IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE)
e-ISSN: 2278-1684,p-ISSN: 2320-334X, Volume 8, Issue 1 (Jul. - Aug. 2013), PP 89-96
www.iosrjournals.org

Adequacy Check of Existing Crest Level of Sea Facing Coastal Polders by the Extreme Value Analysis Method

Md. Saiful Islam [1], Md. Zahirul Haque Khan [2], Rubayat Alam [3],
Md. Nasim Al Azad Khan [4], Ismat Nur-A-Jahan[5]

[1,4,5] *Junior Engineer, Coast, Port and Estuary Management Division, Institute of Water Modelling, Bangladesh*
[2] *Director, Coast, Port and Estuary Management Division, Institute of Water Modelling, Bangladesh*
[3] *Senior Specialist, Coast, Port and Estuary Management Division, Institute of Water Modelling, Bangladesh*

Abstract: *The coastal embankment system has been gradually built during the last 40 years. The embankments were originally designed to increase agricultural production by preventing salt water intrusion not to protect against cyclonic storms. The alignment of the embankments did not consider the changing conditions in bathymetry of the sea and thalweg migration of the rivers and therefore many embankments are located under tidal water level and have severe toe and slope erosion problems during the monsoon season. The crest level and embankment cross sections have not optimized the protection of hinterland and the embankment itself and therefore the embankments typically only provide protection for the cyclones with 5-12 year return periods and the designed crest level of the sea facing coastal polder equal to the sum of normal maximum recorded water stage plus 1.50m. In this study to estimate the design crest level and side slope for sea facing embankment have been established based on maximum storm surge level, wave run-up for cyclonic wave, freeboard allowing 5 l/m/s overtopping ,potential climate change impact and land subsidence. Statistical analysis of surge level and wave run-up is carried out using Extreme Value Analysis (EVA) in MIKE Zero.*

Keywords: *Cyclonic wave, Overtopping Storm surge, Tidal surge, Wave Run-up*

I. INTRODUCTION

In recent years cyclones Aila and Sidr hit the south western coastal zone only reminding that the whole coastal zone is vulnerable to cyclones and storm surges. During the period 1960-2009, 19 severe cyclones have hit the coast of Bangladesh. Most of the cyclones hit the coasts of Bangladesh with north-eastward approaching angle. Surge wave generated at the deeper sea is driven towards the coast by the wind and propagates over land being amplified near the coast. The coastal areas around the Meghna estuary are one of the most vulnerable areas that experience very high surge attack. Very high surge height is caused by the shoaling effect in shallower zone, funneling effect of the land geometry and sometimes coinciding with tidal motion. As a result, the surge level at the north of Sandwip Island, which is the tip of the converging land geometry, attains the highest elevation in most of the cases. The Bay of Bengal is one of the favourable areas for the generation of tropical cyclones. About one-tenth of the global total cyclones forming in different regions of the tropics occur in the Bay of Bengal (Gray, 1968; Ali, 1980). Not all of the tropical cyclones formed in the Bay of Bengal move towards the coast of Bangladesh. About one-sixth of tropical storms generated in the Bay of Bengal usually hit the Bangladesh coast.

Embankments in the coastal region are exposed to cyclone surge and related waves. Such embankments are ideally constructed as “retired” embankments at some distance from the high-tide coastline. The performance of such embankments during cyclones is determined by the crest elevation and the S/S and C/S slopes compared to the elevation of the surge level and wave height during cyclones. The water level during cyclones is determined by the cyclone surge, but also by seasonal variation, tide and water level setup by breaking short waves. In the following we will refer to water levels taking into account seasonal variation, tide and cyclone surge as the surge level. The system of coastal embankments was initially constructed during the Sixties. The design used for the construction was “institutionalised” in a study by Leedshill DeLeuw Engineers (LDL) completed in 1968.

