	6.64	1.29	7.93	7.50	215.19	226.00	226.40	20.8	19.3
1460	1460-1480	1460-1480	2.30	2.30	660.40	5.56	6.95	20.86	4.56	6.06	0.84	6.90	7.50	215.63	226.00	226.70	20.0	20.1
1480	1480-1500	1480-1500	2.30	2.30	660.40	5.56	6.95	23.87	3.98	5.48	0.24	5.72	7.50	216.24	227.00	227.00	16.2	22.9
1500	1500-1520	1500-1520	2.30	2.30	660.40	5.56	6.95	24.26	3.92	5.42		5.42	6.00	216.60	227.00	228.00	20.0	20.1
1520	1520-1540	1520-1540	2.30	2.30	660.40	5.56	6.95	26.37	3.60	5.10		5.10	6.00	217.45	228.00	230.00	20.0	20.0
1540	1540-1560	1540-1560	2.30	2.30	660.40	5.56	6.95	25.81	3.68	5.18		5.18	6.00	218.00	228.00	230.00	20.0	20.0
1560	1560-1580	1560-1580	2.30	2.30	660.40	6.02	7.53	24.01	3.66	5.16		5.16	6.00	218.42	229.00	230.00	20.0	20.1
1580	1580-1600	1580-1600	2.30	2.30	660.40	6.02	7.53	25.21	3.48	4.98		4.98	5.00	219.00	230.00	230.00	20.0	20.1
1600	1600-1620	1600-1620	2.30	2.30	660.40	6.02	7.53	26.76	3.28	4.78		4.78	5.00	218.21	230.00	230.00	18.9	21.2

1620	1620-1240	1620-1240	2.30	2.30	660.40	6.02	7.53	28.40	3.09	4.59		4.59	5.00	219.46	231.00	230.00	19.5	20.6
1640	1640-1660	1640-1660	2.30	2.30	660.40	6.02	7.53	26.68	3.29	4.79		4.79	5.00	220.13	231.00	230.00	20.0	20.0
1660	1660-1680	1660-1680	2.30	2.30	660.40	6.02	7.53	27.19	3.23	4.73		4.73	5.00	221.00	232.00	230.00	19.5	20.4
1680	1680-1700	1680-1700	2.30	2.30	660.40	6.08	7.60	27.01	3.22	4.72		4.72	5.00	221.00	232.00	230.00	19.0	21.0
1700	1700-1720	1700-1720	2.30	2.30	660.40	6.08	7.60	23.98	3.62	5.12		5.12	5.50	220.86	232.00	230.00	20.2	20.0
1720	1720-1740	1720-1740	2.30	2.30	660.40	6.08	7.60	20.74	4.19	5.69		5.69	6.00	221.29	234.00	230.00	20.0	20.0
1740	1740-1760	1740-1760	2.30	2.30	660.40	6.08	7.60	20.16	4.31	5.81		5.81	6.00	221.28	234.00	233.00	20.1	20.0
1760	1760-1780	1760-1780	2.30	2.30	660.40	6.08	7.60	21.41	4.06	5.56		5.56	6.00	222.53	235.00	233.40	20.0	20.0
1780	1780	1780		0.00				21.85						223.00	235.00	233.70	Check Dam	
1780	1780-1800	1780-1800	2.30	2.30	660.40	6.08	7.60	21.85	3.98	5.48	2.00	7.48	7.50	223.00	235.00	234.00	20.0	20.0
1800	1800-1810	1800-1820	2.30	2.30	660.40	7.21	9.01	24.19	3.03	4.53	1.00	5.53	6.00	224.00	235.00	234.00	Ramp	18.52
1810	1810-1840	1820-1840	2.30	2.30	660.40	7.21	9.01	24.69	2.97	4.47	1.64	6.11	6.00	223.36	235.00	234.30	32.2	19.7
1840	1840-1860	1840-1860	2.30	2.30	660.40	7.21	9.01	24.51	2.99	4.49	1.00	5.49	5.50	224.00			18.1	21.3
1860	1860-1880	1860-1880	2.30	2.30	660.40	7.21	9.01	25.82	2.84	4.34	0.87	5.21	5.50	224.13	235.00	235.00	19.4	20.7
1880	1880-1900	1880-1900	2.30	2.30	660.40	9.85	12.31	31.06	1.73	3.23		3.23	5.50	225.03	235.00	233.00	12.2	23.7
