BEFORE THE NATIONAL GREEN TRIBUNAL
SOUTHERN ZONE, CHENNAI

Application No.272 of 2024 (SZ)

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Aged about 37 years
30, ururkuppam, Besant Nagar, Chennai – 90 - Applicant

Vs

1. Tamil Nadu Coastal Zone Management Authority
By its Member Secretary
1, Jennis Road, Panagal Building
Ground Floor, Saidapet, Chennai – 600 015
2. The National Highways Authority of India
Rep by its Project Director
Sri Balaji Towers, 54-28, Butt Road, near kathipara Junction
South Phase, SP Industrial Estate Area, Parangi malai
Guindy, Chennai, Tamil Nadu – 600 016 - Respondents

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Dated at Chennai on this 24th Day of June 2025

Counsel for 2nd Respondent

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# MUGAIYUR – MARAKKANAM SECTION

# HYDRAULIC MODELLING REPORT
# For
# Effect of NH-332A Design Ch. 55.350 km Highway Embankment on Water bodies in Kottaikadu Village

**Prepared for**

## National Highway Authority of India


**Prepared by**
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**24th JUNE 2025**

# DOCUMENT NO: DPR-HYD-RPT- HYW/01/2025

| Date | Rev No | Prepared by | Checked by | Approved by |
|---|---|---|---|---|
| 24.06.2025 | R0 | Venkat | J. Sangeeth | Dr. K Sravan Kumar |

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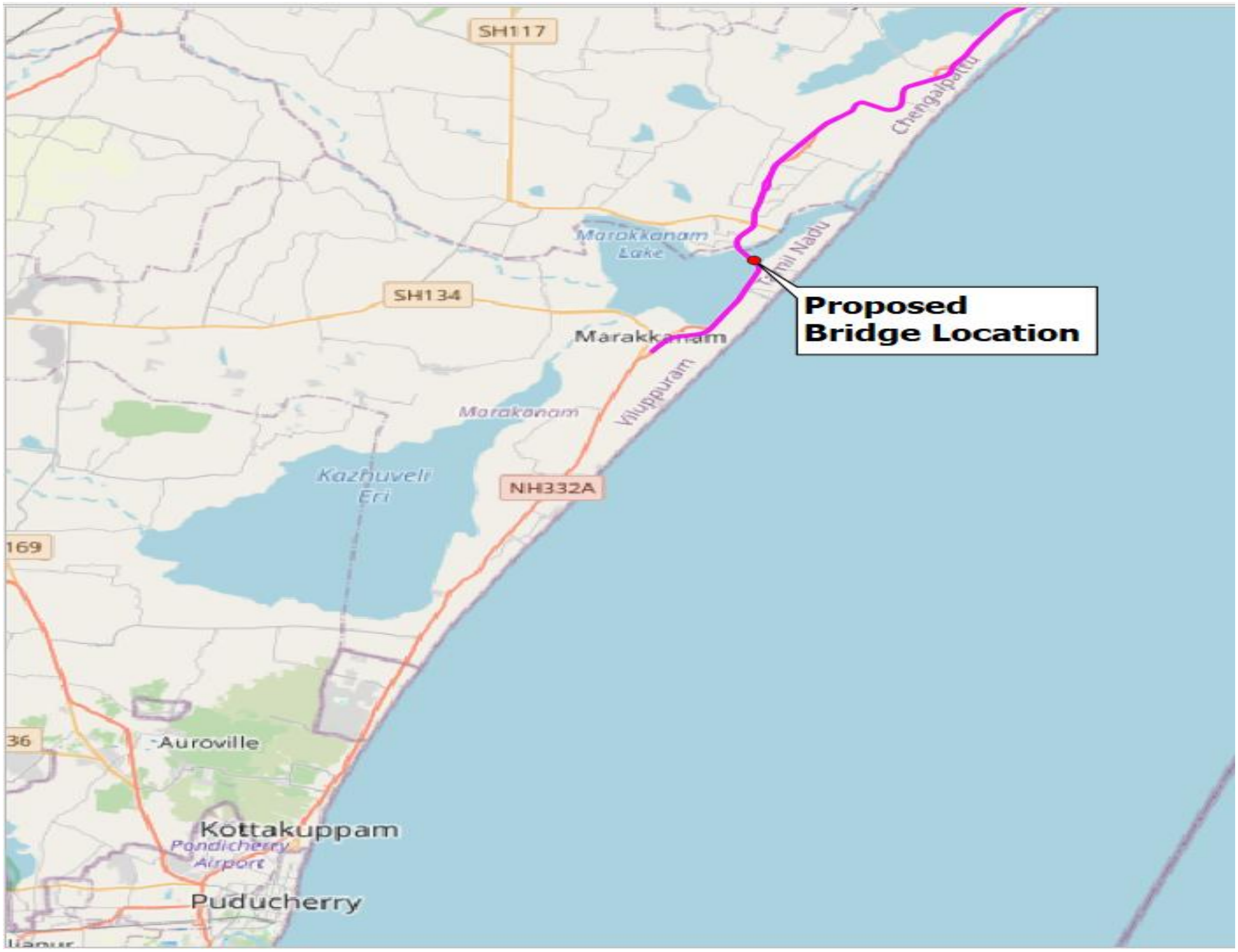
# 1 Introduction

As part of the Mamallapuram – Puducherry Project, a four-lane highway has been proposed for the Mugaiyur – Marakkanam section between Puducherry and Mamallapuram along NH-332A in Tamil Nadu state, with the primary goal of enhancing the efficiency of freight movement in the region. To facilitate this improvement, the project includes the construction of a bridge at the 55+350 chainage located in Cheyyur Taluk of Chengalpattu District in Tamil Nadu. The specific location of this bridge is clearly indicated in the figure below. This bridge is a key component of the overall highway development and aims to support smoother and more efficient transportation along this vital corridor.




# 2 Purpose/Objective of Report:

The primary objective of this report is to conduct a thorough hydrological and hydraulic analysis to evaluate the potential impacts of the proposed National Highway, specifically at the 55+350 meters chainage, on the watercourse in Kottaikadu village in Cheyyur Taluk, Chengalpattu District. The analysis is vital to understanding how the project might influence water flow within the estuary, particularly during flooding events, and to ensure that the construction adheres to the environmental regulations as stipulated by the National Green Tribunal (NGT).

This report is particularly focused on:

- **Hydrological Analysis**: Understanding the flow dynamics, catchment areas, and rainfall patterns impacting the project corridor.
- **Design of the Bridge**: Assessing whether the bridge design accommodates the expected water flow during peak conditions, as per flood frequency analysis.
- **Environmental Considerations**: Evaluating how the construction impacts local ecosystems, watercourses particularly on the tidal flows, and flood plains.

The study will also delve into the necessity of a 1.2 km-long major bridge, which will enable the passage of floodwaters, ensuring that the highway does not exacerbate flooding downstream or obstruct natural water flow within estuary.

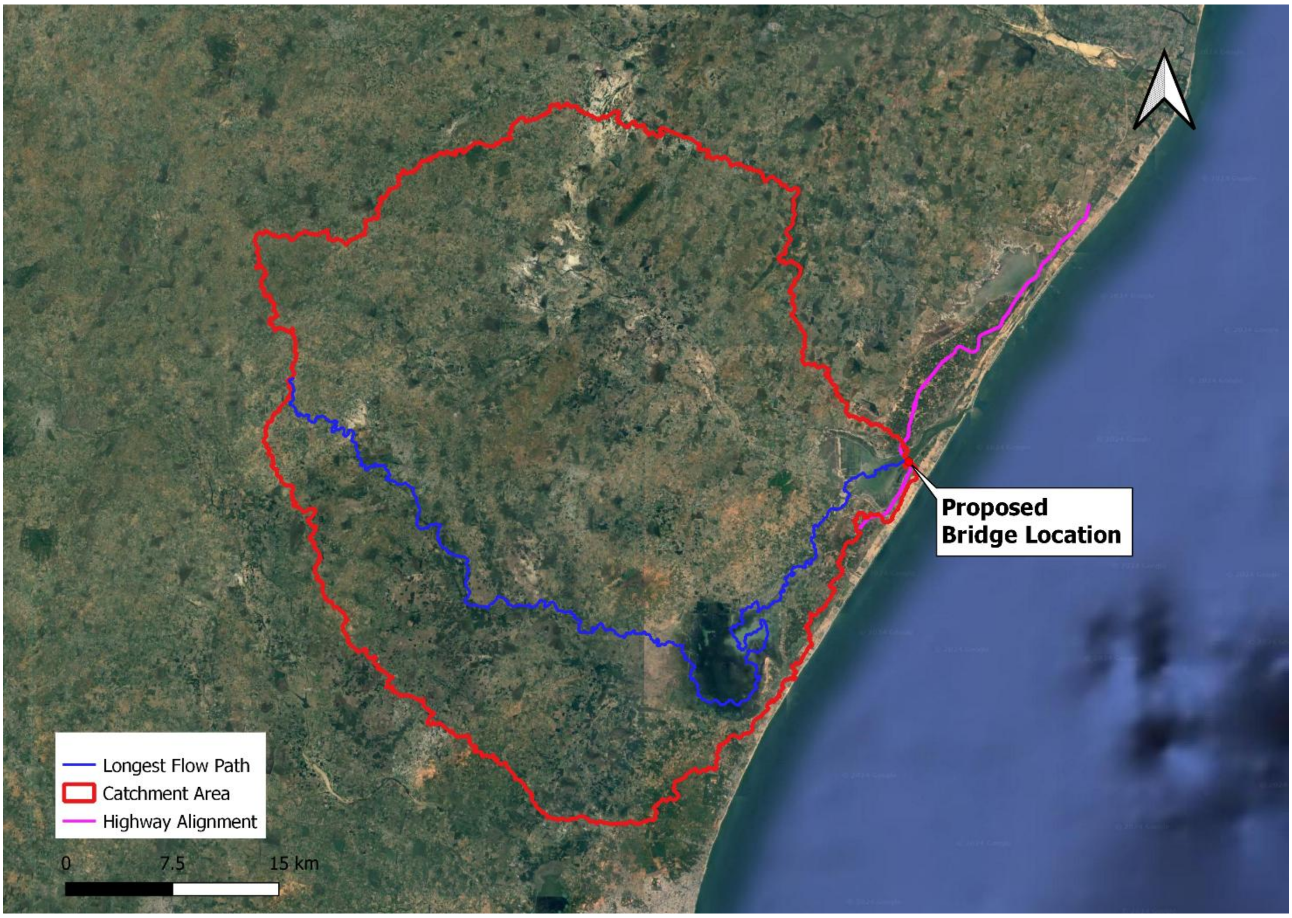
# 3 Hydrological Analysis

Hydrological analysis is crucial in understanding how water moves within a specific catchment area, particularly how it influences the design of proposed infrastructure such as bridges, roads, and embankments. This analysis helps to ensure that such projects do not exacerbate flooding or disrupt the natural flow of watercourses. In the context of the proposed four-lane highway project, the hydrological analysis focuses on assessing catchment characteristics, runoff, rainfall patterns, and the corresponding design discharge to determine the potential impact of the project on local water systems.

