Development of National Flood Protection Plan-IV (NFPP-IV) and Related Studies to Enhance Capacity Building of Federal Flood Commission-FFC

DESIGN CRITERIA FOR FLOOD PROTECTION BUNDS, SPURS, STUDS AND FLOOD RETAINING WALLS

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LIST OF ABBREVIATIONS/ACRONYMS

ADB	Asian Development Bank
AJ&K	Azad Jammu and Kashmir
DEM	Digital Elevation Model
DSS	Decision Support System
DTM	Digital Terrain Model
FATA	Federally Administrated Tribal Areas
FEWS	Flood Early Warning System
FFC	Federal Flood Commission
FPSP	Flood Protection Sector Project
GB	Gilgit Baltistan
GCP	Ground Control Point
GIS	Geographical Information System
GoP	Government of Pakistan
HEC-HMS	Hydrological Engineering Centre - Hydrologic Modeling System
HEC-RAS	Hydrological Engineering Centre - River Analysis System
HFL	Highest Flood Level
IRSA	Indus River System Authority
KP	Khyber Pakhtunkhwa
LAN	Local Area Network
NESPAK	National Engineering Services Pakistan
NFPP	National Flood Protection Plan
NGO	Non-Governmental Organization
PC	Planning Commission
PID	Provincial Irrigation Department
PMD	Pakistan Meteorological Department
PMPIU	Project Management and Policy Implementation Unit
RS	Remote Sensing
SDLC	Software Development Life Cycle
SoP	Survey of Pakistan
SPOT	Système Pour l'Observation de la Terre (System for Earth Observation)
SRTM	Shuttle Radar Topography Mission
SUPARCO	Pakistan Space and Upper Atmosphere Research Commission
SWH	Surface Water Hydrology
ToR	Terms of Reference
WAPDA	Water and Power Development Authority
WCAP	Water Sector Capacity Building and Advisory Services Project
WRD	Water Resources Division of NESPAK

DESIGN CRITERIA FOR FLOOD PROTECTION BUNDS, SPURS, STUDS AND FLOOD RETAINING WALLS

1. GENERAL

Low lying flood plains have attracted people to settle for hundreds of years. Flood plains provide fertile areas to grow crops. Rivers form navigation routes to carry goods. This all leads to an increase in population in and around flood plains over the years. Many large cities are located in and around an adjacent river's flood plain. Rivers flood periodically when the flow increases and breaches the natural river banks.

It is the worldwide practice that hydraulic structures are being constructed at suitable locations on rivers. For constructing a hydraulic structure across a river, a water resources engineer must also consider the effect of the structure on the hydraulics of the river and the best ways to train the river such that the structure performs satisfactorily and also there is no significant damage to the riverine environment.

Destructive floods, both large and small have been taking a heavy toll of life and property in Pakistan. The catastrophic flood events in the recent past and consequent losses to the national economy have triggered action to deal with flood problems on a comprehensive basis rather than to continue with the traditional crisis provoked approach. Whereas, complete prevention of floods is almost a physical impossibility, flood protection to the extent technically and economically feasible, is a socioeconomic necessity. With proper planning, means can be devised out, not only to reduce flood losses but also to conserve the surplus flood waters for augmenting water availability for productive uses and to promote welfare for the community.

Pakistan is traversed by the mighty Indus, its four eastern rivers of Jhelum, Chenab, Ravi and Sutlej and also a large number of big and small tributaries including the hill torrents, a major portion of the catchment of the main rivers lies in the high mountains where due to steep grades, heavy and sudden run-off is generated by melting of snow and the rainfall. These high flows inundate a large area in the plains because of flat topography. The numerous hill torrents bordering the plains also bring flashy floods which too contribute towards the creation of flood situation enroute. The problem is diversified in nature and varies from area to area depending upon the physical, hydrological and socio-economic conditions. The more severe flood problem is however encountered during the monsoon season from July to September.

The flood protection bunds have been generally constructed either to protect head works and other irrigation structures or to safeguard certain towns and cities. The construction had been done mostly by manual labor and not resorting to any compaction as is being practiced in the modern times. The old bunds are therefore comparatively much weaker in strength in relation to the recent construction done with the machinery.

The phenomena of floods in Pakistan are periodic one. The super floods of the year 1973 caused large scale devastation and for quite a while the normal life in the country remained paralyzed. The rail-road traffic in the flood stricken area was disrupted and the canal water supply to a large area remained cut off for a considerable length of time. While provision of the remedial measures was still underway, the floods of the year 1976 (which too were of unprecedented nature) clearly demonstrated existence of major deficiencies in the system and high-lighted the need to tackle the problem on a national level. These were followed by the major floods of 1976, 1988, 1992 and have caused widespread destruction. The devastation caused by recent floods of 2010, 2012 in River Indus and the most recent of 2014 in Rivers Jhelum and Chenab demands for a long term strategy to cope with the future floods.

Keeping in view the devastation and destruction caused by past flood events, it is imperative that the flood protection structures may be designed in accordance with the flood magnitude and extents. The existing flood protection and river training structures must be rehabilitated and redesigned to help keep safe against expected future flood events. Bunds and embankments where necessary must be raised to avoid flood breaching and overtopping in case of existing structures.

2. SCOPE

The scope of this document includes:

i) review of existing design criteria, national flood protection plans and other relevant documents/reports which formed the basis for design of existing flood protection structures
ii) Redevelopment of criteria for design and construction of new flood structures (embankments, bunds, spurs, studs, retaining walls etc.)
iii) redevelopment of evaluation criteria for assessment of existing flood bunds
iv) evaluation of existing flood protection structures to assess adequacy and safety against design flood
v) to suggest raising and strengthening of flood protection structures to meet the developed design criteria

3. REVIEW OF EXISTING DESIGN CRITERIA AND REPORTS

In order to assess existing flood protection structures and to propose new ones, the review of existing design criteria and relevant reports/documents has been taken up under this study. Evaluation of practices and procedures adopted for the construction of existing flood protection structures is also an important element for the review of these criteria and reports.

Following documents have been reviewed for the assessment of existing flood protection structures (embankments, bunds, spurs, studs, retaining walls etc.):

3.1 National Flood Protection Plan 1978 (NFPP-I)

The National Flood Protection Plan of 1978 was the first planning document regarding flood management for Punjab, Sindh, Balochistan and Khyber Pakhtunkhwa (KP). The planning, design and construction practices of flood protection structures for all provinces of Pakistan based upon the knowledge and resources available at that time was discussed in detail in this document.

Design Criteria for Bunds

The design practice for flood bunds in Sindh and Balochistan was based on the criteria set forth by Indus River Commission and documented in Bund Manual, Sindh. In Punjab and KP (formerly NWFP), the criteria followed were based on instructions contained in Manual of Irrigation Practice (1963). The circulars prepared after 1973 and 1976 flood events resulted in major revisions to the manual and were abstracted in the Revised Flood Protection Plan for Punjab, June 1978.

The criteria for design of flood protection structures considered the adequacy of freeboard against overtopping, adequate top and base width, stable land and river side slopes, prevention of water percolation to avoid slippage and stone pitching to avoid wave wash damage.

Due to the absence of detailed knowledge and computer software assistance, the hydraulic gradient of the phreatic line was assumed as 1V:6H for all flood protection bunds. After finalization of cross section of the bund/embankment i.e. top width and side slopes the hypothetical hydraulic gradient was superimposed on it. If necessary, a back berm (pushta) was provided to keep the hydraulic gradient line within the bund section and to avoid its opening on the country side slope of the bund/embankment. Since the hydraulic gradient was hypothetical, bunds and embankments were constructed to a greater size than actually required resulting in increased project cost.

The embankment section must be stable under all flood and seismic scenarios to avoid embankment breaches. Stability of the cross section depends on the foundation conditions and strength of the fill material. It is highlighted in the document that detailed and systematic sub surface investigations for foundation evaluation may be provided by Departments. However, this recommendation is mostly not practiced in field.

Construction Practices and Procedures

The main construction practices adopted in Sindh, Punjab, KP and Balochistan provinces are as per the Sindh Bund Manual and Manual of Irrigation Practice Punjab.

It was mentioned that bund construction in Sindh would be 12.5% higher than the design crest height to account for future settlements if manual labour is used while 6.5% higher if mechanical compactors are used.The lack of compaction and moisture control has led to poor bund construction. It was also mentioned that specifications regarding spreading, moisture control and compaction were to be in line with modern specifications for construction of embankments.

According to NFPP-I, the construction specifications were fairly uniform throughout the country, but a need for adopting modern methods was specified. Also, for achieving better results for embankment/bund construction,the construction specifications and practices must be improved and enforced.

Maintenance of Bunds

The maintenance of flood bunds during non-flood periods has been outlined as a major problem. The flood bund maintenance aspects considered include bund inspection, identification and repair of burrow holes, weakness caused by rain cuts and development of cracks in soils. Poor maintenance and considerable deterioration of bunds resulted in breaching of bunds/embankments. A field program is provided in the document which consists of investigation procedure and reporting of field findings.

Improvement of flood protection in upper reaches was observed to create problems for lower reaches by increasing the flood peaks at downstream reaches. Channel confinement due to construction of bunds on both sides at lower reaches further aggravated the situation.

Under NFPP-I, existing conditionsurvey of thirteen (13) selected flood protection structures was carried out which included Lahore Protection Bund, Left Marginal Bund Khanki Headworks, Left T-Spur Rasul Barrage, Main River Bund Kahirpur, F.P. Bund Dadu, Manchar Lake Bund Sehwan and Ali Bahar Loop Bund Thatta. These observations were carried out at specified test location of the bunds/embankments and not to its entire lengths. The field observations made were reported in the NFPP-I document.

Recommendations

Recommendations were also made for the designing, construction and evaluation of flood protection bunds. Some major recommendations made in NFPP-I are as under:

i) In order to economize the construction and maintenance costs it was recommended to have different design criteria for protection works in urban, industrial, agricultural and rural areas keeping in view the level and extent of acceptable flood damages, instead of adopting samecriteria for each category.

ii) Frequency analysis was considered necessary for the safety evaluation of a bund with its design based on highest historical flood, especially when the period of record is short.

iii) The sub surface investigation for exploration of foundation soils is imperative to determine the extent, location and physical characteristics of various possible soil types for which the bund foundation has to be designed.

iv) It was recommended to improve the prevailing general practice, of using nearest available soil for construction of bunds/embankments without investigation of soil characteristics.

v) It was recommended that freeboard must be provided for safety against embankment overtopping keeping in view the higher design flood level, wave height, wave run up and expected soil settlement.

vi) The prediction of the profile of phreatic line across the bund/embankment should be carried out to ensure safety against piping failure of bund/embankment.

vii) Detailed slope stability analysis should be carried out using Method of Slices.

viii) The bund fills must be properly compacted for maximization of shear strength, imperviousness, prevention of shrinkage cracks, excessive settlements and rain gully formation alongside slopes.

ix) It was suggested that remedial measures shall be provided for poorly constructed bunds with an unsatisfactory level of fill compaction. A sloping filter was suggested to be introduced on landside slope of the bund in vulnerable reaches to protect against piping and internal erosion. A suitable thickness of cover material shall be provided on the sloping filter. The horizontal filter connected to the sloping filter will deliver the seepage water to the landside toe of the bund.