The purpose of the embankments was to enable cultivation of land in areas otherwise exposed to tidal flooding of saline water. The embankments were designed with no regard to cyclone surges, and very little regard to waves. Crest height of the embankments was determined as “the maximum normal high tide” (as recorded during 1960-1968) plus a freeboard of 5 feet. In this study storm surge model and wave model has been simulated considering the following climate change conditions based on IPCC projections from 2007 (Synthesis Report, 2009)

- Sea level rise of 0.50 m and
- 10% increase in maximum wind speed of cyclone

II. DEVELOPMENT OF MODEL

2.1 *Storm surge modelling*

Storm surge model comprises of Cyclone model and Hydrodynamic model. The existing Bay of Bengal Model has been applied in this study for storm surge modelling. The storm surge model is the combination of Cyclone and Hydrodynamic models. For simulating the storm surge and associated flooding, Bay of Bengal model based on MIKE21 hydrodynamic modelling system has been adopted. In the hydrodynamic model simulations meteorological forcing due to the cyclone has been given by applying wind and pressure fields derived from the analytical cyclone model. The MIKE 21 modelling system includes dynamic simulation of flooding and drying processes, which are very important for a realistic simulation of flooding in the coastal area and inundation.

2.1.1 *Hydrodynamic model*

The model is two-way nested and includes four different resolution levels in different areas. The coastal region of Bangladesh and the Meghna estuary are resolved on a 200 m grid. The model has two open boundaries; one is in the Lower Meghna River near Chandpur and another one is in the open sea located along the line extending from Vishakhapatnam of India to Gwa Bay of Myanmar. In the river boundary measured water level has been prescribed and in the sea boundary has been generated from Global Tide Model.

2.1.2 *Cyclone model*

The description of a cyclone is based on few parameters related to the pressure field, which is imposed to the water surface and a wind field which is acting as a drag force on the water body through a wind shear stress description. The pressure field creates a local level setup close to the eye up to one metre only. Whereas the wind shear contributes more to the surge giving a level setup on the right side of the eye and a level set down on the left side. To generate the wind field, Holland Single Vortex theory has been applied. The cyclone model needs following data/information for the description of wind field and pressure field:
(1)Radius of maximum winds, Rm, (2) Maximum wind speed, Vm (3) Cyclone track forward speed Vf and direction. (4) Central pressure, Pc, (5) Neutral pressure, Pn (6) Holland Parameter, B= 2.0-(Pc-90)/160

2.2 *Development of Wave Model*

Flexible mesh used for the Bay of Bengal model has been further updated for wave simulation. The model domain has been extended further. Finer resolution has been used in deep sea. Cyclonic wave generated from cyclonic wind and cyclonic wind originated far away from the Bangladesh coast. Considering generation locations of nineteen severe cyclones, the model domain for cyclonic wave simulation extended up to 4 degree latitude.

III. MODEL RESULT ANALYSIS

3.1 *Frequency Analysis of Storm Surge levels for Different Return Period*

Frequency analysis is carried out to find the storm surge level for different return periods along the sea facing embankment to investigate the performance of existing coastal embankment. The present crest level of the embankments of coastal polder is based on extreme tide level with some free board added. Storm surge usually exceeds this embankment level. Under the present study, time series storm surge levels in the sea facing coastal polders at different locations have been generated from simulation results. There are 19 (nineteen) severe cyclones that hit the coastal area from 1960 to 2009. These cyclones made landfall at different tidal phases i.e. either at low tide or at high tidal phase. All these 19 cyclones were simulated at original tidal phase and then opposite tidal conditions, i.e. if a cyclone made landfall on low tide then both low tide and high tidal conditions of each cyclones were simulated. Total 38 cyclone tracks for the whole costal area have been considered based on 19 observed cyclones. In order to generate time series storm surge level, 38 cyclones have been considered under baseline condition and climate change scenario. Maximum surge levels for every cyclone have been extracted at the locations of interest in Fig III.*1*.