1900	1900-1920	1900-1920	2.30	2.30	660.40	5.73	7.16	31.17	2.96	4.46		4.46	5.50	225.02	235.00	233.00	Bridge	Bridge
1920	1920-1930	1920-1930	2.30	2.30	660.40	2.71	3.66	32.22	5.60	7.10		7.10	7.50	225.15	235.00	233.00	Bridge	Bridge
1930	1930-1940	1930-1940	2.30	2.30	660.40	2.71	3.66	32.57	5.54	7.04		7.04	7.50	225.85	0.00	0.00	Bridge	Bridge
1940	1940-1960	1940-1960	3.70	3.70	660.40	2.71	3.66	32.57	5.54	7.04		7.04	7.50	226.83	0.00	0.00	20.0	ext wall
1960	1960-1980	1960-1980	3.70	3.70	660.40	2.71	3.66	30.81	5.86	7.36		7.36	7.50	227.65	0.00	0.00	20.1	19.9
1980	1980-2000	1980-2000	3.70	3.70	660.40	8.45	10.56	31.74	1.97	3.47		3.47	4.00	228.00	237.00	239.00	21.4	17.6
2000	2000-2020	2000-2020	3.70	3.70	660.40	8.45	10.56	25.44	2.46	3.96		3.96	4.00	227.78	237.00	240.00	20.0	22.4
2020	2020-2040	2020-2037	3.70	3.70	660.40	8.45	10.56	25.44	2.46	3.96		3.96	4.00	228.00	238.00	241.00	22.6	Stream
2040	2040-2060	2040-2060	3.70	3.70	660.40	8.45	10.56	21.09	2.96	4.46		4.46	4.00	228.93	238.00	243.00	16.7	25.9
2060	2060-2080	2060-2080	3.70	3.70	660.40	8.45	10.56	30.35	2.06	3.56		3.56	4.00	230.00	238.00	243.50	14.5	24.0
2080	2080-2100	2080-2100	3.70	3.70	660.40	8.96	11.20	31.34	1.88	3.38		3.38	4.00	230.40	239.00	244.00	9.3	26.3
2100	2100-2130	2100-2120	3.70	3.70	660.40	8.96	11.20	31.34	1.88	3.38		3.38	4.00	231.20	239.50	243.00	28.0	21.2
2130	2130-2140	2120-2140	3.70	3.70	660.40	8.96	11.20	23.01	2.56	4.06		4.06	4.50	232.00	241.00	241.00	Ramp	22.08
2140	2140-2160	2140-2160	3.70	3.70	660.40	8.96	11.20	19.95	2.96	4.46		4.46	4.50	232.00			20.8	19.4
2160	2160-2180	2160-2180	3.70	3.70	660.40	8.96	11.20	20.46	2.88	4.38		4.38	5.00	232.01	244.00	241.00	20.2	20.6
2180	2180-2200	2180-2200	3.70	3.70	660.40	5.85	7.31	25.98	3.48	4.98		4.98	5.00	232.21	245.00	241.00	25.6	12.5
2200	2200-2220	2200-2220	3.70	3.70	660.40	5.85	7.31	26.92	3.35	4.85		4.85	5.00	232.89	245.50	242.50	20.2	19.2
2220	2220-2240	2220-2240	3.10	3.10	660.40	5.85	7.31	21.62	4.18	5.68		5.68	5.50	233.39	246.50	243.50	19.2	20.6
2240	2240-2260	2240-2260	3.10	3.10	660.40	5.85	7.31	25.25	3.58	5.08		5.08	5.50	234.36	247.00	244.50	20.0	21.2
2260	2260-2280	2260-2280	3.10	3.10	660.40	5.85	7.31	32.64	2.77	4.27		4.27	5.50	236.00	247.00	245.00	17.4	22.6
2280	2280	2280		0.00				28.41						236.40	248.00	246.00	Check Dams	
2280	2280-2300	2280-2300	3.10	3.10	660.40	7.31	9.87	28.41	2.36	3.86	2.00	5.86	6.00	236.40	248.50	246.50	19.7	20.7
2300	2300-2320	2300-2320	3.10	3.10	660.40	7.31	9.87	22.46	2.98	4.48	1.12	5.60	6.00	237.28	248.50	246.50	17.9	21.7
2320	2320-2340	2320-2340	3.10	3.10	660.40	5.53	7.47	22.01	4.02	5.52		5.52	6.00	238.90	249.00	247.00	20.4	19.7