3.2 National Flood Protection Plan 1988 (NFPP-II)

National Flood Protection Plan of Pakistan Phase-II was a follow up to NFPP-I (1978). It comprised of a feasible flood protection program as per Government of Pakistan policies and objectives at the time. Efforts were made to develop a favorable, integrated and unified flood protection plan to achieve maximum benefits during planning and execution of flood control projects. The planning strategy for NFPP-II was mostly based on the observations and results achieved from execution of NFPP-I. Planning objectives were updated and revised criteria were formulated in light of the lessons learned from NFPP-I to further reduce the threat to loss of life, economy, emergency evacuation costs and impairment of national security. The second phase planning document of NFPP-II also included the flood management planning for Northern Areas, Federally Administered Tribal Areas (FATA) and Azad Jammu and Kashmir, which were not addressed in NFPP-I.

Expected flood hazard problem and corresponding protection requirements were also addressed in the NFPP-II planning document. The threat of flooding in plain areas of Sindh and Punjab was expected to arise from overbank flooding of main rivers caused by heavy monsoon storms in river catchments. As the valley slope in plain areas of Punjab is towards east, the flood water spillage along left side would not flow back to the river.Keeping in view the breaching sections in marginal bunds were provided on the right side of the river. In Sindh, flooding of Indus River was mentioned as a major threat to the province. It was mentioned that Indus River flows on a high ridge in the province and overbank flood spills do not return back to the river, which causeserious damages. Table3.1from NFPP-II (1988) given below providesthe length of embankments in miles, at the time, along different rivers of each province.

Table3.1: Lengths of Major Rivers and Embankments in theFour Provinces of Pakistan

Sr. No.	Province	River	River Reach (miles)	Embankment Length (miles)
1.	Punjab	Sutlej	349	221.3
		Ravi	406	277.4
		Chenab	405	694.0
		Jhelum	224	59.3
		Upper Indus	522	443.6
	Sub Total for Punjab			1,695.6
2.	Sindh	River Indus	519	1,457.0
3.	KP	Misc. Rivers	-	116.0
4.	Balochistan	Misc. Rivers	-	162.8
	Total			**3,431.4**

Freeboard

Improvement was made to the procedure for estimation of freeboard through introduction of development in scientific knowledge parameters such as wind velocity, wind direction, fetch, bund side slopes and roughness. It was pointed out that past experience of arbitrary fixing of freeboard resulted in over or under designed flood bunds.

The formulaegiven by Creger and Justin, Bretschelder, Moliter-Stevenson, Sverdrup Munk, Stevenson and Stephenson along withthe Tables presented by USBR were recommended for wave height calculation. Hunts formula and USBR Approach were recommended for the computation of wave ride. Freeboard was calculated for Risalpur, Jehlum, Lahore and Sargodha for 5, 25, 50 and 100 year return period floods using these formulae. USBR formula was recommended for freeboard estimation as it was observed to provide comparatively more realistic results as compared to other formulae.

The values recommended in NFPP-II, for wave height determination from wind velocities and reservoir effective fetch are given in Table 3.2 below:

Table 3.2: Effective Fetch, Wind Velocity and Corresponding Wave Height Recommended by USBR

Effective Fetch (miles)	Wind Velocity (miles/hr)	Wave Height (feet)
1	50	2.7
1	50	3.0
2.5	50	3.2
2.5	75	3.6
2.5	100	3.9
5	50	3.7
5	75	4.3
5	100	4.8
10	50	4.5
10	75	5.4
10	100	6.1
Reference:*Design of Small Dams, USBR*		

Incase wave height is taken from Table 3.2, the wave ride is taken as 1.5 times the wave height.

$$\text{Wave Ride (R)} = 1.5 \times \text{Wave Height } (H_W)$$

Recommendations

Following major recommendations were made for flood protection works in NFPP-II:

i) A Master Planning Study shall be undertaken for major rivers (Sutlej, Ravi, Chenab, Jhelum, Upper and Lower Indus), KP, Balochistan, FATA, Northern Areas and Azad Jammu and Kashmir to comprehensively identify the flood problems in each area and river.

ii) Eight schemes on Ravi River shall be executed on high priority basis considering the high damage potential.

iii) Extension of Haripur bund up to Sohdhra village shall be included in 8thFive Year Plan (FYP) period.

iv) For Upper Indus River Basin, feasibility studies for river training works upstream of Ghazi Ghat Bridge, flood protection bunds in Derajat Circle and improvement of river approach upstream of Taunsa Barrage shall be carried out.

v) For Lower Indus, the construction of Ghorabari Bund at a cost of 3.51 million rupees shall be computed on high priority basis.

vi) In KP, flood protection schemes for Warsak Canal System, Land downstream of Terbela (from flood action of Indus), Kabul River Canal System and Kalpani Nullah etc. shall be executed.

vii) Feasibility studies for Flood Management of Hill Torrents in FATA shallbe taken up during 6thFive Plan Period and completed during first year of 7th Five Plan Period.

viii) InAzad Jammu and Kashmir, flood protection schemes of Suketar, Bhimber, Randian, Bhunder and other nullahs shall be executed on immediate basis.

ix) Flood protection in Azad Jammu and Kashmir for villages and agricultural lands of Bela Muhammad Khan Patika, Kohri Sheikh Bola, Tandali, Prak, Subri, Badiara, Bagh Town, Ajra, Harighel and Dhulli villages shall be executed.

x) In Northern Areas, flood management scheme for villages in Astore, Kharmog and Khaplu Valley, protection works and erosion control along KKH, and flood protection of Hydel Power Station in Gilgit area shall be completed by 7th Five Year Plan Period.

3.3 Bund Manual, Sindh (2008)

The Bund Manual, Sindh was developed as a guidelinefor engineers involved in design, construction and maintenance of flood bunds and embankments. The manual includes criteria and recommendations for design of new bunds, stone pitching, spurs, bund sluices and other related structures. It also includes plans for flood fighting and proceedings for maintenance and repair of existing bunds.

Bund Design Criteria

A freeboard of 4.0 ft above designed HFL has been considered sufficient to accommodate unexpected rise of water level or height of any generated waves. The manual states that for a fetch of 1.0 mile, the height of generated wave will be 3.0 ft. The wave heights can be calculated using the available standard formulae (such as Stevenson’s formula).

It wasmentioned that hydraulic gradient from the designed HFL on country side slope should cut the base well within the downstream toe and have sufficient cover of earth over it. Hydraulic gradient of 1V: 6H was specified to be used for the designing of bunds.

The riverside and landside slopes were also discussed and appropriate values were suggested. It was mentioned that the embankment/bund side slopes were dependent upon behavior of its constituent material in saturated state. The manual also discussed the leaks/ free passage of water, piping action along with stability of the section. Key design parameters are reported in Table 3.3.

Tree plantation was not allowed within specified distances of the toe, particularly on the upstream side, as roots tend to loosen the structure when shaken by wind storms and encourage creep which leads to leaks.

The importance of fill material was highlighted to avoid differential settlements. Soils containing organic matter were recommended to be rejected. The cohesive materials with excessive clay content shrink and crack on drying and slide when saturated and hence were not recommended for use in bund construction.

It was recommended that sandy soilshaving 30% to 40% clay content may be used for construction of bunds and embankments.

Construction of Bunds

Relative merits of different soils for bund construction were provided in the manual. Sand, sand mixed with clay, loam and kalar were some recommended materials available in Sindh for this purpose. A method for bund construction was also provided in the manual. The construction method for new bunds or loopsstates that no organic matter should be allowed in bund to prevent development of cavities upon decay and the seepage line along base of the bund. The removal of trees and grubbing roots from the foundation area must be done along with the filling of cracks and cavities in the proposed embankment/bund seat. If any ridges or mounds are present, steps are worked out to provide a better bond between new and old construction. The site preparation should be done for additional five hundred (500) ft. material must be provided in six (6) inch layers for full width of embankment. Sandy material should be placed inside and clayey material at the outer side of bund. Proper compaction must be done carefully. Each layer must be consolidated thoroughly using rammers and rollers. Sufficient number of works must be assigned to the job for clod breaking, ramming, rolling, etc. It was mentioned that the earthwork may be stopped if the consolidation is not being achieved to the satisfaction of Executive Engineer.

It was also mentioned that the top of newly constructed bunds must be made 12.5% higher than the design height to provide allowance for subsequent settlement after completion of bund construction. Stone pitching for bunds was recommended to be provided against heavy wave wash. Specifications and construction procedure for stone pitching were laid down in the manual.

3.4 Manual of Irrigation Practice, Punjab

The manual briefs basic parameters for design of embankment section such as freeboard, top width, landside and riverside slopes, stone pitching, etc. but are not provided in detail.A freeboard of 5.0 ft was recommended to be used but no criteria or approach was discussed. The slope of hydraulic gradient was not defined in the Manual of Irrigation Practice Punjab as it was defined in Sindh Bund Manual or NFPP and FPSP documents.

The slope stability of embankment section was not discussed nor was any method of analysis specified (e.g. method of slices). It was mentioned that the top of embankment should be slightly sloped towards the landside to prevent chances of riverside slope guttering during rainfall.Details for the stone pitching of river side slopes due to severe wave wash were not provided.

The construction practices and procedures for bunds and embankments werediscussed in the manual. The manual does not contain details regarding the sub surface investigations which areconsidered essential for embankment construction. The maintenance of existing flood bunds and embankments were discussed but not in detail.

The Manual of Irrigation Practice, Punjab is an old document which has not been updated since 1963 (the year of latest version needs confirmation). A number of areas regarding design and construction of bunds and embankments need to be updated. It is necessary that the design parameters and construction procedures provided in the manual may be updated in accordance with current scientific knowledge and computer applications.

3.5 Flood Protection Sector Project-I (1989)

The Flood Protection Sector Project-I was carried out with the assistance of Asian Development Bank (ADB) with an objective to reduce urban and rural flood damages and human sufferings through construction of 18 high priority sub-projects. The project aimed at the following:

- Preparation of feasibility studies for construction of various structures along rivers and streams in four provinces of Pakistan
- Improvement of Pakistan’s Flood Forecasting and Warning capability
- Identification and pre-feasibilities for management of five most critical structures on major rivers against severe floods.

Bund Design Criteria

A standard design for bunds and embankments was provided in the project report. Top and base width of bunds, freeboard and side slopes for riverside and landside were addressed in the design criteria. The details are tabulated in table 3.3 also provide a comparison with other design documents and reports.