## 3.1 Catchment Delineation and Characteristics

Hydrological analysis begins with determining the catchment characteristics, including size, slope, land cover, and the stream network. The catchment boundaries are delineated using the **QGIS** hydrological analysis tools, which use **SRTM (Shuttle Radar Topography Mission)** data for accurate modeling. The following steps were carried out in QGIS:

1. **Data Acquisition**: Downloading required SRTM DEM data from the USGS Earth Explorer portal.
2. **Projection Setup**: Setting the projection to ensure proper alignment for accurate analysis.
3. **Filling Sinks**: A common procedure in hydrological modeling to fill depressions in the DEM.
4. **Flow Direction and Channel Network**: Modeling the flow direction of water across the landscape to identify the major watercourses.
5. **Catchment Delineation**: Identifying the boundaries of the catchment and assessing its area (1540 km$^2$) and stream length (71 km) as shown in the fig below.




### 3.2 Design Rainfall

Rainfall data for designing waterway crossings is sourced from Isopluvial maps published by the Central Water Commission (CWC) and Indian Meteorological Department (IMD), covering return periods of 25, 50, and 100 years. For this project, the 100-year 24-hour rainfall of 32 cm, based on the centroid of the catchment area, is used for design discharge calculations. This value is crucial for sizing waterway crossings and ensuring the infrastructure can handle extreme rainfall events, preventing flooding and structural damage.

### 3.3 Design Discharge Estimation

Design engineers essentially need the design flood of a specific return period for fixing the waterway crossing, design HFL of bridges depending upon their size and importance to ensure safety as well as economy. The manual of specifications and standards for the project specify that the waterway bridge is to be designed for a maximum flood of 100-year return period for Important / Major bridges and 50-year return period for other cross drainage structures consisting of minor bridges.

The following methods are to be used to estimate the peak discharge at bridge sites on major and minor streams:

Rational Method for crossings with minor catchments (area < 25km$^2$)

Synthetic Unit Hydrograph (SUH) Method for crossings with medium catchments (area between 25km$^2$ to 5000km$^2$).

Flood frequency analysis of recorded discharge data/ Rainfall Routing Method for crossings with major catchments (area > 5000km$^2$).

As the catchment area is greater than 25sqkm and there is no gauge data available at the site, the Synthetic Unit Hydrograph Method has been used to calculate the discharge at the proposed bridge location.

## 3.4 Synthetic Unit Hydrograph (SUH) method

This method is used for those bridges, which cater to streams fed from catchment areas larger than 25 km$^2$. The catchment area for the proposed bridge location coming under 4b- East Coast subzone of CWC Flood Estimation Reports (FER) (applicable range for a catchment area from 25 km$^2$ to 5,000 km$^2$).

1. In the subzone 4(b) the 1-hour Synthetic Unit Hydrograph is determined for an ungauged catchment. Following steps have been followed as suggested in the CWC FER report for the determination of discharge by the method mentioned in this report.
2. Physiographic parameters of the un-gauged catchment viz. Catchment Area (A), Length of Longest Stream (L), Length of longest stream opposite to centroid (Lc), and Equivalent Slope (S) determined from QGIS.
3. SUH parameters have been computed using the following equations:

*Synthetic Unit Hydrograph Parameters for Subzone 4(b)*

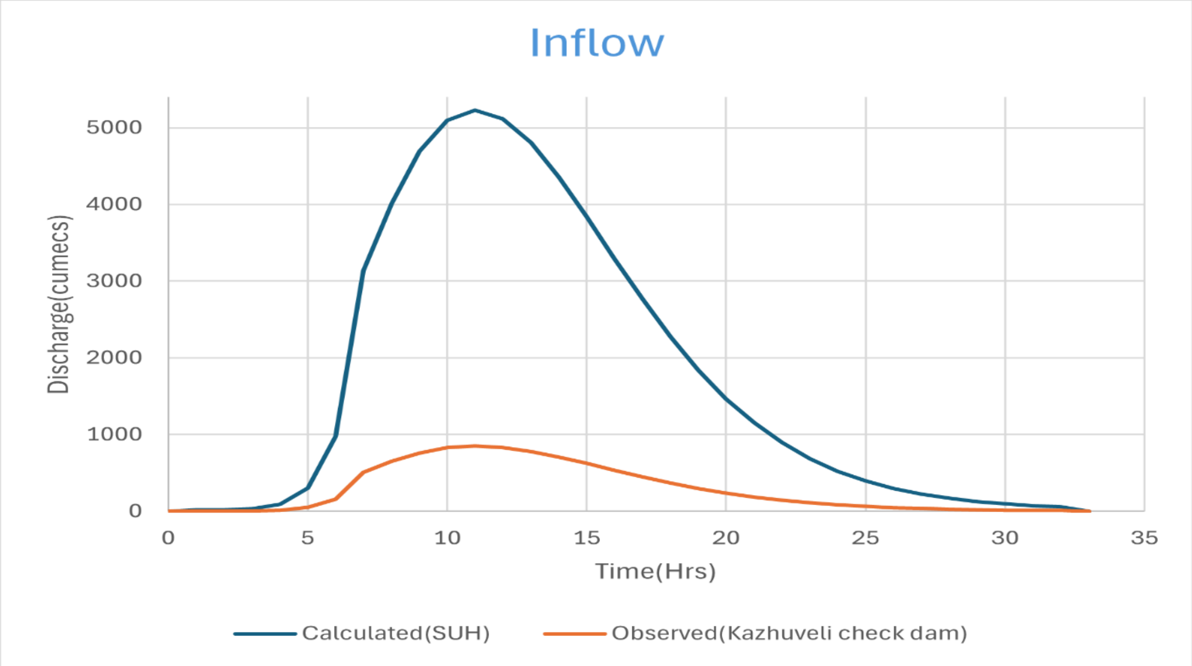
| Parameter | Subzone 4(b) |
|---|---|
| $t_p$ | $0.376\ (L.L_c / \sqrt{(S)})^{0.434}$ |
| $q_p$ | $1.215/(t_p)^{0.691}$ |
| $W_{50}$ | $2.211/(q_p)^{1.07}$ |
| $W_{75}$ | $1.312/(q_p)^{1.003}$ |
| $W_{R50}$ | $0.808/(q_p)^{1.053}$ |
| $W_{R75}$ | $0.542/(q_p)^{0.965}$ |
| $T_B$ | $7.621(t_p)^{0.623}$ |
| $T_m$ | $t_p+t_r/2$ |

4. The estimated parameters of unit-hydrograph in 4(b) have been plotted and the plotted points are joined to draw a synthetic unit hydrograph. The discharge ordinates of SUH at an interval of unit hour duration were found out from the equation of the plotted graph. The obtained value of the ordinates is adjusted to get proper unit-hydrograph shape and area under the unit-hydrograph. The unit-hydrograph ordinates are summed up and multiplied by the unit hour duration and compared with the volume of 1 cm direct run-off depth over catchment computed by the formula as given below.

$$Q = (A / 0.36) \times t_r$$

5. The design storm duration has been taken as equal to the base period of the unit graph ($TD=1.1*t_p$).
6. Point rainfall is read from the given plate in the CWC report for 100-year 24hr rainfall and has been converted to areal-rainfall of 100 years and design storm duration.
7. The areal rainfall of design storm duration is split into 1-hour rainfall increments using time distribution coefficients.
8. Estimation of effective rainfall excess unit has been done after taking design loss rate into account.
9. Base flow has been estimated based upon the catchment area.
10. Finally, for 100-year peak discharge, the effective rainfall excess after removing the losses from rainfall increments are arranged against unit-hydrograph ordinates such that the maximum of effective rainfall is placed against the maximum UG-ordinate, next lower value of effective rainfall against next lower value of UG-ordinate and soon. Sum of the product of the above two added together with base flow gives peak discharge value for the catchment at the point of interest.

The peak flow calculated using the SCS Unit Hydrograph (SUH) method is 5230 cubic meters per second (cumec). However, the observed flow at the Kazuveli check dam 7km upstream of the proposed bridge location is 849 cumec, which indicates a discrepancy between the calculated and approved flow values. To reconcile this difference, the flow hydrograph has been scaled down to match the approved flow value of 849 cumecs. This adjustment ensures that the model aligns with the approved design parameters and reflects the expected conditions according to the GAD. The image below illustrates the scaled hydrograph, showing the modified flow profile that corresponds to the approved 3454 cumecs peak flow. This step is crucial for ensuring that the hydraulic analysis and design meet regulatory requirements and accurately represent the anticipated water flow during peak conditions.




## 4 Hydraulic Modelling

HECRAS has been selected as the numerical hydraulic modelling software to assess the hydraulic impact of the proposed bridge. It is a suite of advanced 1D/2D computer simulation software for flooding, urban drainage and coastal hydraulics. With over 30 years of continuous development, HECRAS is internationally recognized for hydraulic modelling, speed and workflow efficiency.

- HECRAS is one of the most widely used and accepted hydraulic modelling software for flood modelling worldwide.
- HECRAS has been evaluated by several flood-responsible agencies worldwide and it is accepted by relevant organizations.
- HECRAS results are comparable to the ones from any other software.
- 1D river modelling, 2D flood plain modelling, 1D-2D modelling can be performed in HECRAS in a single model.

HECRAS is designed to perform one-dimensional and two-dimensional hydraulic calculations for a full network of natural and constructed channels, overbank/floodplain areas or levee protected areas.

## 4.1 Modelling Approach

The overall flood flow hydraulic modelling and mapping is mainly dependent on the river characteristics and data availability. The main steps required to implement a hydrodynamic model are common to all hydraulic modelling projects. A project must be defined; the geometry of this project, including the river geometry, the cross sections and the structures has to be defined; boundary conditions, including upstream (flow) and downstream (water level or slope) conditions; the roughness coefficients, simulation parameters have to be defined, and the output of the model have to be carefully checked.