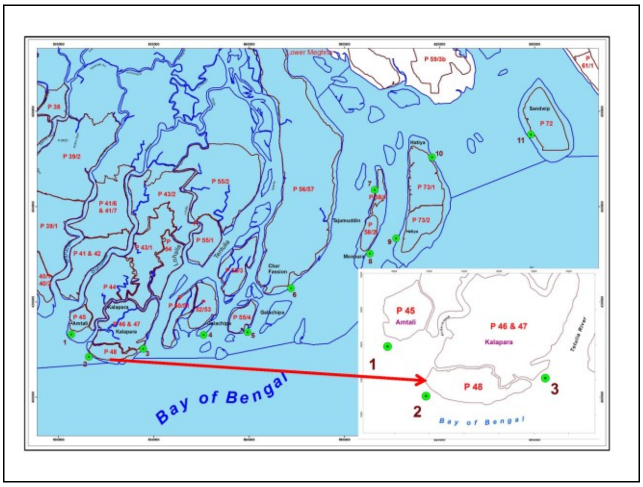

Fig III.1: Locations of storm surge level for 25 year return period Storm surge height

These surge level values are analysed to determine the 25-yr return period for all the locations. Statistical analysis of surge level is carried out using Extreme Value Analysis (EVA) in MIKE Zero. For evaluating the risk of extreme events a parametric frequency analysis approach is adopted. This implies that an extreme value model is formulated based on fitting a theoretical probability distribution to the observed extreme value series.

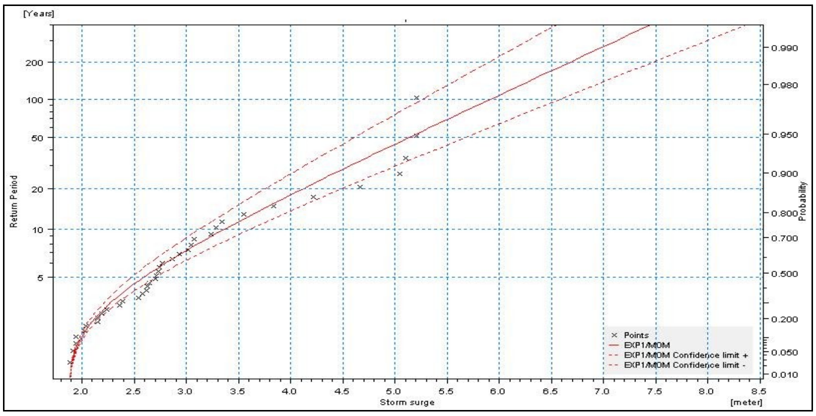

Fig III.2: Statistical analysis of Strom Surge Level by Exponential Distribution for the Point No. 2.

Two different extreme value models are provided in EVA, the annual maximum series (AMS) method and the partial duration series (PDS) method, also known as the peak over threshold (POT) method. For this study PDS method has been applied. For estimations of the parameters of the probability distribution, method of momentum has been used. In this study exponential distribution function has been used shown in Fig III.1.

3.2 *Statistical analysis of wave and determination of design wave height*

The wave heights generated by the cyclonic wind speeds are quite considerable along the sea-facing dykes in the south. To know the wave characteristic during cyclone, 19 wave simulations were carried out using the cyclonic wind and pressure field for 19 naturally occurred cyclones. The domain of wave model for cyclonic wave up to 4 degree latitude to cover the whole area including these areas where cyclonic wind was first generated. The maximum significant wave height was obtained from the pre-selected locations in the model domain, for each cyclone of the 19 wave simulations. The 19 values obtained for each location were then analysed to obtain the 25 years return period significant wave height. Statistical analysis of significant wave height is carried out using Extreme Value Analysis (EVA) in MIKE Zero. Same procedure are applied to determine extreme value for wave as applied in determining for surge level but here Weibel distribution function has been used shown in Fig III.3.

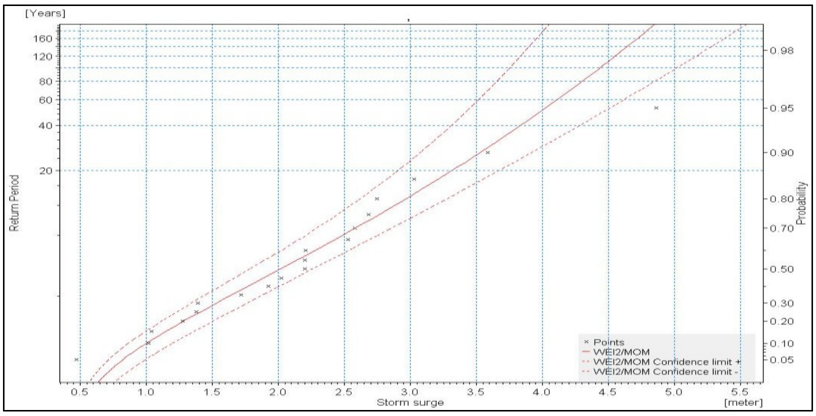

Fig III.3: Statistical analysis of Significant Wave Height by Weibel distribution function for the Point No.2