The existing practice of hydraulic gradient assumed to a slope of 1V: 6H for checking the base width of the structure was to be continued. It was mentioned that if structure width did not accommodate the hydraulic gradient, either outer slope should be flattened or a pushta should be provided.

Stone apron has to be designed considering scour depths. A typical stone apron design for Chenab River was provided in the document. It was also mentioned that stone pitching size should vary between 80 to 120 lbs (36.28 to 54.43 kg).

For Balochistanand KP provinces, the structures consisted of wire crated gabion, retaining walls, guide bunds and spurs. It was established in the design criteria that each of these structures will be designed for safety according to the conditions of its location. A typical design was provided for Camp Koroona retaining wall at Kabul River as a reference.

Hydraulic Model Studies

In FPSP-I, hydraulic model studies were carried out for all major river reachesof Punjab, Sindh and KPto check the erosion problems. For the construction of 60 structures, fifty two projects in Punjab, Seven in KP and one in Sindh were undertaken. In some cases already carried out model studies by PID’s were updated.

According to FPSP-I, model studies were carried out under certain limitations such as:

- There was large distortion between horizontal and vertical scales
- Modeling of exact morphological conditions as at site, required to achieve an exact Froude number,was not possible

- The model running time was only for a few hours but the actual situation required a time span of years
- The required mathematical formulaewere not used to account for mentioned discrepancies

Construction of Works

The report included construction works for bunds and embankments carried out by the Contractors under supervision of PIDs and review Consultants. The construction of works as per schedule was reported as a major problem for FPSP.

The construction material for bunds included earth, stone, wire crates, filter material etc. It was reported that Consultants ensured that soils used in construction were of required gradation and free of organic matter and harmful salts. Also, the borrow areas for obtaining the material were reported to have a history of use.It was ensured that stone was as per specifications laid down in the tender documents.

The performance of various bunds and spurs in Punjab, Sindh, KP and Balochistan were provided in the report. The Consultants visited different schemes to observe the firsthand performance of these bunds and other flood protection works. These observations were included in the project completion report. Certain sites could not be visited for which it was assumed that all flood protection works at these sites are operational and workingsatisfactorily.

3.6 Flood Protection Sector Project - II (2001)

The Flood Protection Sector Project-II was carried out with the assistance of Asian Development Bank (ADB) and consisted of four packages. It aimed at reducing the flood damages on major rivers and hill torrents in the country through construction of 217 training structures and 453 kilometers long flood protection bunds and levees benefiting an approximate area of 8 million acres. These flood protection works included all types of structures such as spurs, flood bunds, studs and pitched embankments. The Package-B report titled “Design Criteria and Methodology” (October, 2001) contains recommended criteria for flood embankments. The design criteria provided in this report were reviewed and are briefed in following paragraphs.

Bund Design Criteria

The design criteria for flood bunds and river training works are discussed in detail in the report. Different types of spurs or groynes including Bar, Mole Head, Hockey, T-head, Guide-head, J-head and Sloping spurs were discussed. Criteria were provided for the alignment, spacing, length, side slope (shank and head), top width etc.for spurs.Other river training works such as studs, guide banks and gabion retaining walls have also been addressed.

Freeboard

The provided design criteria for determination of freeboard takes into accountthe wave run up (R),wind setup (S, above still pond level) and river set or super elevation. Hunts formula, also mentioned in NFPP-II, has been used for calculation of wave run up, where as Zuider Zee formula is used for determination of wind setup. The total rise of water level due to combined effect of wind setup and wave run up is obtained as follows,

$$\text{Rise in Water Level} = \mathrm{S} + \frac{2}{3}\,\mathrm{R}$$

The super elevation formed at the concave or outer side curve of river banks has to be considered also when fixing freeboard for outer banks. Schoklitsch formula has been recommended for calculating river set or super elevation.

Minimum freeboard for various flood protection structures were recommended in the report which included 1.0 ft additional freeboard for contingent requirement as factor of safety. The embankment settlements for foundation and fill are to be neglected against this 1.0 ft contingent provision. The recommended minimum freeboards for typical protection structures were influenced by different hydraulic conditions with design wind velocity over land assumed as 50 miles/hr (80 km/hr).

Stability Analysis of Embankment Slopes

The slope stability of embankments was discussed in detail in the report. It was emphasized that the embankment slopes must be stable under all conditions of construction, design flood discharge, rapid flood drawdown, low flow level and earthquake forces.

The stability analysis was recommended to be carried out according to Method of Slices, using Bishop’s Method with a computer package. The required information for performing the analysis will be embankment geometric data, soil properties, design flood level, low water level of river, phreatic line, porewater pressures, surcharge on embankment and earthquake loads.

The importance of phreatic line was also emphasized in the report and its relation to stability analysis was explained. The flow nets were to be constructed manually or using computer software.

A surcharge loading of 200 lb/ft^2 (975 kg/m^2) for stability analysis was recommended to be applied at embankment top.Specific earthquake zone criterion in relation to the location of embankment under analysis will be used for adopting OBE (Operating Base Earthquake) values.

Minimum safety factors were recommended against shear failure for both with and without earthquake conditions.

Hydraulic Gradient

The hydraulic grade line was recommended to be assumed as 1V:6H with a minimum cover of 2.0 ft provided over it.It was mentioned that the practice for providing pushta on landside does not solve the problem of seepage through embankment on impervious foundation. It was recommended to be advisable only where duration of steady flood level against the embankment is not more than the period of saturation required for the section with respect to permeability of the fill material.

Foundation Stability

The stability of embankment/bund foundation was addressed in the design criteria. The embankment/bund slopesare dependent upon strength of fill materials and foundation characteristics. Hence for proper side slopes, strong and stable foundation is necessary. Detailed sub surface investigations were advised to be carried out for proper foundation evaluation. Necessary foundation treatments were recommended for strengthening and protection as per site and foundation conditions. The recommended site investigations included bore holes, test pits, trenches and laboratory testing.

Hydraulic Model Studies

Two types of physical modelstudieswere recommended to be carried out for investigation of various aspects regarding the overall performance of proposed protection works.First was a comprehensive modelrecommended for refinement of proposed river training works layout and design. The second was a distorted movable river bed model recommended to study the optimization of configuration, scour protection, flow velocity and other details of the river training works.

A check list was provided for checking main hydraulic design components for model studies.The scale relation of physical models based on Froude’s law was tabulated in the report.

Scour

The determination of scour using different empirical formulaesuch as regime approach methods (Lacey and Inglis), USBR methods (Lacey’s, Blench, Neill, Veronese, Jain, Schoklitsch and Zimmerman and Maniak) along with other developed methods (Molesworth and Yeindunia, Farraday and Charlton, Browns formula etc.) was given in the report.A table was provided for use of recommended methods for various combinations of river gradient and bed material type.

A comparison of bund design criteria between NFPP-I, NFPP-II, FPSP-I, FPSP-II, Manual of Irrigation Practice Punjab and Sindh Bund Manual is given in Table 3.3.

4. NEED FOR NEW DESIGN CRITERIA

The review of existing flood protection plans, manuals and reports provided an insight to the past studies and criteria available for bund design and construction. It was observed that the existing national flood protection plans (NFPP-I and NFPP-II) of 1978 and 1988 did not include the computer aided engineering design software’s such as Slope-W and Seep-W for the stability and seepage analysis of new and existing embankments.

The reviewed documents were formulated prior to 2010 flood which is considered to be of exceptionally high magnitude for irrigation and hydraulic structures. Due to this extreme flood event, numerous breaches were observed at various embankments. Loss of life and considerable damages to property and infrastructure took place. The existing bunds at several locations were found deficient, either in design or due to poor construction, to sustain such severe floods.

It is considered necessary that the existing criteria and procedures may be updated to ensure better performance of newly constructed and existing flood protection works. The physical model studies for river training and flood protection works are also important and must be included in the design of flood bunds and embankments.

5. DESIGN CRITERIA FOR FLOOD BUNDS/EMBANKMENTS AND RIVER TRAINING WORKS

5.1 Hydrological Design Criteria

Flood management requires prediction of flood magnitude for introducing the use of structures to change the physical characteristics of floods. These structures include (purpose-built) flood protection works such as embankments, spurs, pitched islands, guide banks etc. The records of hydrologic events, stream gauge data, rainfall data etc. form the basis for calculating the flood magnitudes for selected return periods. Stream gauges are generally installed at structures along the rivers i.e. at barrages, bridges, weirs etc. to record the water levels and consequently the discharges. However, on channels where such record is not available, precipitation and catchment data will be considered for calculation of discharges of various return periods.

Table3.3: Comparison of Bund Design Criteria

Document	Year	Height of Bund	Freeboard (above HFL)	TopWidth	Side Slope (V:H)		Hydraulic Gradient	BackBerm(Pushta)	StonePitching
					Riverside	Landside			
NFPP-I	1978	HFL+FB	6.0 ft	20-25 ft	1:3	1:2	1:6 (assumed)	Min. 2 ft cover above HGL	For Severe wave wash damage
NFPP-II	1988	HFL+FB	USBR Approach	N.A	1:3	1:2	1:6 (assumed)	Min. 2 ft cover above HGL	For Severe wave wash damage
FPSP-I	1989	HFL+FB	6 ft (Indus/Chenab) 5 ft (Ravi)	25 ft (Ravi/ Chenab) 30ft (Indus)	1:3	1:2	1:6 (assumed)	Min. 2 ft cover above HGL	For Severe wave wash damage
FPSP-II	2001	HFL+FB	6.0 ft	25 ft	1:3	1:2	1:6 (assumed)	Min. 2 ft cover above HGL	For Severe wave wash damage
Manual of Irrigation Practice, Punjab		HFL+FB	5.0 ft	16 ft	1:3	1:2	N.A	Min. 2 ft cover above HGL	For Severe wave wash damage
Bund Manual, Sindh	2008	HFL+FB	4.0 ft	20 ft	1:3	1:2	1:6 (assumed)	Min. 2 ft cover above HGL	For Severe wave wash damage

NFPP – National Flood Protection Plan
HFL – Highest Flood Level
HGL – Hydraulic Grade Line
FPSP – Flood Protection Sector Project
FB – Freeboard
N.A – Not Available

Hydrological studies related to the design and evaluation of new and existing flood protection and river training works will be carried out by analyzing the available stream gauge data/discharges influencing individual sites.

5.1.1 Data Collection

5.1.1.1 Available Data

Availability of relevant, adequate and accurate data greatly facilitates the realistic estimation of flood magnitude. The period of available flow data is available for a minimum period of fifty (50) years along the major rivers. The available data collected from concerned agencies will be reviewed and results will be derived for fixing of the design flood parameters.