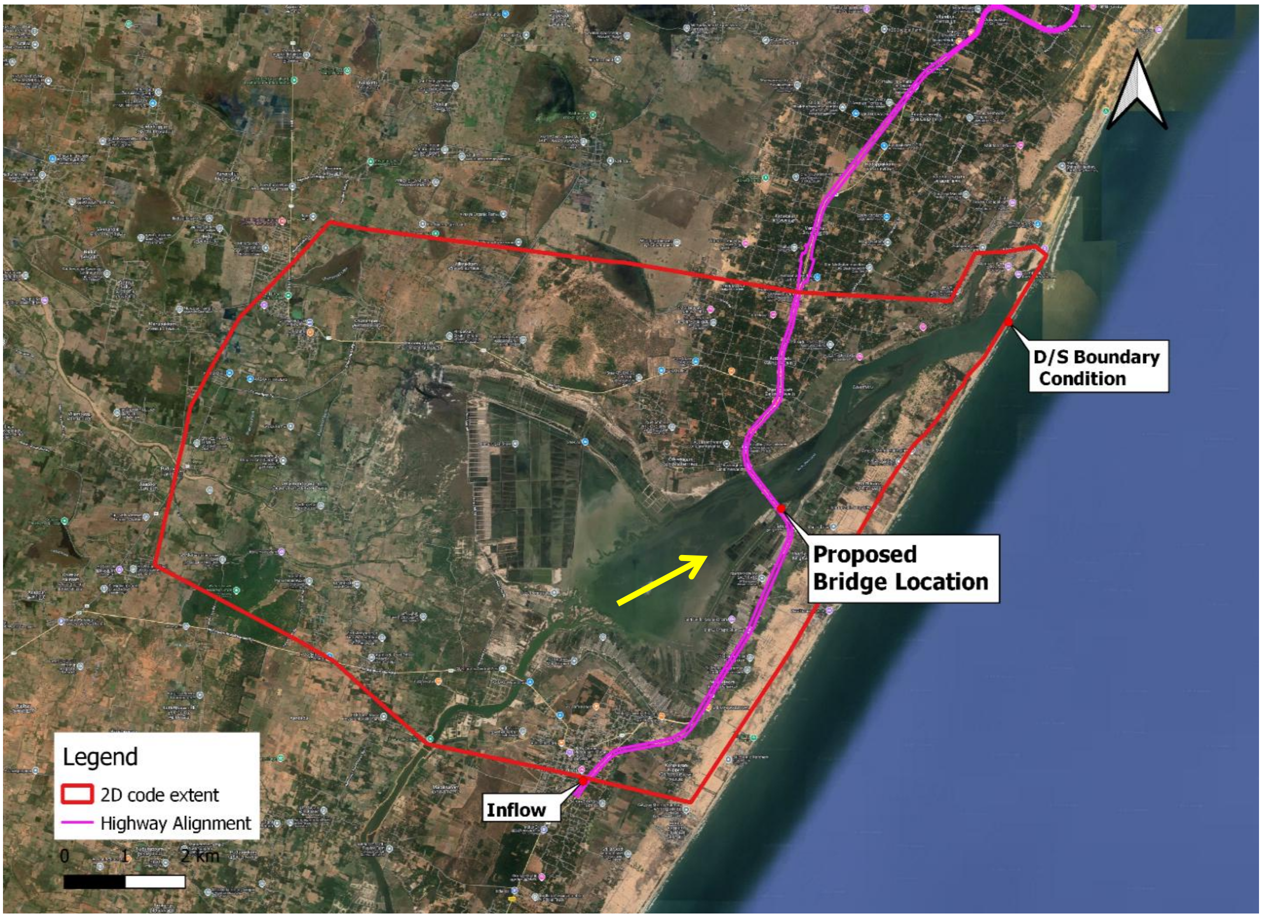
To run a hydraulic model, several data requirements exist, such as geometry information (including a digital elevation model, cross sections and structures) and boundary conditions. It should be noted that within the framework of this project, the bathymetry surveyed cross-section data was not readily available and hence the SRTM (Shuttle Radar Topography Mission) data has been used for Hydraulic modeling. This SRTM data has been merged with available survey data at the bridge location. SRTM Digital DEM of 30m resolution was downloaded from USGS Earth Explorer.

The inflow boundary condition in the model was defined based on the hydrograph, which was scaled down to the designed capacity of the Kazhuveli check dam upstream of the proposed bridge. This ensures that the model reflects the expected flow conditions during peak flood event. To further assess the impact of different boundary conditions, two optioneering runs were modelled, each with a different approach for defining the boundary condition at downstream:

1. **Observed Tidal Water Levels Boundary Condition** – The first scenario run with the downstream boundary condition with observed tidal water levels at Marakkanam location near to project area, accounting for the influence of tidal fluctuations on the flow entering the system. The inflow boundary condition for peak flow (design capacity of the Kazhuveli check dam) has been applied for this scenario and this is crucial for areas where tidal effects are significant, as these can impact the flow dynamics, water levels, and overall hydraulic behaviour, especially in coastal or estuarine environments.
2. **Observed Tidal water levels including storm surge** – The second scenario run with the boundary condition with observed tidal water levels including storm surge. The storm surge at the project location is observed as 2.68m for the 11 % climate change 1 in 100year. The inflow boundary condition for normal flow (1 in 20 year) has been applied as the worst-case scenario is Storm surge plus normal flow.

By running both scenarios, the model can better capture the range of possible conditions and provide insights into how the system might behave under different boundary constraints. The results from these two optioneering runs can help inform decisions regarding the most suitable design and operational parameters for the project.

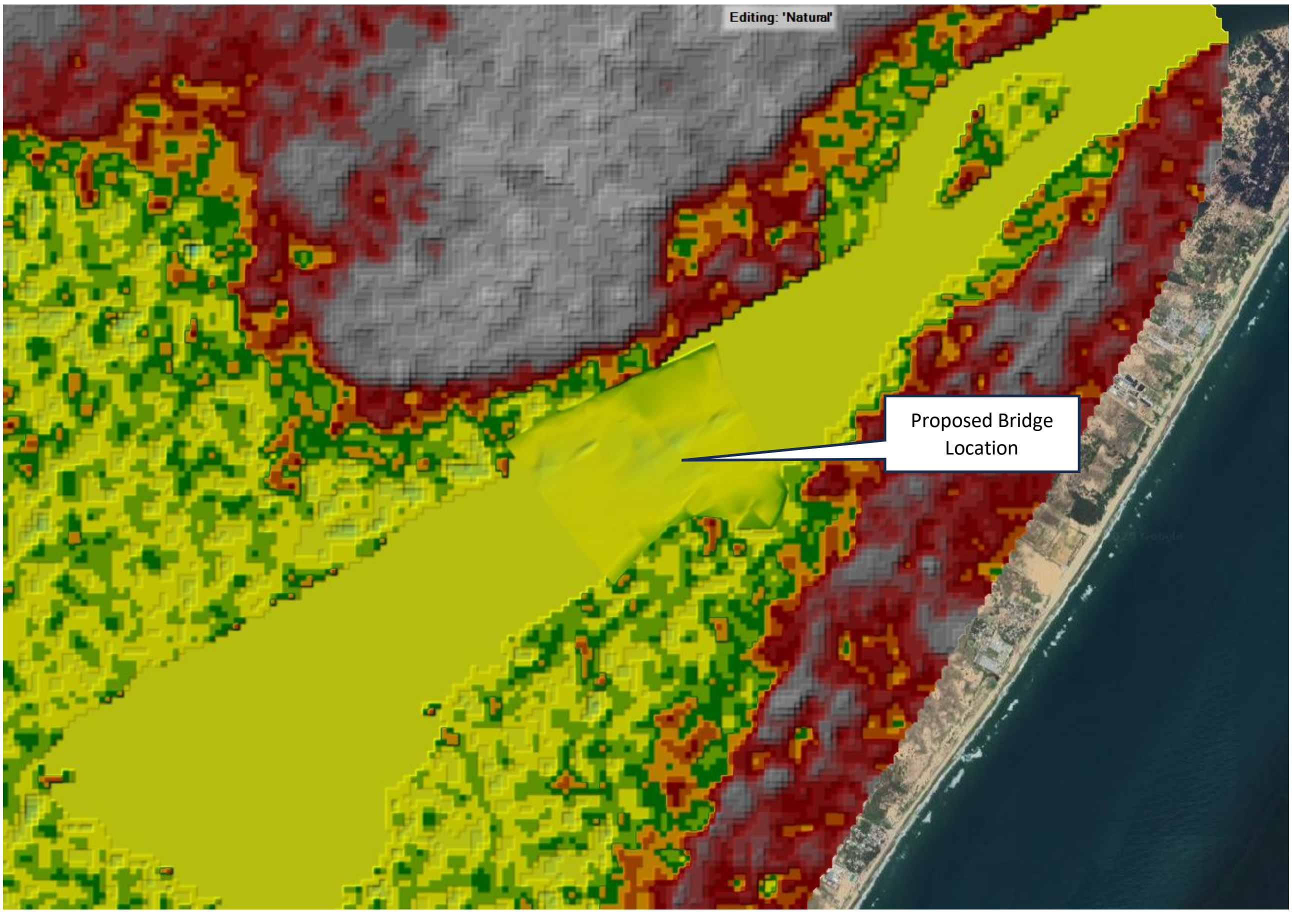
For better visualization of the flood flow within the floodplain, the area of interest is modelled in 2D (Two Dimensional) flow model, incorporating the information from SRTM DEM for the terrain information along with surveyed data. The model domain extends 7km upstream and 3.5km downstream of the proposed bridge. The proposed hydraulic model's code extent, inflow and downstream locations and other features are shown in the fig below.