IV. EMBANKMENT CREST LEVEL CALCULATION

4.1 *Wave run-up and freeboard for embankment crest level*

Wave generated during nineteen severe cyclones is significant and causes considerable wave run-up and overtopping during cyclone which are important for designing embankment crest level and slope. The wave run-up height is z number of incoming waves at the toe of the structure. The idea behind this was that if only 2% of the waves reach the crest of a dike or embankment during design conditions, the crest and inner slope do not need specific protection measures other than clay with grass. It is for this reason that much research in the past has been focused on the 2%-wave run-up height.

Wave overtopping is the mean discharge per linear meter of width, q, for example in m3/s/m or in l/s/m. In reality, there is no constant discharge over the crest of a structure during overtopping. The process of wave overtopping is very random in time and volume. The highest waves will push a large amount of water over the crest in a short period of time, less than a wave period. Lower waves will not produce any overtopping.

Significant wave height for different return period during 19 severe cyclones was determined. For designing embankment crest level, wave run-up caused by 25-yr return period significant wave height, was predicted. Wave run-up was computed using the methodology given in the Eurotop Manual (2007). Free board for embankment crest level was determined allowing 5 l/m/s overtopping.

Example of Wave run-up and freeboard calculation:

Point No. 2, Located at Polder-48

Table IV.1: Wave run-up and wave overtopping calculation

Wave run-up height and wave overtopping					
Based on EurOtop Manual, chap 5, August 2007					
	Parameter	**Value**	**Unit**	**Description**	**Notes**
Input					
Run-up	Hm0	3.48	[m]	Significant wave height	Hm0 in front of the structure
	Tp	6.26	[s]	Peak wave period	Associated Tp
	n	7.00		1/n: Slope of front of structure	
	B	0.00	[m]	Berm width	Berm width - Set to 0 if no berm.
	Lberm	1.00	[m]	Berm Length	Berm length - see figure 5.27 (b). Set to arbitrary number > 0 if no berm.
	db	0.00	[m]	Depth at berm	>0 if berm lying below still water level, <0 if berm lying above still water level
	Ru2% estimate	4.20	[m]	Estimate of Ru2%	Used for calculation of berm influence factor for db<0 (use iteratively)
	γf	0.80		Slope roughness factor	=1.0 for grass (ref table 5.2)
	β	45.0	[deg]	Angle of attack	=0 for perpendicular waves. No effect if <20.
Overtopping	RC	1.78	[m]	Freeboard	Vertical difference between still water level and crest height
extras	γv	1.00		Vertical wall factor	Ranges from 0.65 for a vertical wall to 1.0 for a 1:1 slope (see page 94)
Calculations					