5.1.1.2 Additional Data

Following additional geometric field data will be required for routing of flood wave through the channel under consideration:

- Channel cross section
- Alignment of the channel
- Expansion and contraction of the channel due to natural or manmade features
- Details of existing structures, etc.

5.1.2 Data Analysis

5.1.2.1 Flood Frequency

The primary objective of the flood frequency analysis is to determine the return periods of recorded stream flows of known magnitude and then to estimate the magnitude of stream flows for selected return periods within and beyond the recorded period. For flood frequency analysis, annual maximum peak discharges for particular site will be arranged in descending order and curve-fitting exercise will be performed using the following Weibull's formulae [7]:

$$\mathrm{P} = \frac{\mathrm{m}}{\mathrm{n}+1}$$

$$\mathrm{T} = \frac{\mathrm{n}+1}{\mathrm{m}}$$

Where,

P	=	probability of the event
T	=	return period
n	=	rank of a value in a list arranged in descending order
m	=	total number of values to be plotted

Return periods or probability of occurrence of an event will be determined from the fitted line. The results drawn for different return periods will be tabulated for further analyses to decide the magnitude of design flood, from the developed frequency curve. Return period of the design flood will be obtained for an individual subproject/scheme, by considering the following:

- Flood damage potential of the river reach
- Land use pattern of the flood prone areas
- Degree of severity of human sufferings caused by floods of various intensities
- Safe capacity/integrity of existing structures across the river reaches
- Design flood adopted for adjacent river reach

Reports on flood prediction at barrages/headworks of Punjab and Second Flood Protection Sector Project (FPSP-II) are available, covering flood data up to the year 2004 [8],[9]. The results obtained from data analysis presented in all the previous reports will be reviewed and would be updated by using additional available data or extrapolated beyond the periods considered.

Estimation of design flood will be carried out for a return period of hundred (100) years for existing barrages/headworks for the improved design, where required. A return period of fifty (50) years will be adopted for flood bunds, spurs and banks protection design. However, the actual recorded peak flood discharge will be used for design if it exceeds the discharge calculated for the above return periods.

5.1.2.2 Flood Routing

Flood routing analysis for different return periods under consideration will be performed through the river reaches to obtain water surface profiles and inundation extents along river reaches. In a river reach, the water surface profile corresponding to the adopted “Design Flood” against which protection is desired gives the design flood level for the structure. HEC-RAS Version 4.1 “River Analysis System” computer software by USBR will be used for obtaining one-dimensional “Steady Flow” water surface profile. This computer model computes single or multiple profiles for both sub-critical and supercritical flow regimes. Friction lossescomputed using Manning‘s equation and expansion and contraction loss are accounted for in the model. The following capabilities are also part of the model;

- Effects of bridges, culverts, inline weirs, embankments, and gated spillways on water surface elevations can be calculated
- Effects of channel modifications and encroachments on water surface elevations can be calculated
- Manning’s roughness coefficient ‘n’ from known flows and water surface profiles can be incorporated in the model
- The dimensions of a floodway, i.e. main channel and flood plain that should be kept free of obstructions to prevent development of excessive velocities and increased flooding. The model can incorporate manmade and natural obstructions

5.1.2.3 Rainfall Runoff

In the absence of stream gauge data, flood estimates for catchments will be derived by means of HEC-HMS “Rainfall-Runoff Model”, a computer package developed by Hydraulic Engineering Centre (HEC) of US Army Corps of Engineers. This computer model uses precipitation data and catchment loss formulae to compute the rainfall excess for particular storm duration. The rainfall excess is distributed in time by arranging rainfall increments in a suitable storm form. Storm rainfall is then represented by a unit hydrograph and its shape is dependent on the time of concentration for the catchments. Base flows are added to the storm runoff hydrograph and flows may be routed though a system of channels and storages, if required.

5.2 Flood Bunds and Embankments

5.2.1 Freeboard

Freeboard is the vertical distance between maximum expected water level and the top level of the embankment/bund. The top level of bund should be so fixed that there is no danger of overtopping, even with the intense wave wash or any other unexpected rise in water level due to sudden change in river course, river bed aggradationsor embankment settlement. In order to estimate the embankment or river training works freeboard the wave height, wave run-up and wind setup must be calculated.

5.2.1.1 Fetch

According to USBR Design of Small Dams fetch is the distance over which the wind can act on a body of water. It can generally be defined as the normal distance from the windward shore to the structure that is being designed or evaluated. However, the “effective” fetch may have a slightly curved path, for a case when the wind sweeps down a winding river valley between land ridges [10].

The fetch plays an important role in estimation of embankment freeboard since the generated waves depend on it.

5.2.1.2 Wave Height

The waves are generated on the surface of water by blowing winds. The height of a water wave mainly depends on the wind velocity (V_W) and effective fetch or straight length of water stretch(F). The Wave height (H_W) may be calculated using various empirical formulae.Some well known formulaefor wave height determination are given below [11]:

a) Creger and Justin Formula

If wind velocity (V_W, in miles/hr) and fetch (F, miles) are known, the wave height (H_W, in feet) can be calculated using the Creger and Justin Formula given below:

$$H_W = \frac{F^{0.37} {V_W}^{0.48}}{3.41}$$

b) Bretschneider Formula

In deep waters, where the depth is more than half of fetch length, the significant wave height “H_S” (in feet)can be calculated as follows,

$$H_S = 0.0555 \sqrt{{V_W}^2 \, F}$$

In shallow waters, where the depth is less than half of fetch length, the wave height (in feet) can be calculated as follows,

$$H_W = 0.65 \, H_S$$

Where,

F	=	fetch (miles)
V_W	=	wind velocity (knots)
		[1 knot = 1.687809ft/sec; and, 1 ft/sec = 0.59248 knots]

c) Moliter-Stevenson Formula

The Moliter-Stevenson formula for wave height is as follows:

$$H_W = 0.17 \, (V_W \, . \, F)^{0.5} \quad (\text{for } F > 20 \; miles)$$

$$H_W = 0.17 \, (V_W \, . \, F)^{0.5} + 2.5 - F^{0.25} \quad (\text{for } F \leq 20 \text{ miles})$$

Where,

H_W	=	wave height (feet)
F	=	fetch (miles)

V_W = wind velocity (miles per hour)

d) Sverdrup Munk Formula

The formula calculates wave height in meters i.e. SI units. The wind velocity "V_W" is used in m/sec. Acceleration due to gravity 'g' is taken as 9.81 m/sec^2.

$$H_W = \frac{0.26\, V_W}{g}$$

e) Stevenson Formula

Wave height in meters can also be calculated using Stevenson's formula.

$$H_W = \frac{1}{3}\sqrt{F}$$

Where,

F = fetch (kilometers)

f) Stephenson Formula

The wave freeboard can be computed using Stephenson's formula provide below:

$$F_W = 0.0206\, F^{0.5} - 0.117F^{0.25} + 2.5$$

Where,

F = fetch (feet)
F_W = wave freeboard (ft)

5.2.1.3 Wave Run-Up

The wave run-up is also known as wave ride is the distance a wave travels on the sloping face of an embankment or bund. In shallow waters, the wave ride can be measured vertically above the mean water surface. Hunts formula can be used to determine the wave ride [11].

Hunts Formula

$$\frac{R}{H_W} = \frac{K\, \tan\alpha}{\left(\frac{8}{T}\right)\left(\frac{H_W}{2g}\right)^{1/2}}$$

Where,

R = wave run up or wave ride (ft)
H_W = wave height (ft)
K = surface roughness coefficient of embankment slope(ft)
2.3 for rough surface; 1.8 for earthen surface
T = wave time period (sec)
= 0.5 (V_W^2. F) $^{0.25}$
F = fetch (miles)
V_W = wind velocity (knots)
α = embankment slope angle with horizontal (degrees)
g = acceleration due to gravity (ft/sec^2)

5.2.1.4 Wind Setup

The rise of water level above undisturbed pool water surface due to the movement of wind on the water surface, particularly in shallow waters is known as wind or wave setup. It can be calculated using Zuider Zee formula [11].

Zuider Zee Formula

$$S = \frac{{V_W}^2 F}{1400 D} \cos \theta$$

Where,

S	=	wind or wave setup above pool level (ft)
D	=	average depth of water (ft)
θ	=	angle of incidence of waves (degrees)
V_W	=	wind velocity (mile/hr)
F	=	fetch (miles)

The combined effect of wave run-up and wind setup to estimate the freeboard can be is equal to the wind setup plus two thirds of wave run-up.

5.2.1.5 USBR Approach

US Bureau of Reclamation has also specified an approach for determination of wave height and wave ride. The Table 5.1, taken from the Design of Small Dams, can be used to determine the wave height based on fetch and wind velocity data [10].

Table 5.1: Wave Height versus Fetch and Wind Velocity (by USBR)

Fetch (miles)	Wind Velocity (miles/hr)	Wave Height (feet)	Wave Ride (feet)
1	50	2.7	4.05
1	75	3.0	4.50
2.5	50	3.2	4.80
2.5	75	3.6	5.40
2.5	100	3.9	5.85
5	50	3.7	5.55
5	75	4.3	6.45
5	100	4.8	7.20
10	50	4.5	6.75
10	75	5.4	8.10
10	100	6.1	9.15

In USBR approach, wave ride is taken as 1.5 times the wave height:

$$\text{Wave Ride (R)} = 1.5 \times \text{Wave Height } (H_W)$$

When fetch, wind velocity and wave height data is available, the empirical relations provided in sections above can be effectively used to calculate wave height, wind setup and wave run-up. The estimated freeboard can be calculated by adding wave run-up and wind setup accordingly.These formulae provide a minimum height for freeboard estimation.

Addition of estimated freeboard to the designed high flood level gives the embankment top level. Freeboard must be carefully provided since freeboard provision beyond its design requirement will result in unnecessary raising of embankment height which ultimately increasesthe cost of construction.

In the absence of wind velocity, fetch and wave height data, an appropriate value for freeboard must be adopted that satisfies the design criteria preventing embankment overtopping. The embankment freeboard in no case should be less than 6.0 ft above design high flood level.

5.2.2 Top Width

The designed embankment top width must provide motorable path for inspection of bunds and at the same time facilitate the movement of maintenance machinery and material loaded vehicles.during flood season and otherwise. If inadequate slope protection is provided with a lesser top width, chances of material erosion from side slope are increased during flood season that result in reduction of top width.These issues must be considered when deciding top width of flood bunds.

A minimumtop width of 25.0 ft must be provided for all flood bunds and embankments.The top of embankment must be given a slight slope towards the landside to diminish tendency of gutter or groove formation atwater side slope during rainfall.