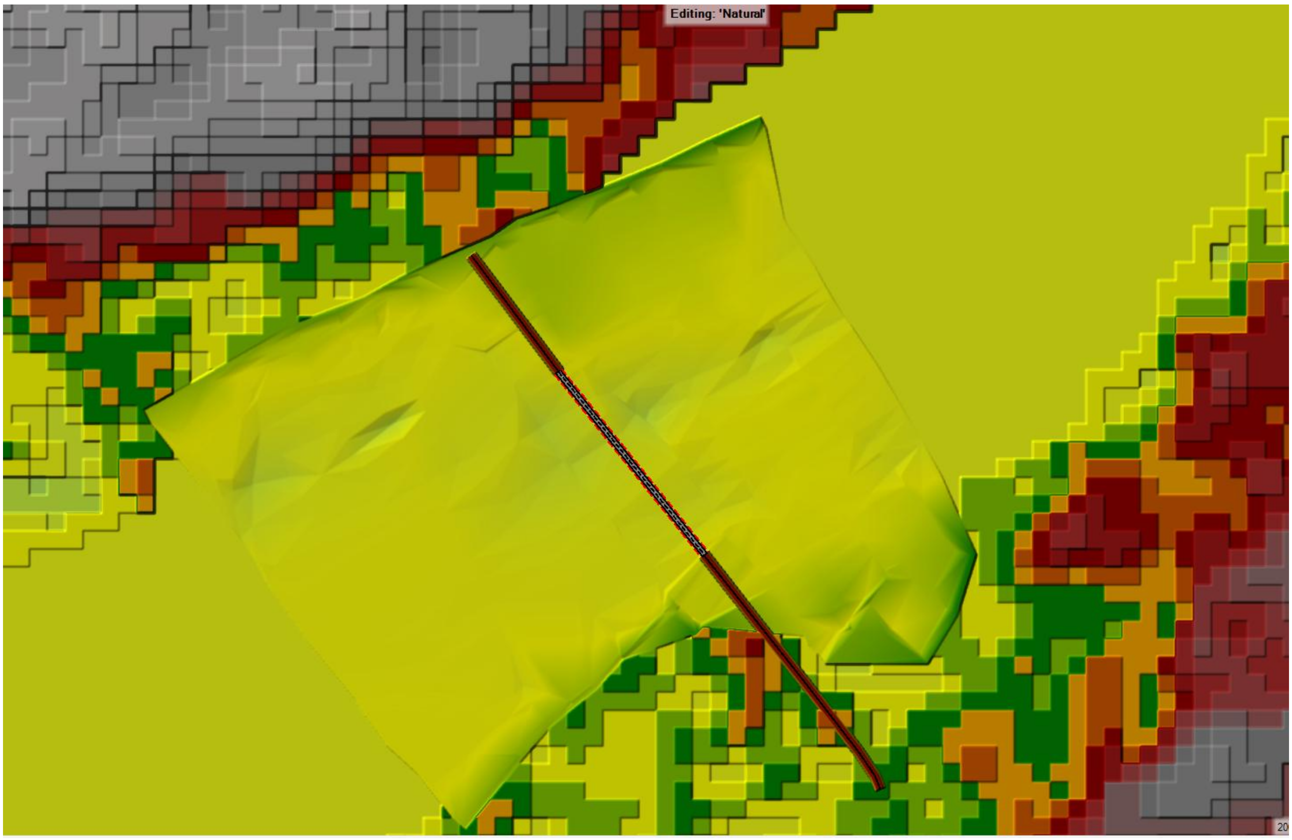

The model extent covers the highway alignment chainage 55+30`0. The hydraulic modelling has been undertaken for the predicted 1 in 100-year design peak flow at the proposed bridge location. The cell size considered for modelling is 30m. The cell size has been considered adequate for describing flooding behavior given the rural nature of the study area and topography of the floodplain.

The model has been prepared for four distinct scenarios, each aimed at assessing different bridge configurations and their impact on water levels in the surrounding area.

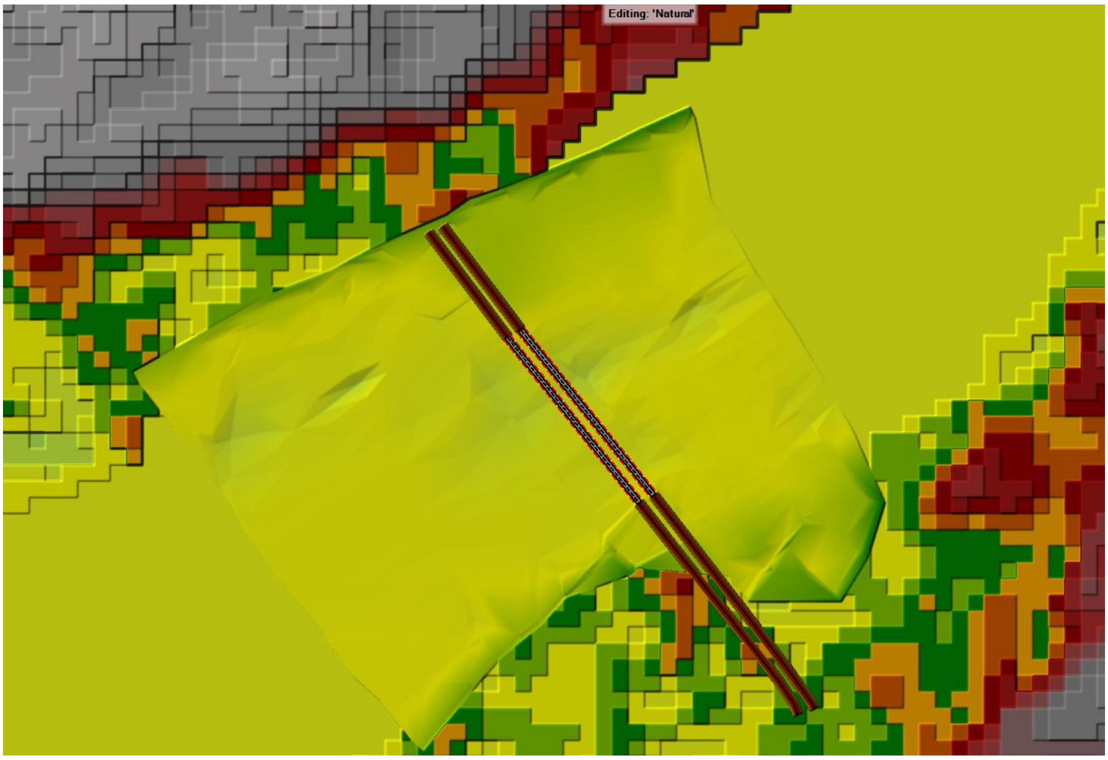
**Scenario 1(Natural)** – In this case, the water levels have been calculated assuming the absence of the existing bridge and embankments. Essentially, this scenario models the conditions before the construction of the current bridge, providing a baseline for comparison. The purpose of this scenario is to simulate the hydrological conditions before any intervention, as the National Green Tribunal has proposed replacing the existing bridge and constructing a new one with the same length of the stream, ensuring minimal disruption to the environment around the bridge and water flow within estuary.



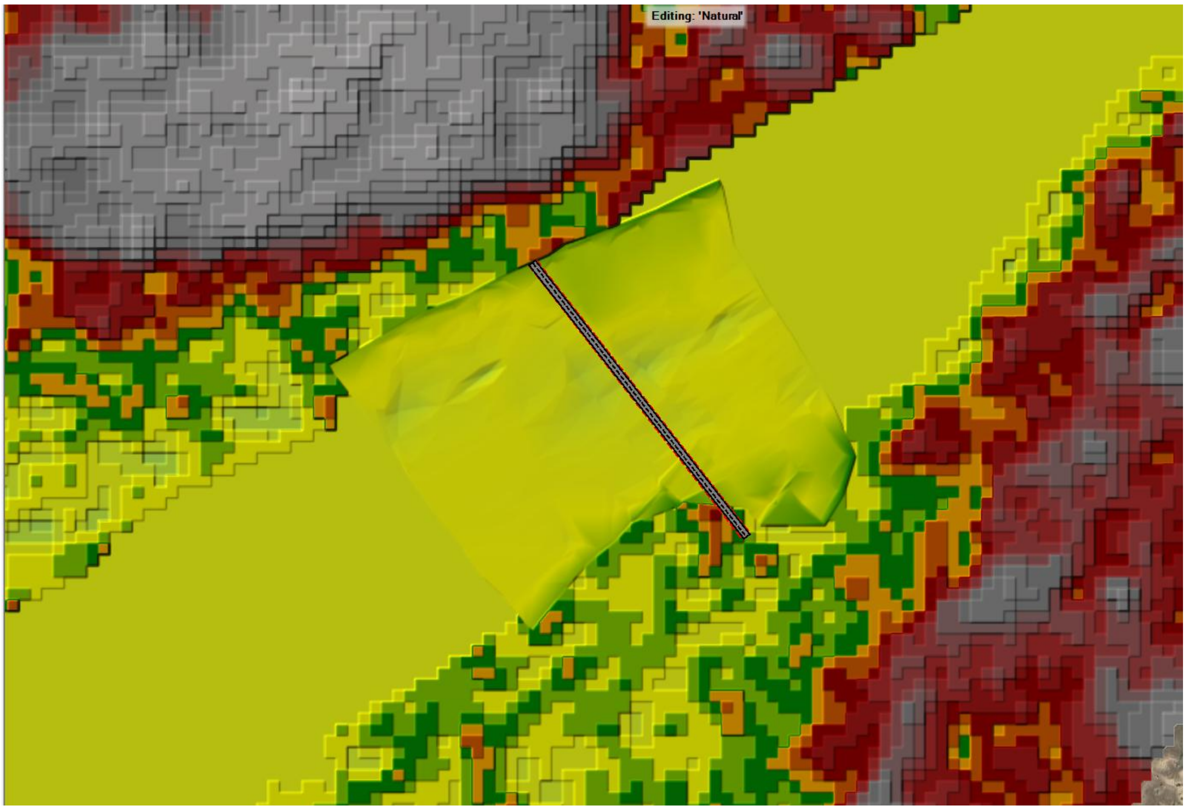

**Scenario 2(Existing Bridge)** – This scenario replicates the current situation, with the existing bridge and embankments in place. It involves modelling the 44 spans of 9 meters each, which make up the current bridge. This scenario serves as a reference point for understanding how the current structure influences water flow and how it compares to the proposed changes.




**Scenario 3(Proposed Bridge)** – In this scenario, a new bridge is modelled in addition to the existing one. The proposed bridge is designed with 11 spans of 36 meters each, which significantly alters the bridge's geometry compared to the existing structure. This scenario helps in evaluating the impact of the new bridge design alongside the current one, to understand its effects on water levels, flow patterns, and potential environmental consequences.




**Scenario 4(Combined)** – This is an alternative proposal in which the new bridge 26 spans of 36 meters each, with both the existing and proposed bridges being integrated into the same deck, which spans the entire estuary. The scenario helps in exploring whether this option might be more beneficial in terms of maintaining water flow, structural stability, and environmental considerations.



### 4.1.1 Hydraulic Roughness Values

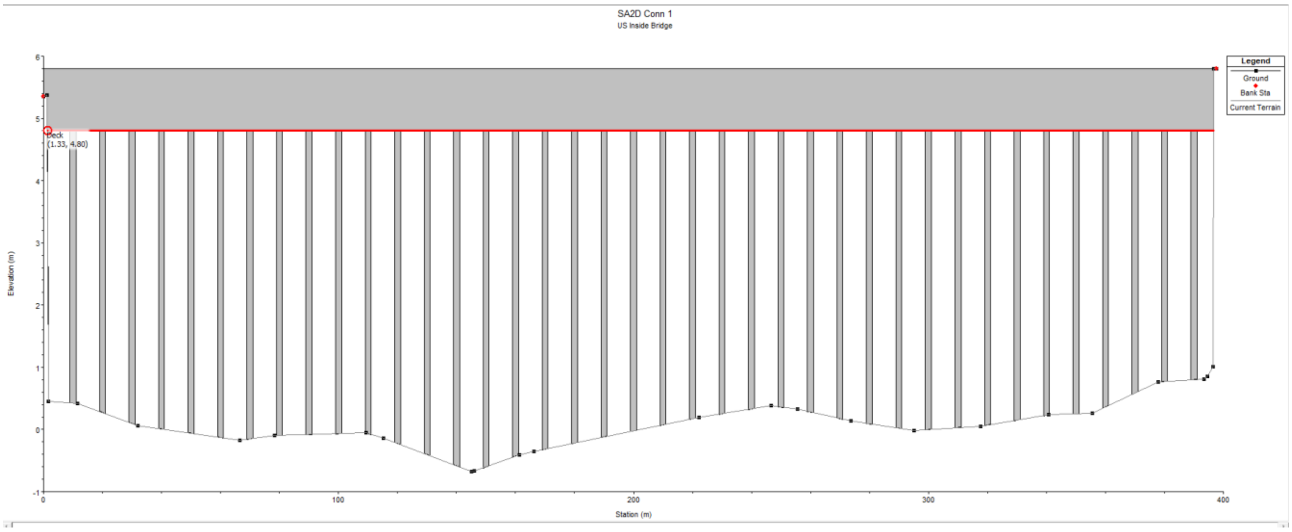
The roughness (Manning's 'n') values used for the 2D modelling is listed in the Table 1.