Wave run-up height and wave overtopping					
Based on EurOtop Manual, chap 5, August 2007					
	Parameter	**Value**	**Unit**	**Description**	**Notes**
Run-up	tan(α)	0.14		Slope of front of structure	tan(α) = 1/n
	Tm-1,0	5.69	[s]	Spectral wave period	Tm-1,0 = Tp / 1.1
	g	9.81	[m/s**2]	Gravitational acceleration	Constant
	Lm-1,0	50.6	[m]	Deep water wave length	Lm-1,0=g*Tm-1,0^2/(2*PI)
	sm-1,0	0.07		Wave steepness	Hm0/Lm-1,0
	ξm-1,0	0.55		Breaker parameter, Xi	ξm-1,0 = tan(α)/(sm-1,0)**0.5. Surf similarity or Iribarren number. Xi<2-3 for breaking waves
	rB	0.00		Berm reduction, first part	=B/Lberm
	rdb	0.60		Berm reduction, second part	db>0: 0.5-0.5*cos(pi*db/Ru2%) db<0: 0.5-0.5*cos(pi*db(/2*hm0)). Always >=0.6
	γb	1.00		Berm influence factor	Between 0.6 and 1.0
	γβ	0.90		Oblique wave attack factor	=1-0.0022*abs(β). =1 for 0<β<20deg. =0.824 for abs()>80deg
	Rel Ru2%	0.69		General wave run-up height	Relative wave run-up height, equation 1
	Max Rel Ru2%	1.54		Max wave run-up height	Relative maximum wave run-up height, equation 2
	Rel Ru2%	0.69		Relative wave run-up height	Wave run-up relative to Hm0 (minimum of equation 1 and equation 2)
Overtop ping	γβ	0.85		Oblique wave attack factor	=1-0.0033*abs(β). =1 for 0<β<20deg. =0.736 for abs()>80deg
extras	Rel q	0.0003		General wave overtopping	Relative wave overtopping height, equation 3
	Max Rel q	0.035		Max wave overtopping	Relative maximum wave overtopping height, equation 4
	Rel q	0.0003		Relative wave overtopping	Wave overtopping relative to sqrt(g*Hm0^3) (minimum of equation 3and equation 4)
Results					
Run-up	Ru2%	**1.8**	[m]	Wave run-up height	Final result = Hm0 * Rel Ru2%
Overtop ping	q	**5**	[l/s/m]	Overtopping volume per m	Final result = q*sqrt(g*Hm0^3)

(Freeboard is calculated by Trial and Error method for overtopping 5l/s/m)

Governing Equation of wave run-up height and wave overtopping:
The governing equation (1) and (2) is used to calculate the Wave run-up relative to Hm0.

General wave run-up height (Rel Ru2%):
Relative wave run-up height

$$\frac{Ru2\%}{Hm0} = 1.75 * \gamma b * \gamma f * \gamma\beta * \xi m - 1,0 = 0.69 \qquad \text{---------} (1)$$

Max wave run-up height (Max Rel Ru2%):
Relative maximum wave run-up height

$$\frac{Ru2\%}{Hm0} = 1.00 * \gamma f * \gamma\beta * \left(4.30 - \frac{1.60}{\sqrt{\xi m - 1,0}}\right) = 1.54 \qquad \text{---------} (2)$$

Wave run-up relative to Hm0=0.69 (minimum of equation (1) and equation (2)

The governing equation (3) and (4) is used to calculate the Wave overtopping height.

General wave overtopping (Rel q):
Relative wave overtopping height

$$\frac{q}{\sqrt{g * \mathrm{Hm0}^{\wedge}3}} = \frac{0.067}{\sqrt{tan \propto}} * \gamma b * \xi\mathrm{m} - 1{,}0 * \exp\left(-4.75 * \frac{\mathrm{Rc}}{\xi\mathrm{m} - 1{,}0 * \mathrm{Hm0} * \gamma\mathrm{b} * \gamma\mathrm{f} * \gamma\beta * \gamma\mathrm{v}}\right) = 0.0003 \quad \text{---- (3)}$$

Max wave overtopping (Max Rel q):

Relative maximum wave overtopping height: with a maximum of

$$\frac{q}{\sqrt{g * \mathrm{Hm0}^{\wedge}3}} = 0.2 * \exp\left(-2.6 * \frac{\mathrm{Rc}}{\mathrm{Hm0} * \gamma\mathrm{f} * \gamma\beta}\right) = 0.035 \quad \text{--------- (4)}$$

Wave overtopping relative to sqrt(g*Hm0^3) =0.0003(minimum of equation (3) and equation(4).

The value of wave run-up height and wave overtopping height is used to calculate the free board which is important parameter for the calculation of polder crest level.