5.2.3 Side Slopes

For an embankment section, the riverside and landside slopes are explained below:

a) Riverside Slope

Riverside slope is the embankment side slope directlyin contact with river flow. This is also referred to as front or upstream slope in Sindh Bund Manual. The side slope depends on the behavior of embankment material when saturated. The riverside slope must generally be flatter than the underwater angle of repose of the embankment material, which is flatter than the angle of repose for same material in air. Generally, 1V:3H riverside slope for flood embankments is adopted in all provinces of the country. For the design of all new flood bunds and embankments, the same riverside slope 1V:3H is recommended.

b) Landside Slope

The landside slope must be adopted as 1V:2H from bund crest level to the embankment toe on landside. It depends upon the phreatic or hydraulic grade line in the embankment section. The slope provided must be such that there is at least 2.0 ft earth cover over the hydraulic gradient line.

A back berm, or pushta, can also be provided on the land side without further flattening of the landside slope to ensure required earth cover over the hydraulic gradient line. The back berm also facilitates periodic inspection of bunds. It breaks the side slope on landside and shortens the path of rainfall flows which can lead to erosion of slope surface during heavy rains.

5.2.4 Base Width

The embankment base width can be determined after top width, height and side slopes of the section have been worked out. The base width of the section must be sufficient enough to support the embankment superstructure and prevent creep. Creep can be defined as the deformation under a constant effective stress. It occurs as long as the effective stress exists on soil beneath the embankment/bund foundation.

The embankment base width can mathematically be calculated as follows,

$$\text{Base width (B)} = (Z_{\text{riverside}} .H) + W + (Z_{\text{landside}} .H)$$

Where,

H	=	height of embankment section
$Z_{riverside}$	=	riverside slope of embankment section
$Z_{landside}$	=	landside slope of embankment section
W	=	top width
B	=	base width

5.2.5 Hydraulic Gradient

It is essential toknow the approximated hydraulic gradient line (HGL) in an embankment section. It is also known as line of seepage or phreatic line. After establishment of flood embankment section with top width and side slopes, the section is checked for hydraulic gradient. The existing design criteria and reports adopt an assumed value of 1V:6H for hydraulic gradient starting at the riverside from the highest flood level.

The development of engineering softwares has solved the problem of performing rigorous calculations. Seep-W software can predict the position hydraulic gradient in an embankment based upon the input parameters. The generated gradient can be imported to Slope-W software for carrying out slope stability analysis considering various scenarios (such as normal flow conditions, flood flow, drawdown conditions, etc.) with corresponding phreatic lines. These computer programs must be used for the determination of hydraulic gradient line and stability of the designed section of the embankments/bunds.

The hydraulic gradient is closely related to the rate of seepage in an embankment which results in internal erosion and piping problems. These problems are a threat to embankment stability weakeningit from inside due to seeping away of fine materials from the embankment body. In order to ensure embankment stability, there must be an appropriate cover of earth or embankment fill material above the hydraulic gradient line.A back berm, or pushta, can be provided at the landside slope to allow a sufficient cover over the hydraulic gradient line. The existing practice is to provide 2.0 ft cover over the hydraulic gradient which is recommended to be continued.

Hydraulic gradient line should be determined on the basis of analysis of material properties of soils, which are to be used in the construction of embankments.The phreatic line surface is a boundary between saturated and dry portion of the embankment. Soil properties for these upper dry and lower saturated embankment portions are to be used in stability analysis of side slopes.Typical with and without a back berm or pushta embankments are shown in Figure 5.1 and 5.2 respectively.

5.2.6 Wetting Channels

Wetting channel is a small supplementary bund provided at the riverside for pre-flood soaking of protection bunds and embankments. It closes shrinkage cracks in cohesive soils and helps identification of leakage paths and burrow holes so that necessary repair and maintenance work can be planned and executed prior to commencement of flood season. Wetting channels are provided in both Sindh and Punjab to avoid embankment breaches and failures where high clay content materials are used for embankment construction. Also, these are provided where severe damages are expected if a breach occurs.

Wetting channels can be filled with water from nearby creeks or through pumping water from adjacent irrigation channels and tube wells. Wetting channels can be adopted as a remedial measure to identify and repair any poor construction practices and procedures.

When flood bunds and embankments are constructed using granular material that does not develop shrinkage cracks in general, wetting channels may not be provided. Also, care should be taken during planning and provision of wetting channels that the embankment material is not highly permeable as it will further worsen the situation.

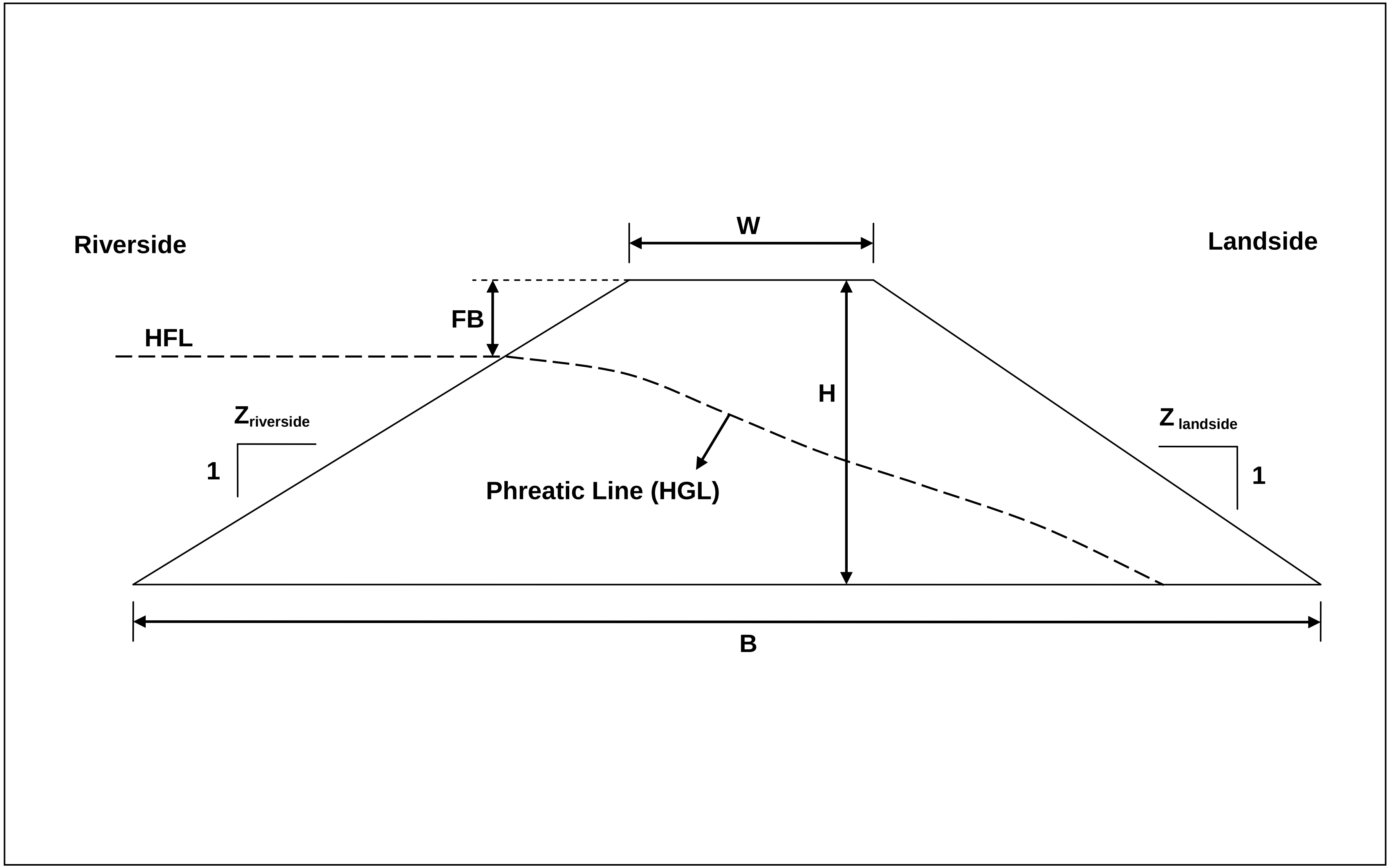

Figure 5.1: Sketch of a Typical Flood Protection Bund

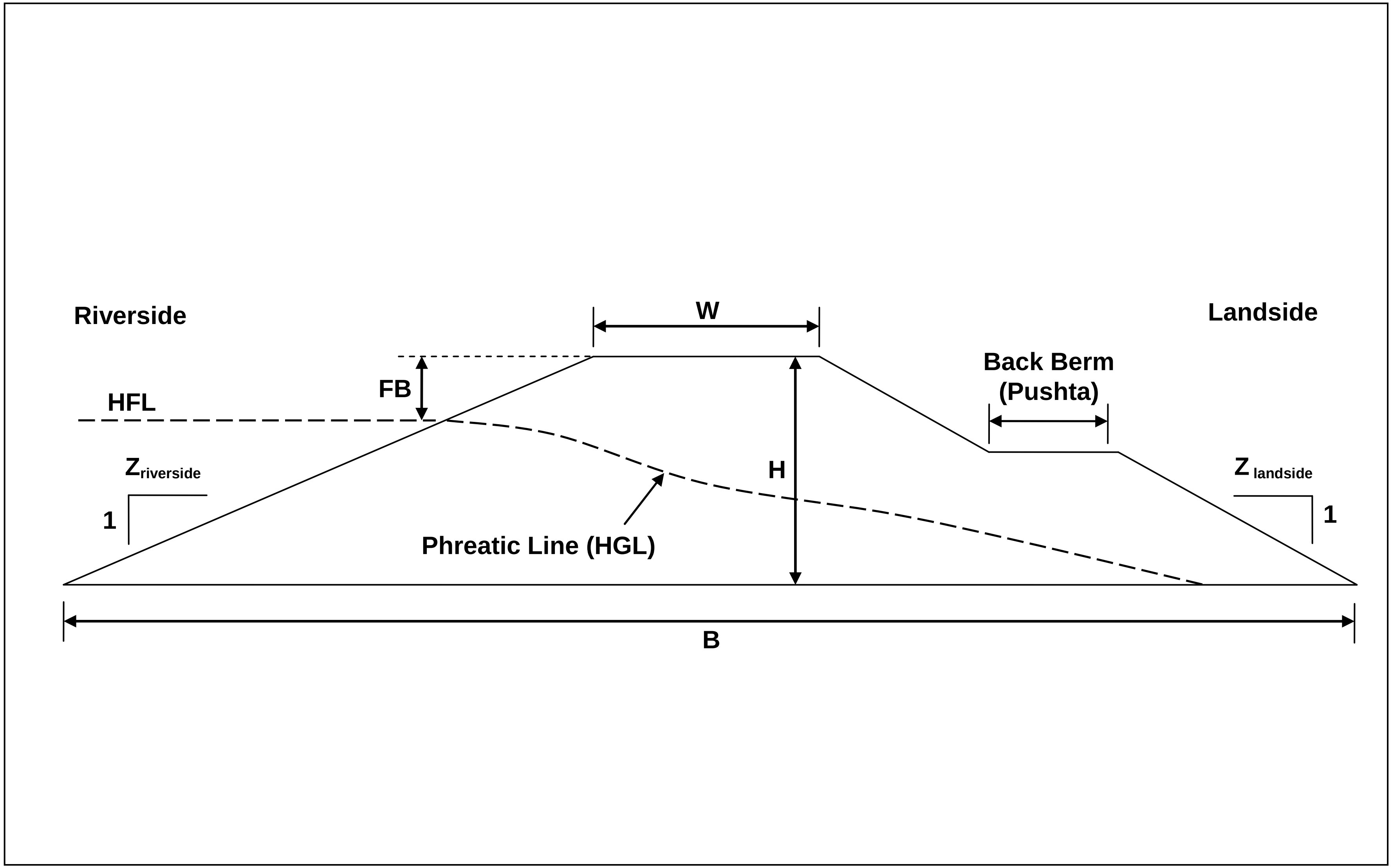

Figure 5.2: Sketch of a Typical Flood Bund with Back Berm (Pushta)

The existing practice of pre-flood soaking of some protection bunds and embankments using wetting channels in different regions of the country is recommended to be adopted as per necessity and embankment material requirement.