*Table 1 Roughness values*

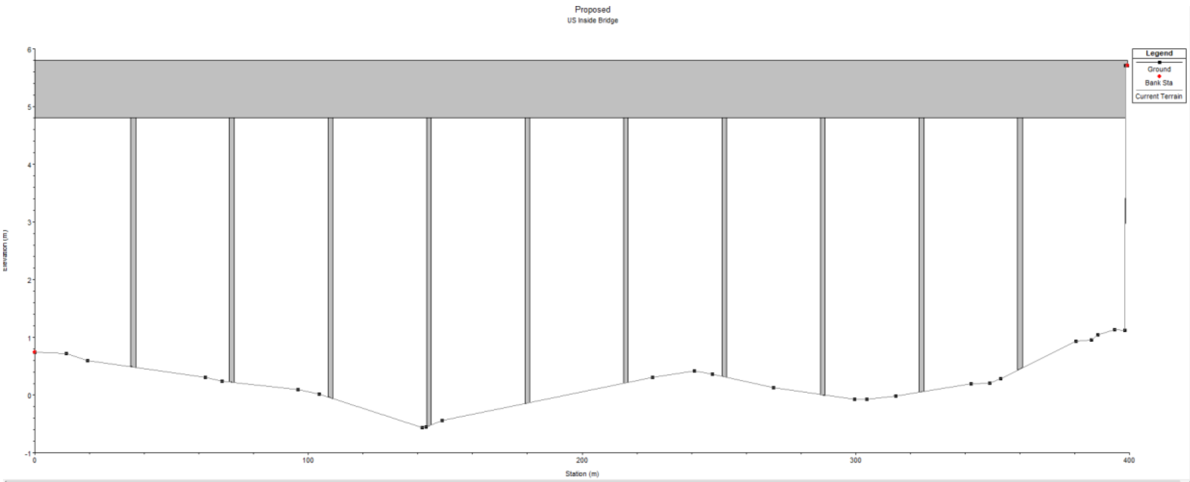
| Sl. No. | Land Use Description | Manning's 'n' |
|---|---|---|
| 1 | Waterbody | 0.035 |
| 2 | Salt Lakes | 0.03 |
| 3 | Land | 0.07 |
| 4 | General | 0.065 |
| 5 | Vegetation | 0.06 |
| 6 | Buildings | 1 |

### 4.1.2 Hydraulic Structures

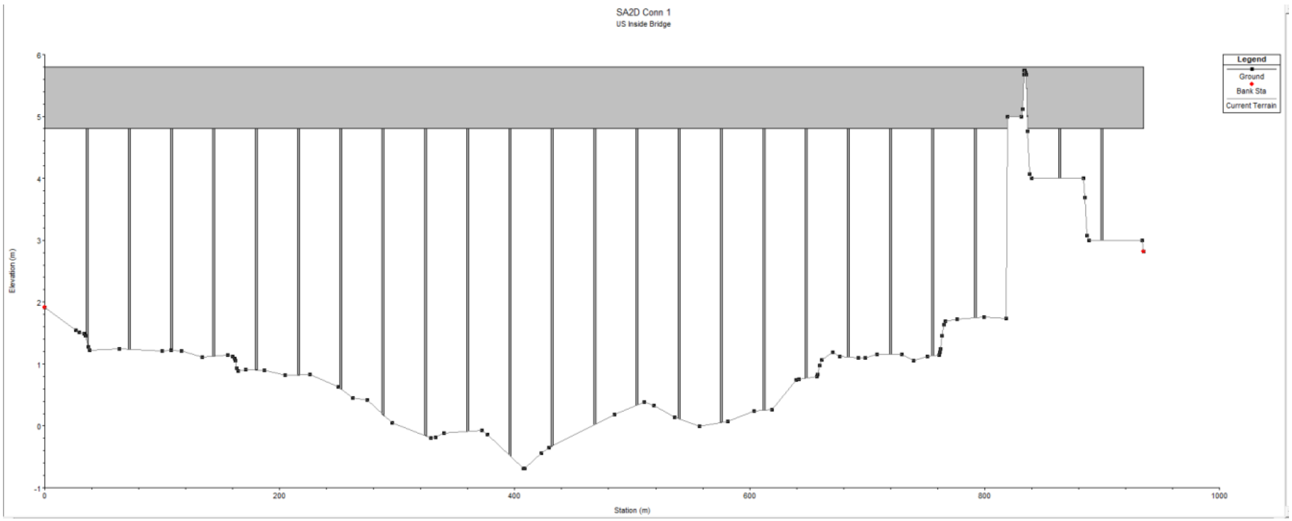
The existing bridge has been modelled with the span configuration of 44*9m as shown in the fig below




Proposed bridge has been modelled with 11*36m span configuration as shown in the fig below



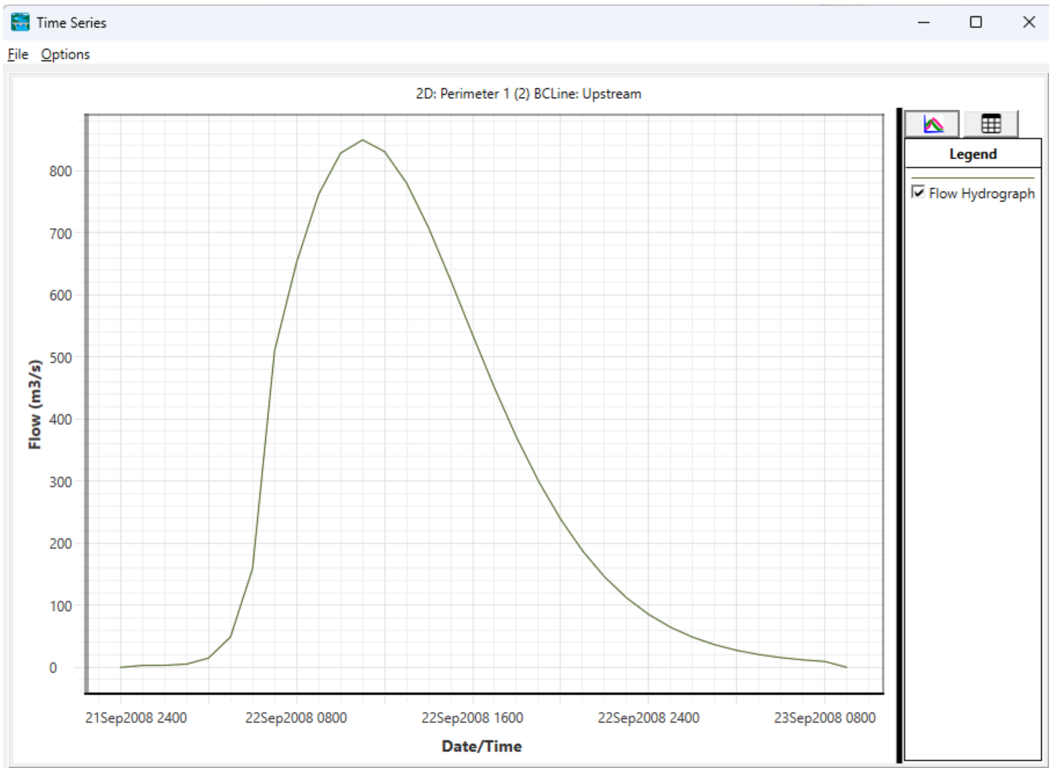

Another option of bridge spanning the full estuary i.e.,26 spans of 36m each as shown in the fig below.



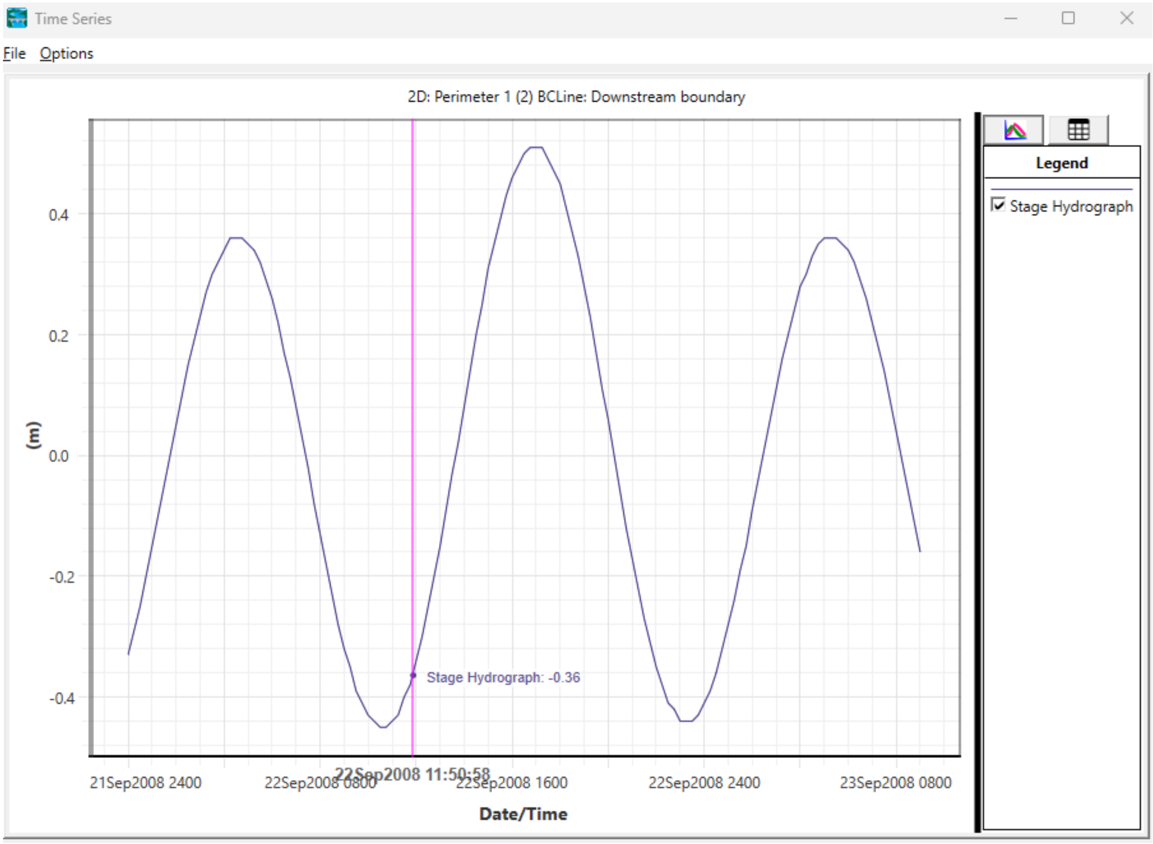

### 4.1.3 Boundary Conditions

The following boundary conditions are provided for the 2D numerical model.