2340	2340-2360	2340-2360	3.10	3.10	660.40	5.53	7.47	21.89	4.04	5.54		5.54	6.00	239.10	249.00	248.00	19.9	20.1
2360	2360-2380	2360-2391	4.50	4.50	660.40	5.53	7.47	20.45	4.33	5.83		5.83	7.00	240.00	249.50	248.50	20.2	31.7

2380	2380-2400	2391-2405.7	4.50	4.50	660.40	5.53	7.47	17.69	5.00	6.50		6.50	7.00	240.00	250.00	248.80	18.2	stream
2400	2400-2420	2405.7-2420	4.50	4.50	660.40	5.53	7.47	15.78	5.61	7.11		7.11	7.00	240.25	251.00	250.00	19.2	14.3
2420	2420-2440	2420-2440	4.50	4.50	660.40	5.53	7.47	16.35	5.41	6.91		6.91	7.00	240.44	251.00	250.00	20.0	20.0
2440	2440-2460	2440-2460	4.50	4.50	660.40	7.75	9.70	16.10	4.23	5.73		5.73	6.00	240.69	251.00	251.00	20.0	20.0
2460	2460-2480	2460-2480	4.50	4.50	660.40	7.75	9.70	17.27	3.94	5.44		5.44	5.50	241.84	252.00	252.00	19.8	20.2
2480	2480-2500	2480-2500	4.50	4.50	660.40	7.75	9.70	17.53	3.88	5.38		5.38	5.50	242.08	253.00	252.00	23.7	15.8
2500	2500-2520	2500-2520	4.50	4.50	660.40	7.75	9.70	20.56	3.31	4.81		4.81	5.50	243.55	253.00	252.00	20.0	20.1
2520	2520-2540	2520-2540	4.50	4.50	660.40	7.75	9.70	19.55	3.48	4.98		4.98	5.50	244.25	254.00	253.00	20.0	20.0
2540	2540-2560	2540-2560	4.50	4.50	660.40	7.75	9.70	17.95	3.79	5.29		5.29	5.50	244.79	254.00	253.00	19.0	21.0
2560	2560-2580	2560-2580	4.50	4.50	660.40	7.75	9.70	16.51	4.12	5.62		5.62	6.00	245.31	255.00	256.00	18.2	21.7
2580	2580-2600	2580-2600	4.50	4.50	660.40	7.75	9.70	15.91	4.28	5.78		5.78	7.50	245.35	255.00	253.00	21.1	20.0
2600	2600-2610	2600-2610	4.50	4.50	660.40	9.70	9.70	13.85	4.92	6.42		6.42	7.50	245.82	255.00	253.20	10.0	10.0
2610	2610-2625	2610-2625	4.50	4.50	660.40	9.70	9.70							247.03	255.30	254.00	Bridge	Bridge
2625	2625-2650	2625-2650	4.50	4.50	660.40	9.70	0.00	11.90						247.03			Bridge	Bridge
2650	2650-2660	2650-2660	4.50	4.50	660.40	9.70	9.70	7.66	8.89	10.39		10.39	6.00	248.00	256.00	255.00	10.9	No Wall
2660	2660-2680	2660-2680	4.50	4.50	660.40	9.70	9.70	12.74	5.34	6.84		6.84	6.00	248.04			22.4	No Wall
2680	2680-2700	2680-2700	4.50	4.50	660.40	9.70	9.70	15.26	4.46	5.96		5.96	6.00	249.04	257.40	no wall	20.9	No Wall
2700	2700-2720	2700-2720	4.50	4.50	660.40	9.70	9.70	17.29	3.94	5.44		5.44	6.00	250.00	259.00	no wall	12.8	No Wall
2720	2720-2740	2720-2740	4.50	4.50	660.40	9.70	9.70	20.28	3.36	4.86		4.86	6.00	250.00	261.00	no wall	19.4	No Wall
2740	2740-2750	2740-2750	2.70	2.70	660.40	9.61	9.61	24.37	2.82	4.32		4.32	6.00	252.16	262.00	no wall	10.2	No Wall
2750	2750-2780	2750-2780	2.70	2.70	660.40	9.61	9.61	26.94	2.55	4.05		4.05	6.00	253.55	259.00	no wall	No Wall	No Wall
2780	2780-2800	2780-2800	2.70	2.70	660.40	9.61	9.61	29.82	2.30	3.80		3.80	6.00	254.81			No Wall	No Wall