5.2.7 Wetting Trenches

These are shallow trenches, six to nine inch deep, excavated by manual labour along the riverside face of flood bund slightly above the high flood level. These trenches are filled with water from rivers throughbuckets. The purpose of these trenches is to soak the embankment for identification of cracks, burrow holes, leakages, or other problems that may pose a threat to the bund during high floods. After flood recession, the trenches are refilled by smoothening the riverside surface.

Wetting trenches are not considered very effective for soaking the embankment fill material where river flows do not touch the bund until high flood level is reached.

Since the provision of wetting trenches helps in identification of bund areas that need departmental attention for repair and maintenance before floods, the practice must be adopted and continued.

5.2.8 Sand Cores

Sand cores are sometimes provided in bunds and embankments when constructed of clay soils. Clay possesses cracking tendencies when it dries and swells when wet. The sand cores reduce the cracking potential of clay soils and block flow through cavities between clay clods. Sand being cohesionless collapses,closing any animal burrow holes developed in the embankment.

5.2.9 Clay Cover on Sandy Bunds

In Pakistan bunds constructed with sandy soils are covered with a six inch to one feet thick layer of clay. The cover protects the bund from action of wind and rains, particularly against wave wash damage on riverside. An additional cover of cobbles or gravel further enhances the life of clay cover. A vegetation cover instead of cobble or gravel cover can also be consideredwhichever is cheaper and easier to be provided.It is recommended that this practice may be continued where necessary.

5.3 Spurs/Groynes

Spurs/Groynes are structures, constructed in transverse direction to the river flow and extended from the bank into the river. These training works project into the river and are provided to keep the flow away from the erosion prone banks. The spurs/groynes consist of a shank and a nose, or head. The shank is a bund of adequate section which connects the spur head or nose to the highest point above the HFL at a river bank. The upstream face and nose of the spur are armoured with stone pitching and apron. The spur head or nose can have different angles and slopes to cater for local requirements. The spurs are provided with stone pitching launching apron to prevent scouring under water and avoid consequent failure of these structures.

In order to design new spurs or to evaluate existing ones, it is essential to understand the functions of these structures. In Pakistan, spurs/groynes are mainly provided for following purposes:

i) For river training along desired course to reduce concentration of flow at the point of attack preventing scour
ii) For protection of river bank under erosion attack or vulnerable to erosion in future by keeping the flow away from it

iii) For correcting/synchronizing the approach of incoming flows towards a barrage or any other structure
iv) For creating slack flow with the object of silting up in the area in vicinity of the river bank and spur

5.3.1 Design Discharge and HFL

The spurs must be designed either for a discharge equal to that of the structure which is in close proximity to the spur or equal to 100 year return period flood, or the highest recorded flood event, whichever is of a greater magnitude.

The high flood level used for design of spur top level and freeboard provision must correspond to the maximum design flood selected.

5.3.2 Alignment of Spurs

Spurs may be aligned either normal to flow direction or at angle pointing towards upstream or downstream of the flow. The training of rivers is done either by attracting, deflecting or repelling of flow. On the basis of alignment, spurs can be classified as follows [12], [13]:

5.3.2.1 <u>Attracting Spurs</u>

These spurs point towards downstream of flow and attract flow towards the river bank and are known as attracting spurs/groynes. The spur head or nose is constructed at an obtuse angle to the flow. The flow currents enfold at the side of spur where scour holes are formed close to the bank to maintain deep current there.

For attracting spurs, the attack of incoming flows is mainly towards the upstream face of the spur and requires more protection than at the downstream. The attracting spur is not constructed at the river bank which is to be protected but on the opposite bank to attract flow currents to the latter bank.

The selection of site for a spur is of utmost importance to achieve the desired purpose. The attracting spur can be constructed at 0.4 times the meander length upstream of the location to be protected. It is usually provided in meandering rivers [12].

A meander can be defined as two consecutive loops pointing in opposite transverse directions. The distance of one meander along the down-valley axis is the meander length or wavelength.

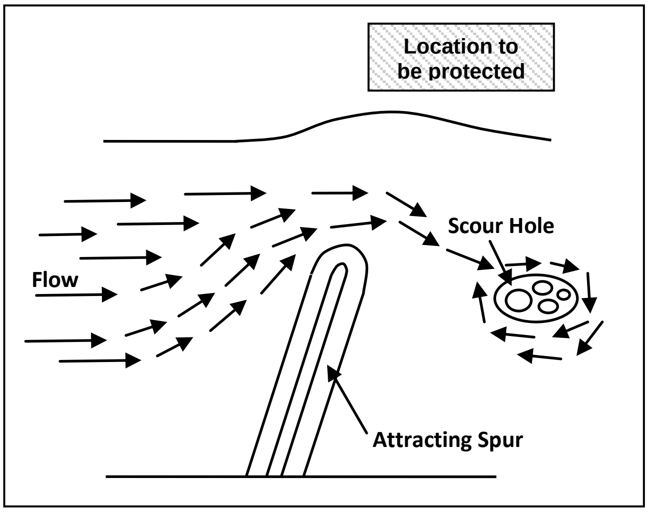

Figure 5.3a: Spur types based on alignment -Attracting Spur

5.3.2.2 Repelling Spurs

These spurs point in upstream direction of flow, repelling flow away from the river bank. A still water pocket is formed upstream of the spur and suspended load brought down by the river is deposited in it. This allows the flow currents to move away from the bank for considerable length.

Repelling spur is provided where the bank is desired to be protected and is so constructed that it subtends an angle varying from 60^0 to 75^0 to the flow direction. The spur head or nose is exposed to a greater river action and requires strong protection [13].

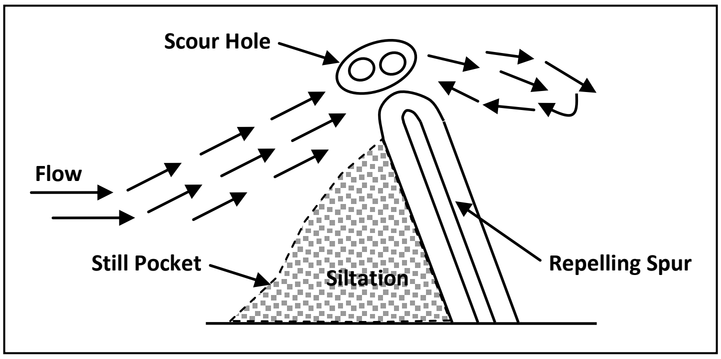

Figure 5.3b: Spur types based on alignment - Repelling Spur

5.3.2.3 Deflecting Spurs

Spurs of short length that only change or deflect the direction of flow without repelling flow away from river bank, are known as deflecting spurs/groynes. It is also known as a holding spur as it tends to hold the river channel at that location, scouring action at upstream and silting action along the bank at downstream side. These are generally provided for anti-erosion measures [13].

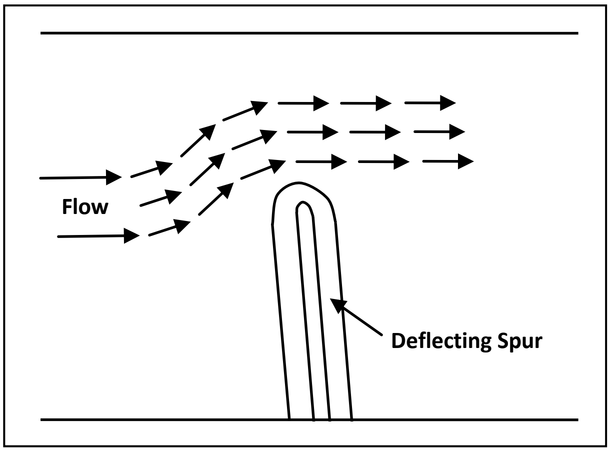

Figure 5.3c: Spur types based on alignment – Deflecting Spur

5.3.3 Length of Spurs

The spur length and spacing are sensitive design features that greatly influence the satisfactory performance of river training works. The length of spur/groyne is site specific and fixed as per site requirements.

The spur/groyne length must be determined on the basis of land availability at the river bank. Since the river banks experience scour and erosion due to action of incoming flows, the provided spur length in no case should be less than the length required to keep the scour hole formed at the nose of the spur away from the bank. Thus assuming angle of repose of sand to be 2.5H: 1V and anticipated maximum depth of scour below bed be 'd_s', the length should be more than 2.5 d_s [13].

$$\text{Length of Spur} = 2.5\ \mathrm{d_s}$$

Where,

d_s = anticipated maximum scour depth below river bed

If the length of spur provided is less than required spur length, it may lead to bank erosion at upstream and downstream of the spur due to formation of eddies at the nose. On the other hand, a very long spur/groyne may constrict the river flow path resulting in discharge concentration at the nose. Hence, it may not be able to withstand the attack due to heavy discharge concentration at the nose and may get damaged.

In case of a single channel, the effective length of spur must not increase $1/5^{th}$ of width of flow. In wide, shallow and braided rivers the extension of spurs/groynes in deep channels is not to exceed $1/5^{th}$ of the channel width on which the spur/groyne is proposed excluding the length over the bank [13].

Effective length of spur is the portion which is countered by the river flow. Extra spur length is provided only for the purpose of attaching the spur with high ground and is not to be considered as effective spur length.

Model studies must be carried out to determine the most appropriate spur length.

5.3.4 Number of Spurs

The spurs can either be used single, or in series. When the river reach to be protected is long, a single groyne/spur may not be enough. This will require the provision of a number of spurs to meet the requirement. Spurs can also be used in combination with other training works.

The number of spurs to be provided depends upon the particular location to be protected, river curvature, discharge intensities, sediment characteristics and control conditions.