- Upstream Boundary – Inflow hydrograph: The flow time series (QT) is provided at check dam which is 7km upstream of the proposed bridge. The designed flow of 849m$^3$/s was adopted for the given stream. Inflow hydrograph as shown in fig below using the peak flow generated by SUH method for 1 in 100-year event considered as upstream boundary condition.



The stage hydrograph, based on the observed maximum tide levels, has been applied as the downstream boundary condition to replicate the actual tidal cycle scenario within the estuary. This approach accounts for tidal influences, especially since the location is near the sea, providing a worst-case scenario. The tidal stage hydrograph at the downstream boundary is shown in the figure below.

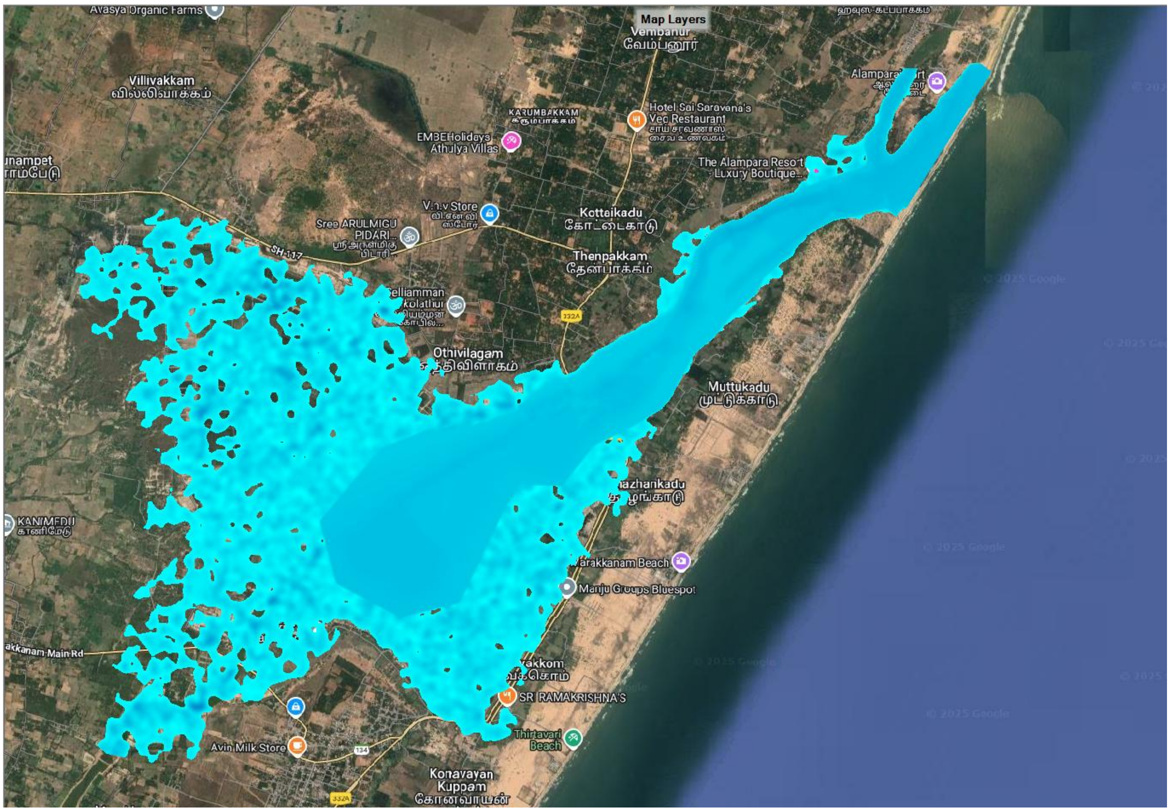



When defining a hydraulic model, it is important to locate the boundary conditions sufficiently far away from the area of interest to avoid boundary effects. This is not always practical or possible because of computational resources, data availability of the geometry of the river system. In this case, it is possible to locate the boundary condition relatively far away from the area of interest.

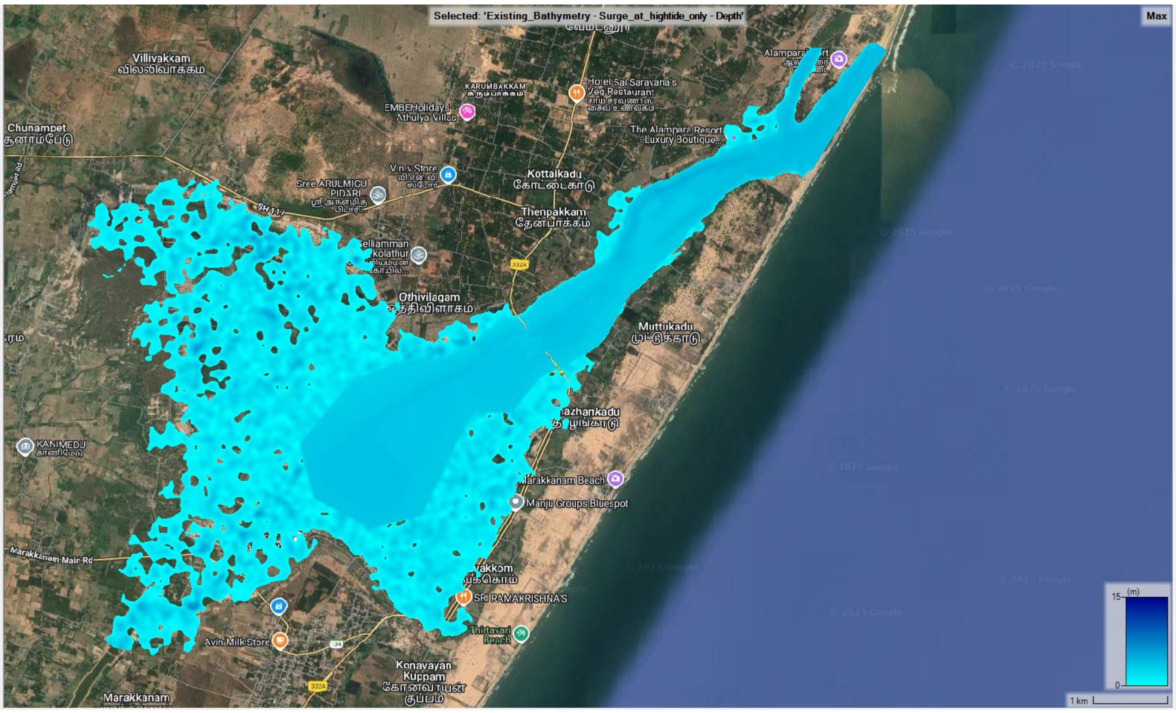
## 4.2 Results

The following figures shows the max flood extent for all the scenarios in observed tidal flood and observed tidal flood plus storm surge cases.

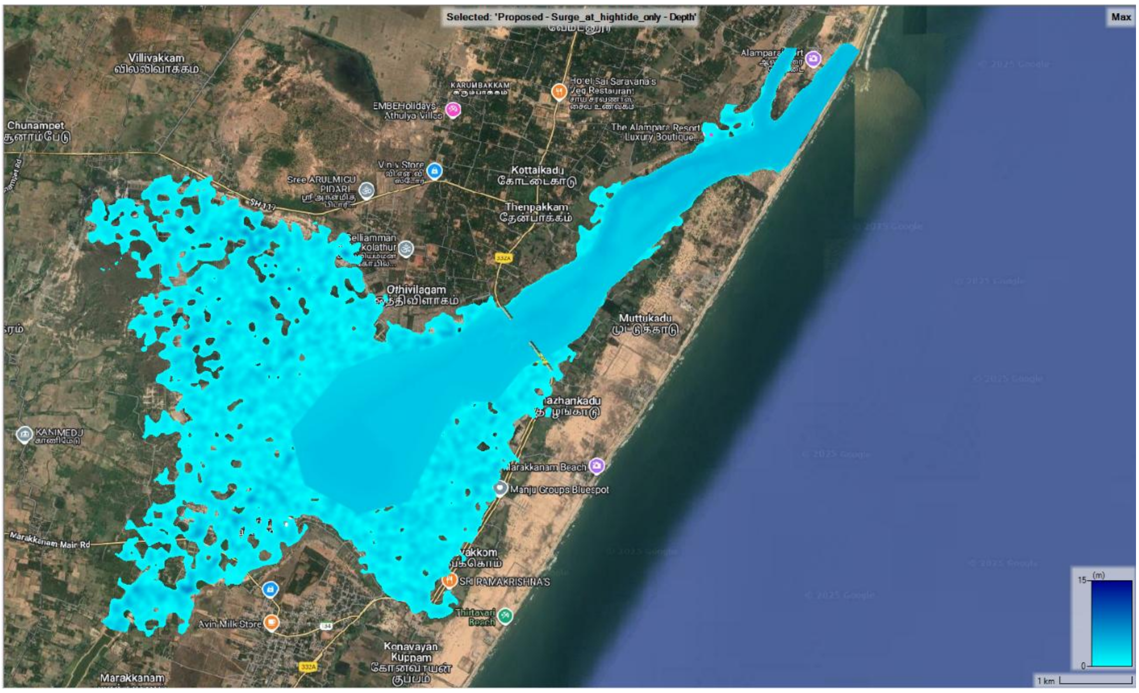
### 4.2.1 Max Flood extent map for Scenario 1(Natural Estuary) without storm surge