2800	2800-2820	2800-2820	2.70	2.70	660.40	9.61	9.61	11.05	6.22	7.72		7.72	6.00	255.96	no wall	no wall	No Wall	No Wall
2820	2820-2840	2820-2840	2.70	2.70	660.40	9.61	9.61	16.91	4.06	5.56		5.56	6.00	257.41	no wall	no wall	No Wall	No Wall
2840	2840-2860	2840-2860	2.70	2.70	660.40	9.61	9.61	22.42	3.07	4.57		4.57	6.00	258.40	no wall	no wall	No Wall	No Wall
2860	2860-2880	2860-2880	2.70	2.70	660.40	8.94	8.94	29.32	2.52	4.02		4.02	6.00	259.01	no wall	no wall	No Wall	No Wall
		2880-2890		0.00										260.00	no wall	no wall		Stream
2880	2880-2900	2880-2900	2.70	2.70	660.40	8.94	8.94	16.72	4.42	5.92		5.92	6.00	260.00			No Wall	14.6
2900	2900-2920	2900-2920	2.70	2.70	660.40	8.94	8.94	15.94	4.63	6.13		6.13	6.00	260.51	no wall	no wall	No Wall	20.7
2920	2920-2940	2920-2940	2.70	2.70	660.40	8.94	8.94	18.75	3.94	5.44		5.44	6.00	261.07	no wall	263.00	No Wall	20.6
2940	2940-2960	2940-2960	2.70	2.70	660.40	8.94	8.94	21.13	3.50	5.00		5.00	6.00	262.00	no wall	264.00	No Wall	20.0
2960	2960-2980	2960-2980	2.70	2.70	660.40	6.54	6.54	20.67	4.89	6.39		6.39	7.00	263.00	no wall	265.00	No Wall	23.3
2980	2980-3000	2980-3000	2.70	2.70	660.40	6.54	6.54	22.50	4.49	5.99		5.99	7.00	263.63	no wall	266.00	No Wall	20.9
3000	3000-3020	3000-3020	2.70	2.70	660.40	6.54	6.54	20.51	4.92	6.42		6.42	7.00	264.00	no wall	266.00	Stream	8.4
3020	3020-3040	3020-3040	2.70	2.70	660.40	6.54	6.54	20.21	5.00	6.50		6.50	7.50	264.84	no wall	267.00	21.5	18.8
3040	3040-3060	3040-3060	2.70	2.70	660.40	6.54	6.54	16.42	6.15	7.65		7.65	7.50	265.00	0.00	268.00	No Wall	16.1
3060	3060-3080	3060-3080	2.70	2.70	660.40	6.12	6.12	18.82	5.73	7.23		7.23	7.50	266.01	0.00	0.00	No Wall	16.4
3080	3080-3100	3080-3100	2.70	2.70	660.40	6.12	6.12	14.78	7.30	8.80		8.80	7.50	267.00	0.00	0.00	No Wall	20.0
3100	3100-3120	3100-3120	2.70	2.70	660.40	6.12	6.12	16.52	6.53	8.03		8.03	7.50	267.00	0.00	0.00	No Wall	20.0
3120	3120-3140	3120-3140	2.70	2.70	660.40	6.12	6.12	15.91	6.78	8.28		8.28	7.50	268.00	0.00	0.00	No Wall	20.1
3140	3140-3160	3140-3160	2.70	2.70	660.40	6.94	6.94	16.48	5.77	7.27		7.27	7.50	268.39	0.00	0.00	No Wall	No Wall

3160	3160-3180	3160-3180	2.70	2.70	660.40	6.94	6.94	21.53	4.42	5.92		5.92	7.00	269.00	0.00	0.00	No Wall	No Wall
3180	3180-3200	3180-3200	2.70	2.70	660.40	6.94	6.94	19.68	4.84	6.34		6.34	7.00	270.00	0.00	0.00	No Wall	18.9
3200	3200-3220	3200-3220	2.70	2.70	660.40	6.94	6.94	19.22	4.95	6.45		6.45	7.00	270.50	0.00	0.00	No Wall	20.2
3220	3220-3240	3220-3240	2.70	2.70	660.40	6.94	6.94	25.86	3.68	5.18		5.18	7.00	271.05	0.00	0.00	No Wall	7.2
3240	3240-3260	3240-3260	2.70	2.70	660.40	7.45	7.45	26.09	3.40	4.90		4.90	7.00	271.57	0.00	0.00	No Wall	20.3