4.2 *Design Embankment crest level*

The design crest level and side slope for embankment have been established based on maximum storm surge level, maximum monsoon water level freeboard allowing 5 l/m/s overtopping , potential climate change impact and land subsidence. Considering the sea facing polders have been designed based on maximum storm surge level and wave run-up for cyclonic wave. After considering the results of the storm surge and the project life, it was decided that the design return period should be 25 years. The crest levels of the sea facing polder are design based on the modelling outcomes described in the preceding sections:

1. The 25 years return period storm surge level
2. Alternatives for freeboard depending on overtopping limit of 5l/m/s for several possible embankment slopes and roughness.
3. Allowance for subsidence

The table shows how the crest levels were computed for eleven locations (Sea facing polders). Because of the high wave action we have also considered 1:7 slope in the wave run up computations. Column 3 shows the original LDL design level) in Old PWD. These levels are not directly convertible to the present New PWD values. All other levels shown in the table are for New PWD datum. In this context it should be noted that there are other safety factors built into this methodology. The 25 year return period is applicable to the environment with climate change effects in year 2050. In earlier years the design would have a higher degree of protection. In this research it is assumed that the highest wind driven waves will occur exactly when the maximum surge occurs – this also introduces a further factor of safety. Wave overtopping is intermittent with only the larger waves running over the embankment crest.

The 25 years return period storm surge level has been calculated from the statistical analysis. In this study exponential distribution function has been selected. Wave run-up and freeboard calculation have been shown in

*Table IV.1.*The existing and design crest level of sea facing polders have been compared to SIDR (2007) which was devastating for the coastal region of Bangladesh. New methodology for the crest level design of sea facing coastal polders has been shown in Table *IV.2*.

Table IV.2: New methodology for the crest level design of sea facing coastal polders

Point_No.	Location	Design Levels (LDL)	Existing Ave. Crest Level (mPWD)	Modelled Storm Surge level (mPWD)	Standard Deviation (m)	Sidr Simulated	Recommended Slope	Free board for Grass or Smooth paved Roughness 1.0	Free board for Slope Roughness 0.8	Allowance for Subsidence	Rqd crest Level w/o roughness no std	Rqd crest Level w/o roughness + std	Rqd crest Level w roughness & no std	Rqd crest Level w roughness + std
1	P-45	4.57	5.8	4.088	0.358	5.27	1:7	2.25	1.78	0.3	6.64	7.00	6.17	6.53
2	P-48	4.23(SRP)	6.1	4.353	0.405	5.05	1:7	2.3	1.8	0.3	6.95	7.36	6.45	6.86
3	P-46	4.57	5.18	4.695	0.475	6.31	1:7	2.4	1.9	0.3	7.40	7.87	6.90	7.37
4	P-52/53	4.57	5.18	4.974	0.516	5.46	1:7	2.9	2.3	0.3	8.17	8.69	7.57	8.09
5	P-55/4	4.87	4.92	3.86	0.329	5.01	1:7	1.25	1	0.3	5.41	5.74	5.16	5.49
6	P-56/57	5.80	5.8	5.827	0.697	5.34	1:7	2.35	1.85	0.3	8.48	9.17	7.98	8.67
7	P-58/1	4.87	5.2	4.921	0.493	5.07	1:7	2.8	2.25	0.3	8.02	8.51	7.47	7.96
8	P-58/2	4.87	5.5	4.771	0.491	5.04	1:7	2.95	2.35	0.3	8.02	8.51	7.42	7.91
9	P-73/2	6.00	6.3	5.042	0.498	5.49	1:7	2.3	1.8	0.3	7.64	8.14	7.14	7.64
10	P-73/1	5.50	6.3	5.574	0.503	5.10	1:7	4	3.25	0.3	9.87	10.38	9.12	9.63
11	P-72	6.70	7.0	6.783	0.617	5.76	1:7	2.15	1.7	0.3	9.23	9.85	8.78	9.40

Source: Technical Feasibility Studies & detailed Design for Coastal Embankment Improvement Project, 2012[5]
Where,
LDL Coastal Embankment Project, Engineering and Economic Evaluation, Volume 2, Polder Maps, Leedshill-De Leuw Engineers, December 1968.
SRP Systems Rehabilitation Project, Halcrow, 1994.

The methodology has been developed according to the "Technical Feasibility Studies & detailed Design for Coastal Embankment Improvement Project (CEIP)", 2012 conducted by Institute of Water Modeling (IWM).