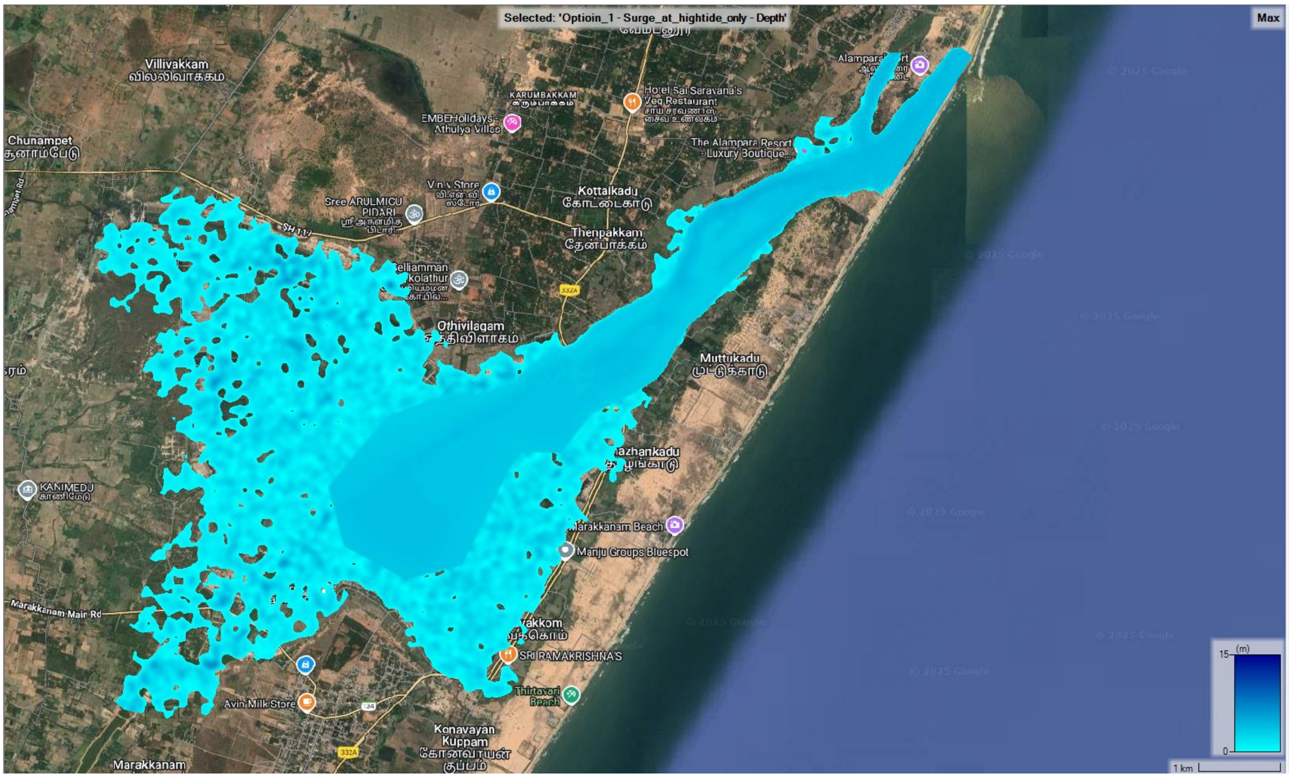

### 4.2.2 Max Flood extent map for Scenario 2(Existing Bridge) without storm surge



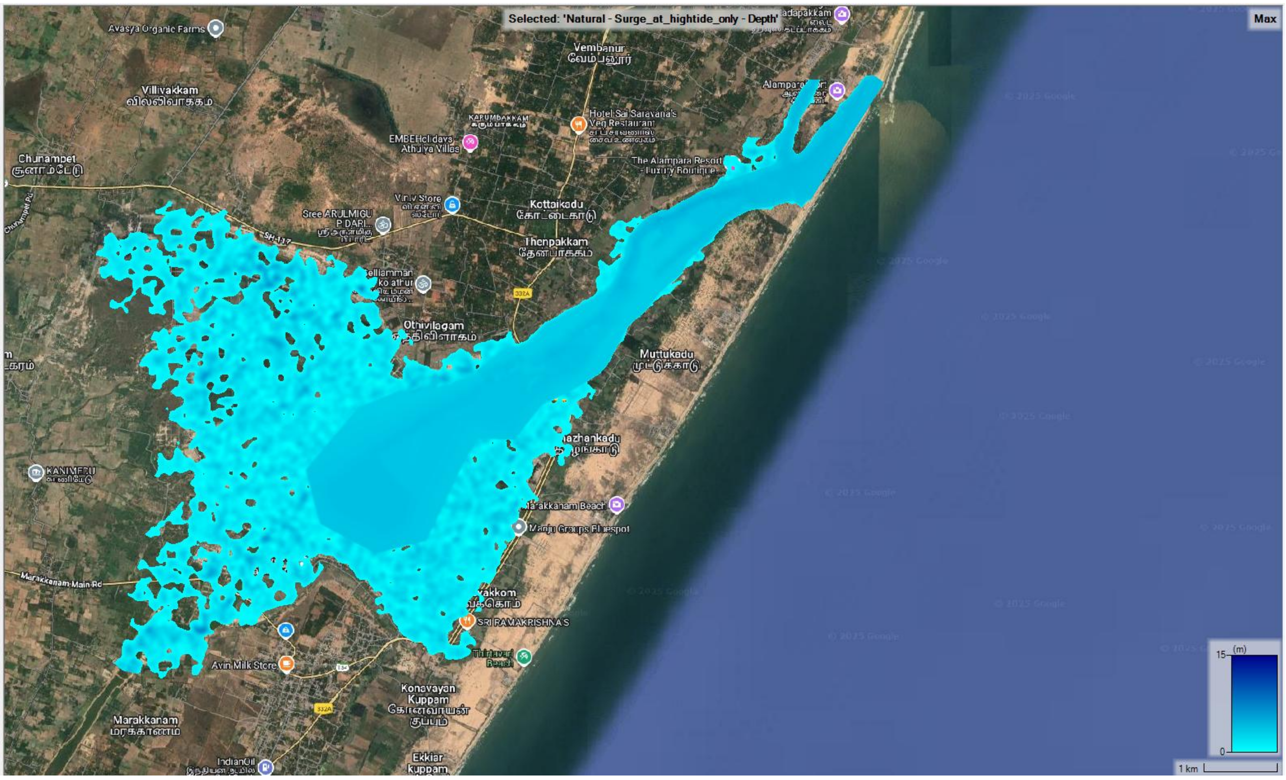
### 4.2.3 Max Flood extent map for Scenario 3(Proposed Bridge) without storm surge



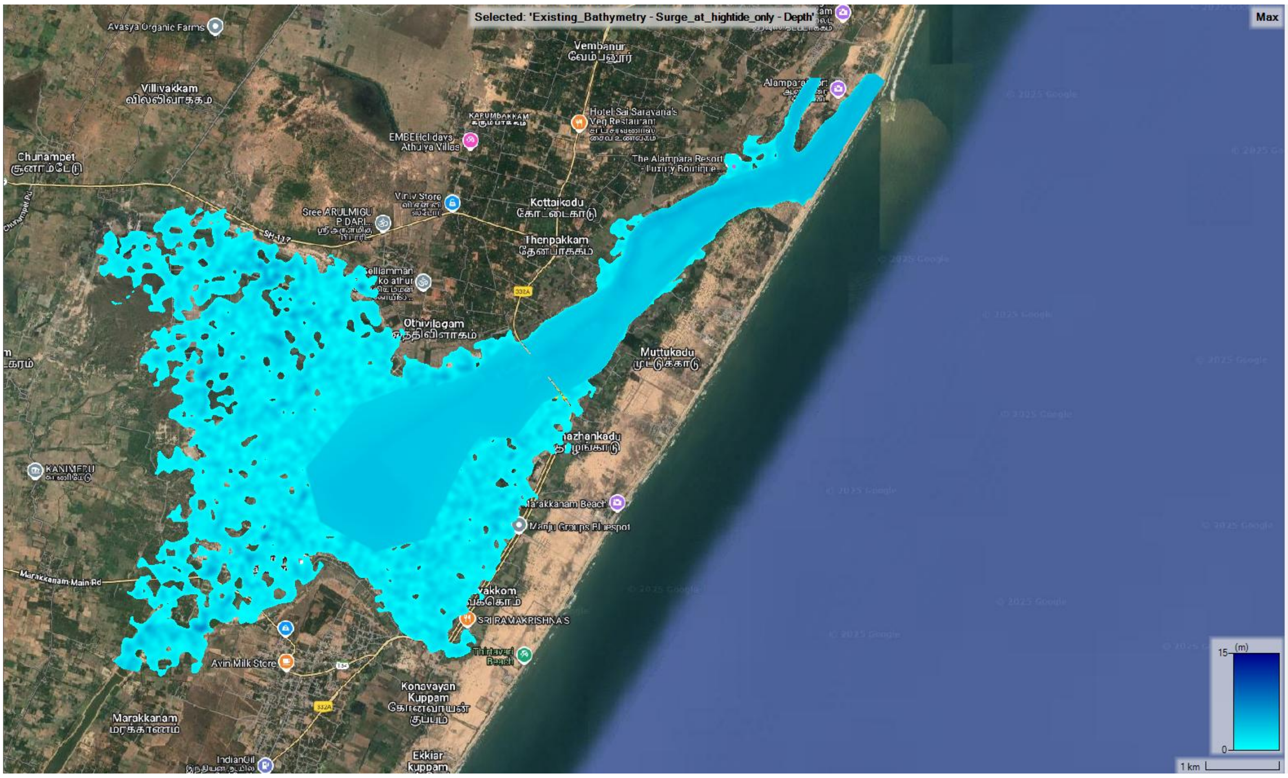

### 4.2.4 Max Flood extent map for Scenario 4(Combined Bridge along full length of estuary) without storm surge



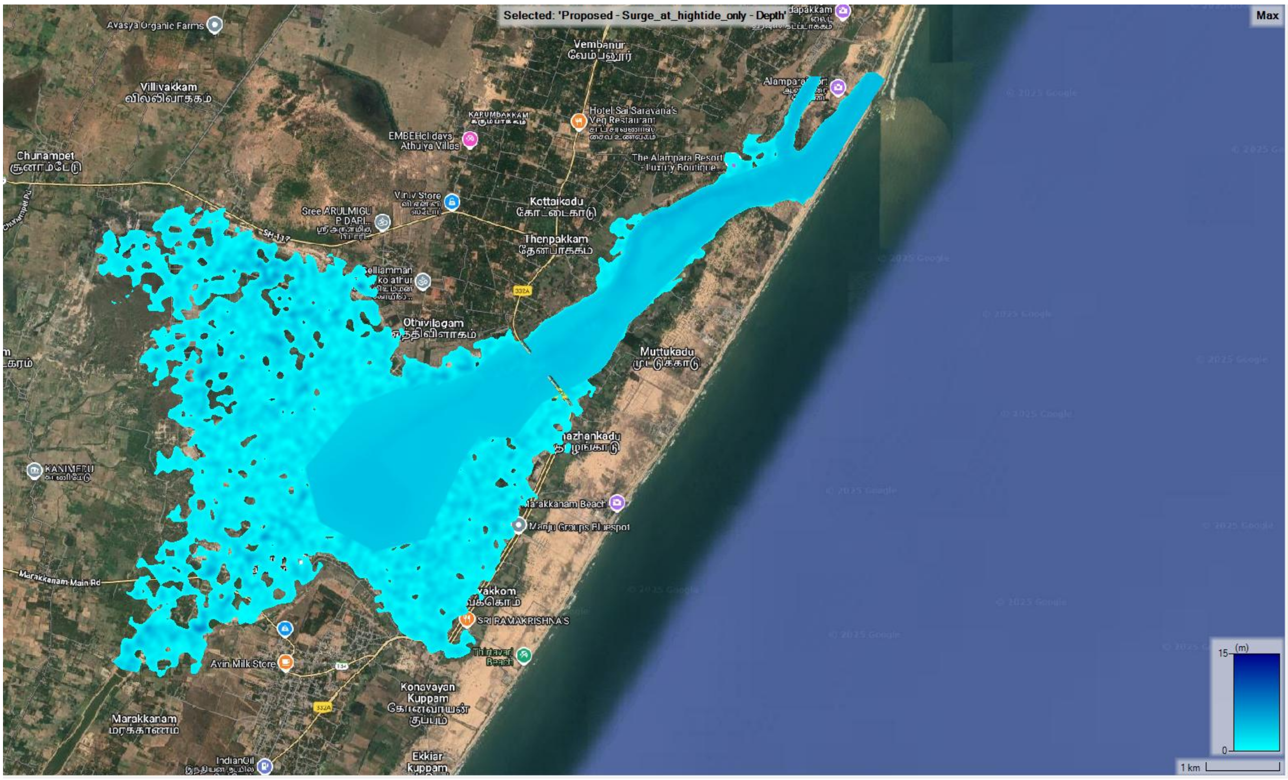
### 4.2.5 Max Flood extent map for Scenario 1(Natural Estuary) with storm surge