5.3.5 Spacing of Spurs

The spacing of spurs/groynes depends upon river width, discharge, spur type, length, location and purpose. For wide rivers spur spacing is greater than that for narrow rivers with same discharge. For a straight reach, the spur spacing must be five times the projected spur length.

The spacing of spurs can vary with river bank curvature. A greater spacing is required for convex banks while a smaller spacing for concave banks. Generally, 2-2.5 times the effective length spacing is provided for convex banks whereas spacing equal to length of spur is provided for concave banks[13],[14].

When the river has a large curvature the spurs/groynes will have frequently varying lengths with varying spacing and changing angle of deflection. In order to ensure effective spacing of spurs/groynes for important and complex cases, model studies must be carried out [14].

5.3.6 Special Types of Spurs

Spurs/groynes are designed withspecial shapes each with respect to theirpurpose for river training. Each of these special spur types must be tested at physical model to ensure satisfactory performance for river training. Some special types of spurs are briefed below:

i) Hockey Spur

This spur type has a curved head like the shape of a hockey stick. It increases the flow attracting tendency of the spur and is not much helpful for bank protection [12].

In a hockey spur, the scour occurs equally at both upstream and downstream side of the spur. The scour hole formed due to river flows is shown in figure 5.4 below [13]. The stone protection is provided at the concave face and head, and for a short length at the convex face.

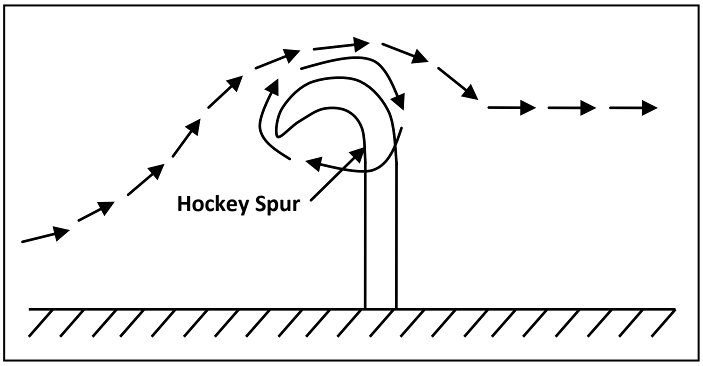

Figure 5.4: Hockey Spur

ii) Inverted Hockey Spur

This spur is a hockey spur only inverted in position. Here the convex face and head are first to make direct contact with incoming river flow. The stone protection is provided at convex face and head. Stone protection provided at the concave face is for a short length.

For these types of spurs, the flow velocities are reduced at the downstream side of the spur resulting in siltation. The silt deposition areas formed at spur downstream are shown in figure below [13].

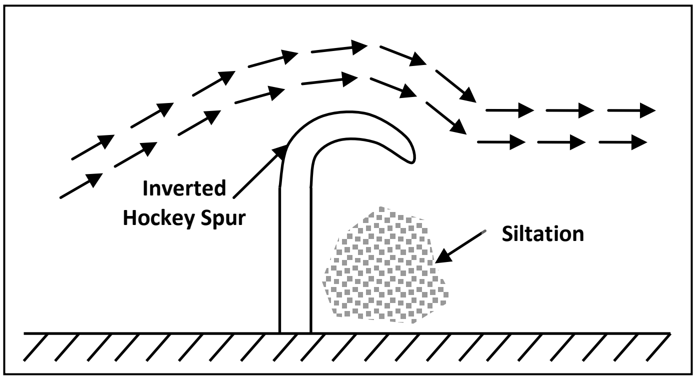

Figure 5.5: Inverted Hockey Spur

iii) T-Head Spur

The T-Head Spur was originally proposed by Denehy's and is also known as Denehy's Groynes. The shape of this spur type is like a 'T'.The spur head is generally placed parallel to the flow with 2/3rd length at the upstream side of the shank and 1/3rdlength to the shank downstream.

The objective for providing a T-head spur is to deposit silt at the concave bank from where it extends so that the erosion at concave banks can be reduced. The following figure shows a series of T-head spurs and silt deposition areas at the concave bank [13].

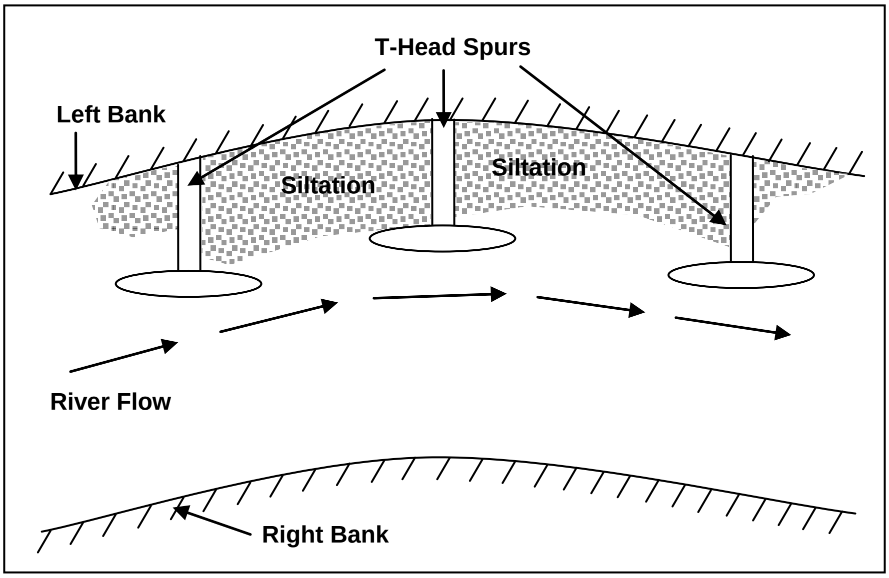

Figure 5.6: T-Head Spur

iv) Sloping Spur

The sloping spur is in the form of wedge shaped sloping ramp of solid stone. The height of ramp decreases from the bank top towards the riverside joining the stone apron provided at the river bed. Generally a ramp slope of 1V:6H to 1V:10H is provided. This type of spur can be provided for the training of a hilly stream.

v) J-Head Spur

A guide head spur is a combination of T-spur, hockey spur and sloping spur. It is also known as T-cum hockey-sloping spur. Basic design elements for a guide head spur are given below:

Hockey part: optimum radius of the spur head at upstream

T-part: alignment and length of spur head

Sloping part: optimum slope for downstream end of the spur head

Stone apron: effective dimensions and stone size on the performance of the spur head and upstream face of the shank

vi) Bar Spur

The bar head spur is the simplest spur type having a straight embankment with an armoured head and projects into the stream transverse to the flow.

vii) Mole-Head Spur

A mole head spur is a bar spur having increased width at the armoured rounded head.

viii) Guide Head Spur

A guide head spur has a similar behavior to that of a J-head spur with similar design features and elements.

5.3.7 Selection of Spur Type

The position, length and shape of spur depend on the site conditions and require significant judgement from the designer. No single type of spur is suitable for all locations. Model studies must be carried out to finalize the design of spur with satisfactory performance.

The selection and corresponding design of spur/groyne is influenced by following factors [12]:

- i) Fall velocity in river
- ii) Width of waterway at high, mean and low water levels in river
- iii) Depth of waterway, height of flood rise and nature of flood hydrograph
- iv) Character bed material such as shingles, boulders, sand or silt
- v) Amount of silt carried in river/stream flows
- vi) Construction material availability
- vii) Funds and financing available for construction

5.3.8 Behavior of Different Spur Shapes at Physical Models

The design of spurs is finalized after carrying out physical model studies. The physical model studies are considered essential for the finalization of design of river training works [15].

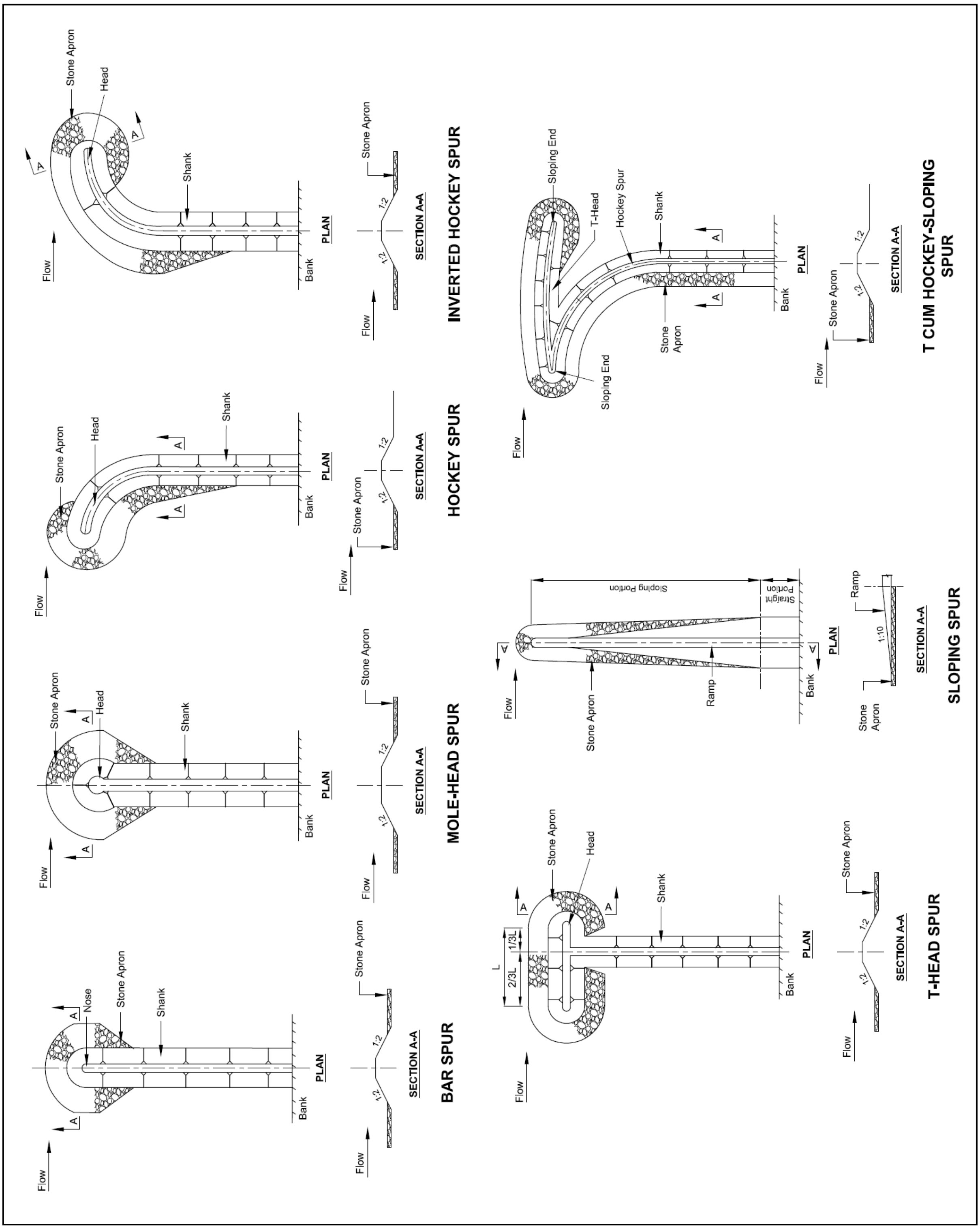

Figure 5.7: Typical Plan and Section of Different Spur Types [15]