3260	3260-3280	3260-3280	2.70	2.70	660.40	7.45	7.45	26.94	3.29	4.79		4.79	6.00	272.15	0.00	0.00	No Wall	19.7
3290	3290			0.00				29.25						273.00	0.00	0.00	Check Dams	
3280	3280-3300	3280-3300	2.70	2.70	660.40	7.45	7.45	29.25	3.03	4.53	2.00	6.53	6.00	273.00			No Wall	18.3
3300	3300-3320	3300-3320	2.70	2.70	660.40	7.45	7.45	29.72	2.98	4.48	1.00	5.48	6.00	274.00	0.00	0.00	No Wall	20.0
3320	3320-3340	3320-3340	2.70	2.70	660.40	7.45	7.45	30.29	2.93	4.43	0.93	5.36	6.00	274.07	0.00	0.00	No Wall	20.0
3340	3340-3360	3340-3360	2.70	2.70	660.40	7.45	7.45	26.35	3.36	4.86	0.00	4.86	6.00	275.00	0.00	0.00	No Wall	14.2
3360	3360-3380	3360-3380	2.70	2.70	660.40	7.72	7.72	27.16	3.15	4.65		4.65	6.00	275.99	0.00	0.00	No Wall	20.0
3400	3400			0.00										276.00	0.00	0.00		Ramp
3380	3380-3400	3380-3400	2.70	2.70	660.40	7.72	7.72	25.68	3.33	4.83		4.83	7.50	276.00	0.00	0.00	No Wall	13.5
3400	3400-3420	3400-3420	2.70	2.70	660.40	7.72	7.72	30.81	2.78	4.28		4.28	7.50	276.00	0.00	0.00	No Wall	No Wall
3420	3420-3440	3420-3440	2.70	2.70	660.40	7.72	7.72	20.10	4.26	5.76		5.76	7.50	276.76	0.00	0.00	No Wall	No Wall
3440	3440-3460	3440-3460	3.70	3.70	660.40	7.72	7.72	15.10	5.67	7.17		7.17	7.50	277.03	0.00	0.00	No Wall	No Wall
3460	3460-3480	3460-3480	3.70	3.70	660.40	7.72	7.72	19.47	4.39	5.89		5.89	7.50	278.29	0.00	0.00	No Wall	No Wall
3480	3480-3500	3480-3500	3.70	3.70	660.40	5.75	5.75	18.20	6.31	7.81		7.81	7.50	279.01	0.00	0.00	No Wall	No Wall
3500	3500-3520	3500-3520	3.70	3.70	660.40	5.75	5.75	15.94	7.21	8.71		8.71	7.50	280.05	0.00	0.00	No Wall	No Wall
3520	3520-3540	3520-3540	3.70	3.70	660.40	5.75	5.75	16.23	7.08	8.58		8.58	7.50	281.00	0.00	0.00	No Wall	No Wall
3540	3540-3560	3540-3560	3.70	3.70	660.40	5.75	5.75	17.87	6.43	7.93		7.93	7.50	281.70	0.00	0.00	No Wall	No Wall
3560	3560-3580	3560-3580	3.70	3.70	660.40	5.75	5.75	11.80	9.73	11.23		11.23	7.50	283.00	0.00	0.00	No Wall	No Wall
3580	3580-3600	3580-3600	3.70	3.70	660.40	5.75	5.75	11.05	10.39	11.89		11.89	7.50	283.00	0.00	0.00	No Wall	No Wall
3600	3600-3620	3600-3620	3.70	3.70	660.40	5.75	5.75	10.58	10.86	12.36		12.36	7.50	285.00	0.00	0.00	No Wall	No Wall
3620	3620-3640	3620-3640	3.70	3.70	660.40	5.75	5.75	10.26	11.19	12.69		12.69	7.50	285.24	0.00	0.00	No Wall	No Wall
3640	3640-3660	3640-3660	3.70	3.70	660.40	5.75	5.75	16.48	6.97	8.47		8.47	7.50	286.68	0.00	0.00	No Wall	No Wall
3660	3660-3680	3660-3680	3.70	3.70	660.40	5.75	5.75	13.69	8.39	9.89		9.89	7.50	286.68	0.00	0.00	No Wall	No Wall
3680	3680	3680	3.70	3.70	660.40	5.75	5.75	14.52	7.91	9.41		9.41	7.50	286.68	0.00	0.00	No Wall	No Wall