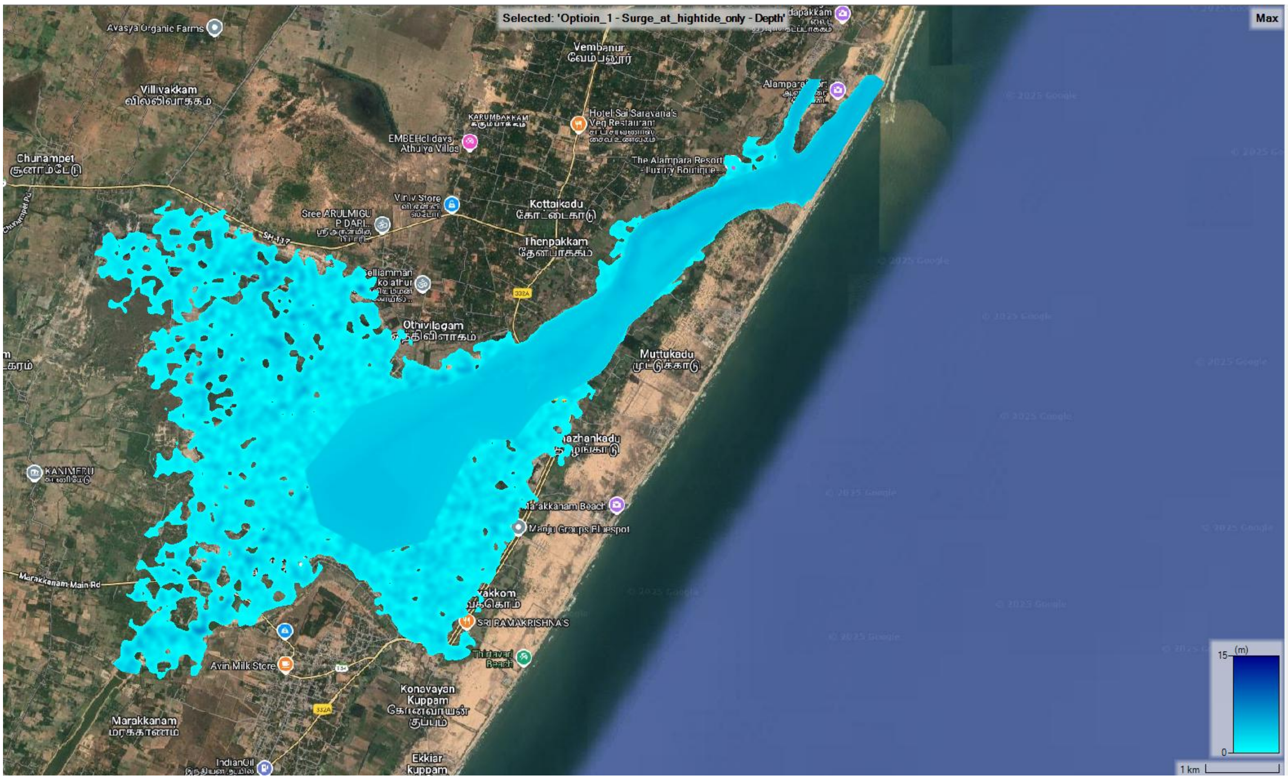

### 4.2.6 Max Flood extent map for Scenario 2(Existing Bridge) without storm surge



### 4.2.7 Max Flood extent map for Scenario 3(Proposed Bridge) without storm surge




### 4.2.8 Max Flood extent map for Scenario 4(Combined Bridge along full length of estuary) without storm surge



### 4.2.9 Comparison of results

From the figures above, it has been observed that there is no significant difference in the flood extent across all the modeled scenarios. This suggests that, regardless of the changes in bridge design and boundary conditions, the overall area affected by flooding remains relatively consistent. However, there is an increase in water levels between the different scenarios:

The maximum increase in water levels in the model, excluding the influence of storm surge, ranges from 70 mm to 80 mm between the existing, natural, and proposed cases. This small increase in water level is expected, as the presence of the bridge and embankments can slightly obstruct or alter the flow, causing a minor rise in water levels. However, the effect on the overall flow pattern was observed to be minimal.

The maximum increase in water levels in the model, including the influence of storm surge, ranges from 15 mm to 18 mm across the existing, natural, and proposed scenarios, which is lower than the values observed in the previous case. The corresponding flow velocities range from 0.9 m/s to 1.0 m/s.

These changes in water levels are summarized in the table below, providing a clear comparison across the scenarios. The relatively small changes in water levels and without appreciable change in flow pattern in various scenarios indicate that the overall hydraulic performance of the system is not significantly impacted by the proposed modifications. It is also observed that the increase in velocities are marginal and well within the maximum allowable velocities.

*Table 2 Maximum Water level in m AOD for 1 in 100-year peak flow event*

| Scenario | Downstream Boundary condition as Stage Hydrograph from observed Tide levels | | Downstream Boundary condition as Stage Hydrograph from observed Tide levels including storm surge | |
|---|---|---|---|---|
| | Water level(m) | Velocity(m/s) | Water level(m) | Velocity(m/s) |
| 1 | 1.451 | 0.430 | 2.848 | 0.968 |
| 2 | 1.521 | 0.585 | 2.863 | 1.019 |
| 3 | 1.530 | 0.580 | 2.866 | 1.073 |
| 4 | 1.455 | 0.427 | 2.848 | 0.959 |

## 5 Limitations

Any hydraulic model is a simplification of the existing situation of river reach and tidal estuary. The implementation of a hydraulic model, therefore, should always be treated with caution and it should be proven during the implementation that the model is behaving as expected, from a hydraulic modelling point of view. The following data limitations shall apply to the 2D hydraulic modelling exercise undertaken.

- SRTM data (30x30m grid) was downloaded from the free domain and used 2D model domain terrain data.
- No gauge data is available on the premises of the bridge location.
- Hydraulic models are not calibrated for historical events as there is no data available.

- There are 2 spans of 9m each bridge located 250 meters left to the existing bridge. However, since the model uses a cell size of 30 meters, even though this bridge is included in the model, its effect on the overall hydraulic behavior will likely be minimal. The relatively large cell size (30 meters) means that finer details of smaller features, like this additional bridge, may not be fully captured in the model. As a result, while the bridge is technically included in the simulation, its influence on flow dynamics, water levels, and flood extent change is not expected to be significant. The resolution of the model may be insufficient to capture any subtle effects that the bridge could have on local flow patterns or water surface elevations in that specific area. Thus, this bridge is unlikely to make a notable difference in the overall results of the simulation.

# 6 Summary & Recommendations

2D hydrodynamic flood modelling approach has been implemented during the hydraulic modelling exercise based on the readily available data. The 2D hydraulic modelling approach has also been adopted to represent the floodplain flow better in the model for scenario comparisons.

The hydraulic performance and the flooding mechanism of the river have been carefully reviewed. The results from the 2D hydraulic modelling exercise have been analysed carefully across the whole domain. The 2D model implemented can be considered to be fit for the purpose of flood mapping under the given conditions and for scenario comparisons. The embankment alignment and cross drainage structures were implemented in the hydraulic models and therefore the assessment of effect of these structures is undertaken.

There is no significant increase in water levels due to the reduction in stream width at the proposed bridge location. The existing natural stream has a width of approximately 250 m, but it discharges into the Kallivelli Tank, which has a considerably wider cross-section of about 1 km. As a result, the upstream flow tends to accumulate before the bridge, with the tank extending approximately 3.5 km upstream from the bridge location. The proposed bridge, with a waterway width of 400 m, is adequately sized to safely convey the flow from the 250 m wide stream without causing adverse upstream impacts.

Additionally, the bridge location experiences backwater effects from the sea. The distance from the downstream side of the bridge to the sea is approximately 6 km. The backwater influence from sea tide extends through a 200 m wide opening at the sea mouth. The flow from this 200 m opening tends to accumulate at the bridge location due to the presence of the upstream pond (Kallivelli Tank), which acts as a storage area, with its width varying from 1 km to 500 m in the downstream direction.

This hydraulic interaction was analysed by applying the observed tide levels combined with storm surge conditions as the downstream boundary input.

The hydraulic analysis concludes that reducing the natural tank width from 1 km to 400 m at the bridge section does not result in any significant rise in upstream water levels or pose a flood risk due to the construction of the new bridge.

The Cost of Widening the Existing two lane bridge to four lane by construction of proposed 2 Lane bridge parallel to existing bridge of length 396m is Rs 20.05 Cr, whereas construction cost of new 4 lane bridge of length 1200m is Rs 121.47 Cr. which is highly uneconomical as compared to widening.

Therefore, Widening of existing two lane bridge of length 396m to four lane configuration which is sufficient to Cater Discharge at bridge location without impacting estuary is recommended for implementation keeping in view the techno economic feasibility.

# 7 References


| | |
|---|---|
| CWC's FER | Central Water Commission's Flood Estimation Reports Subzone 4(b) |
| Storm surge | ESSO-INCOIS-Indian National Centre for Ocean Information Services |
| Software | HEC-RAS |
| Observed Tidal data | National Institute of Ocean Technology |

BEFORE THE NATIONAL
GREEN TRIBUNAL
SOUTHERN ZONE,
CHENNAI

Application No.272 of 2024
(SZ)

K. Saravanan s/o kasinathan
Aged about 37 years
30, ururkuppam, Besant Nagar,
Chennai – 90 -
Applicant

Vs

1. Tamil Nadu Coastal Zone
Management Authority

By its Member Secretary

1, Jennis Road, Panagal Building

Ground Floor, Saidapet, Chennai
– 600 015

2. The National Highways
Authority of IndiaRep by its
Project

- Respondents

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