There are other sea facing polders which crest levels are not calculated here. It is seen that the existing crest level of all sea facing polders is not adequate to face natural hazards: cyclones and tidal surges, salinity intrusion, riverbank erosion, shoreline recession, tsunami etc. So Government should take necessary steps to rehabilitation of the sea facing polders to face some natural hazards. The Comparison of crest level of sea facing coastal polders between new and previous methodologies has been shown in

Table IV.3.

Table IV.3: Comparison of crest level of sea facing coastal polders between new and previous methodologies

Point No.	Location	Existing Slope(Sea Side)	Design Slope(Sea Side)	Existing Ave. Crest Level (mPWD)	Design Crest Level	Total Length(km)	Sea Side Length(km)	Established Year	Area Protected during Established Year(acres)

1	P-45	1:3	1:7	5.8	6.75	27	10.56	1966-67	10,100
2	P-48	1:7	1:7	6.1	7.00	38	19.99	1964-66	11,600
3	P-46	1:3	1:7	5.18	7.50	40	5.20	1966-69	13,300
4	P-52/53	1:7	1:7	5.18	8.15	59	17.59	1970-73	20,100
5	P-55/4	1:7	1:7	4.92	5.50	33	32.53	1972-74	12,800
6	P-56/57	1:5	1:7	5.8	8.75	250	123.00	1966-70	180,800
7	P-58/1	1:7	1:7	5.2	8.00	32	27.00	1971-73	8,900
8	P-58/2	1:7	1:7	5.5	8.00	28	27.73	1973-74	82,000
9	P-73/2	1:2	1:7	6.3	7.75	48	34.62	1968-69	27,500
10	P-73/1	1:2	1:7	6.3	9.75	80.24	10.31	1964-68	52,800
11	P-72	1:3	1:7	7	9.50	56.93	45.12	1968-69	56,000

Source: Coastal Embankment Rehabilitation Project. Hydraulic Modeling study, Prepared by IWM/DHI, 2001[3]

V. CONCLUSION

Historical records show that more than 19 severe cyclones are generated in the Bay of Bengal in every ten years in average. Several of them strike the coast of Bangladesh. Extremely strong storm surges with more than 10m of water elevation hit the coast of Bangladesh at least once in a century. Very strong surge attack results in emotional, physical and economical catastrophe of the country. Considering the complexities of the Bangladeshi coastline with numerous inlets, large estuary, offshore islands and chars, use of sufficiently high-resolution model is necessary to generate the surge dynamics realistically in the shallower region. Sensitivity of shallowness of coastal water to surge height underlines the importance for using adequately resolved bathymetry in surge calculation for the sea facing coastal polders of Bangladesh. Considering the vulnerabilities in the coastal polders, it is likely that the physical interventions under the study would include some combination of (a) rehabilitating and raising the height of the earthen embankments considering the above procedure; (b) afforestation of embankments on the crest and slopes to maintain the coastal polders properly.

Acknowledgements

This work is funded by World Bank in collaboration with Institute of Water Modelling (IWM) and Bangladesh Water Development Board (BWDB). The authors are thankful to all professionals of Coast Port and Estuary Management Division, Institute of Water Modelling.

REFERENCES

[1] IWM, Multipurpose Cyclone Shelter Programme. Planning and Implementation issues, Part 4 Volume IV, prepared for Planning Commission, Government of Bangladesh and United Nations Development Programme/ World Bank, July 1993
[2] IWM, Cyclone Shelter Preparatory Study. Stage 1: Feasibility Phase, Final Report, Prepared for European Commission, Directorate General External Economic Relations and Technical Unit for Asia, ALA-91-16 Bangladesh, May 1998
[3] IWM/DHI, 2nd Coastal Embankment Rehabilitation Project. Hydraulic Modelling Study, August 2001
[4] IWM, Islamic Development Bank, Storm Surge Modelling Study for Construction of Cyclone Shelters in the Coastal Region of Bangladesh under Fael Khair Program, Draft Final Report, August 2011
[5] IWM, Hydraulic and Morphological Modelling Study to Aid "Technical Feasibility Studies & detailed Design for Coastal Embankment Improvement Project (CEIP), Final Report, 2013