Table 5.2: Behavior of Spur Shapes at Physical Model

Spur Type/Shape	Behavior at Physical Model
Hockey Spur	- It can be installed successfully for verifying a deeper embayment. The angle of deflection of the main flow current downstream of the spur increases with increase in embayment depth at spur upstream. - At spur nose, flow concentration takes place resulting in severe eddy and vortexformation at back of spur nose.This directs large secondary currents towards the shank. - It is essential to provide a follow-up spur at adequate distance and position downstream of this spur. Due to this follow-up spur, a pocket is formed which reduces the eddy and vortex formation by keeping away the generated secondary currents.
Inverted Hockey Spur	- In comparison to a hockey spur, the flow deflection of main currents is quite less for this type. The flow moves backward to the spur shank after moving for some distance downstream of the spur. - The spur behavior is to attractflows if it is placed in series at the river bank. - Secondary current is not generated at the back of an inverted hockey spur.
T-Head Spur	- Performance of T-head spur is reasonably acceptable when river channel approaches tangentially to the spur head and forms a mild embayment upstream of spur head. The main current is deflected with a deflection angle depending upon the magnitude of embayment forming upstream of spur head. The stone apron launches gradually and uniformly with risein flow and covers the slope below the apron down to the deepest scour level. - An eddy forms along the nose of spur head, moving up and down the armoured slope. A proper filter under the stone pitching is essential. - At deep embayment formation upstream of spur head, main current is gradually drifted towards the shank and eventuallybegins attacking it. The shank can break at the point of attack. - Smooth launching of stone apron is prevented by high flow concentration and spiral eddy formation at upstream nose of spur head. High velocity spiral currents sweep away the apron stones after lifting them up the apron level. It results in collapse of spur head. - In case of insufficient spur head length upstream of junction point and scour holeextending from nose of spur to the junction point, failure of shank will occur at the junction point. - Surging and heaving up will occur at upstream pocket of the spur head whenthe river flow approaches directly to the spur head. The rise and fall in water level is not safe for the shank.
Sloping Spur	- No secondary current gets generated at upstream of the spur. - There is less resistance to stream flow, resulting in lesser head across the spur. - With increase in stream discharge a greater length of spur comes into action. Thus the extent of the eddy reduces and the main current gets deflected away from the stream bank. - The point of maximum scour is formed away from the spur nose. - The optimum top slope for most effective performance of the spur is 10H: 1V normally adopted for larger rivers.

Spur Type/Shape	Behavior at Physical Model
	- The optimum angle of spur axis with respect to flow axis is ninety (90) degrees. - It is most effective for straight river approach and the effectiveness is decreased with river approach obliquity. - The spur will reclaim land at its upstream by holding the stream close to its nose.
J-Head Spur (T-cum HockeySlopingSpur)	- With the increase in radius of hockey-part the embayment upstream of spur head increases and the entire flow converges at the upstream end of the spur head and after leaving spur nose it swings towards the opposite bank. The silt carrying bottom current move towards the concave bank, forming a sand bar downstream of the spur. - There is no definite relationship for fixing the length of the spur head. The past experience and performance of existing spurs must be considered. - With shorter spur head radius, the extent of eddy formation at upstream end of spur reduces. The flow follows the spur head for most of its length. As the downstream end of the head bends towards the bank most of the flow is directed to it in the form of a secondary channel. In case of longer radius the tail end of the spur extends almost straight. Only a small flow is directed in the secondary channel. With the increase in the radius of spur head the eddy forming along the upstream nose increases in extent and in intensity, thus there is increase in the scouring. - When there is head-on attack of flow at spur tail end, the spur acts as a sloping spur. After impingement, the main current receives a deflection towards the opposite bank and an eddy forms along the downstream face of the shank. When tail end slope of the spur head is flattened or lengthened, the scour hole shall shift further down from the spur tail end. - When the spur head is placed parallel to the main current the flow approach is tangential to the spur head and runs almost parallel to it. With the development of scour hole at the spur head nose, embayment starts forming upstream of the spur head and deepens with time and rise in river flow. At this stage the main current is slightly deflected towards opposite bank and an eddy develops along the upstream end of the spur head. With the deepening of the embayment, the flow concentration and intensity of eddy increases and becomes dangerous to the spur head. However, with the increase in angle of orientation of spur head, the flow concentration at the spur head nose and the eddy along upstream portion of the spur head reduce generally till the eddy is eliminated completely. But at the same time an eddy develops at the back of the spur which gains acceleration with the increase in the angle of orientation and poses a threat to downstream face of the unprotected shank. The maximum angle of orientation recommended is ten (10) degrees.
Guide Head Spur	- The characteristics of guidehead spur and flow behavior around it can be considered similar to those of a J-head spur atphysical model as discussed above.

5.3.9 Studs

Studs are short bar-spurs used as protection against spill flow causing erosion along a river bank or flood embankment. These are providedalong the bank in series at appropriate locations in the river channel. The high velocity flow current is directed away by studs from the channel bank which decreases thebank erosion. The studs are tied up to the banks by extension into banks.

Pre-flood and post flood conditions vary from time to time for studs. Studs repaired and at times additional studs are added depending upon the post and pre-flood conditions prevailing in the river where studs have been provided.Since studs are economical and do not disturb the near bank environment, these are frequently provided in series in place of direct bank protection.

5.3.9.1 Types of Studs

Studs are divided into four different types depending upon velocity of flow in the main channel. The flow velocity is increased from one type to another and so is the amount of protection [9].

i) ***Earthen Stud* –** provided in spill flow channel with low velocity.

ii) ***Earthen Stud With Stone Protection* –** provided in spill flow channel with high velocity.

iii) ***Earthen Stud with Stone Protection and Armoured Leading Nose***– provided in spill flow channel with high velocity whenthe leading nose is under attack by main channel flow current.

iv) ***Solid Stone Stud*** – provided when the river bank is under attack by main channel flow current.

The basic design criteria for the stud embankments are provided below [9]:

Crest width		
Earthen stud	=	12 ft (3.5 m)
Earthen stud with stone protection	=	12 ft (3.5 m)
Solid stone stud	=	8 ft (2.5 m)
Side slopes		
Earthen stud	=	3H: 1V
Earthen stud with stone protection	=	2H: 1V
Solid stone stud	=	2H: 1V

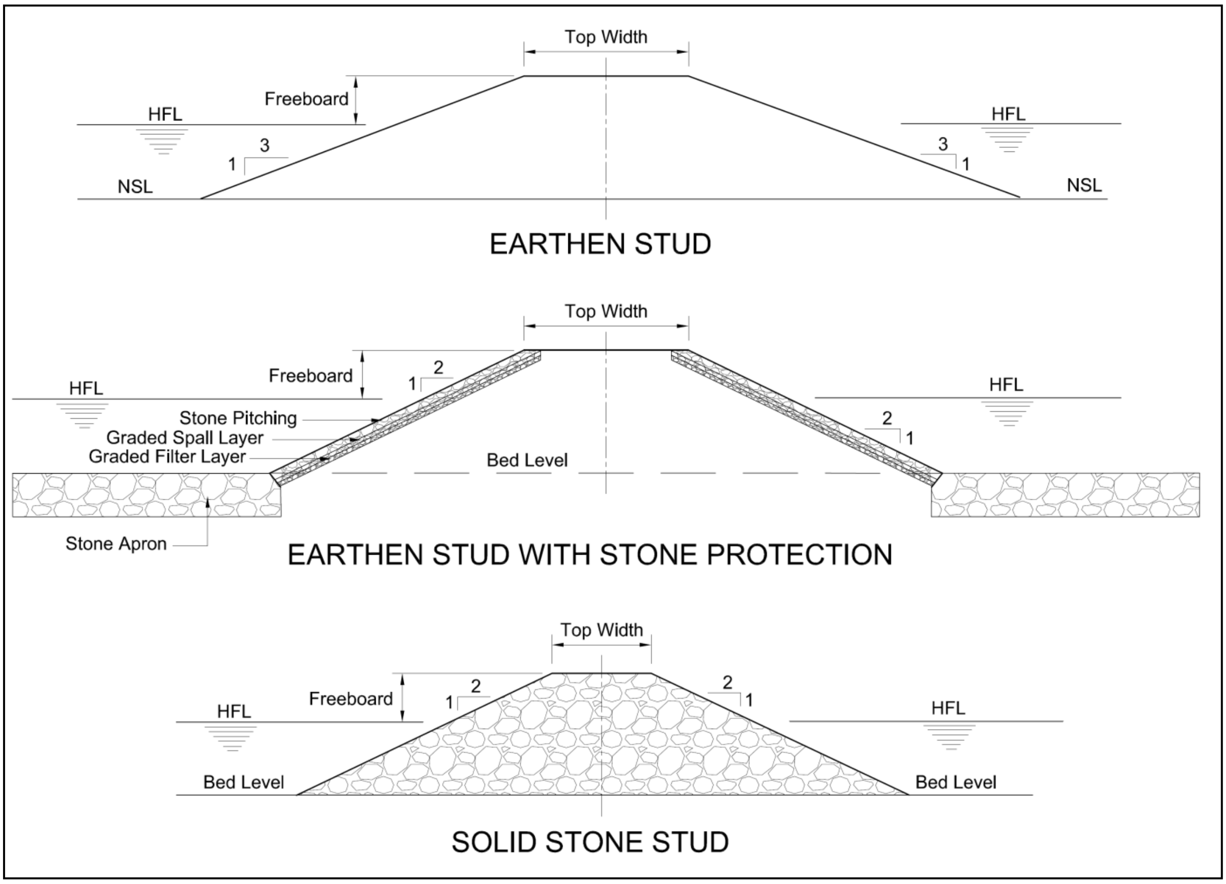

Figure 5.8: Typical Stud Cross Sections

5.4 Scour Protection

Scour occurs due to the erosive action of flowing water that carries material away from river bed and banks.For flood protection bunds and embankments scour can be calculated using empirical methods. Available engineering software's must also be used for the calculation of scour depths at river bed and banks.

Empirical methods that can be employed for the calculation of scour at river bed and banks are given below:

a. *Empirical Formulas*

USBR Methods

Scour calculation methods recommended by US Bureau of Reclamation are provided as follows:

i) Lacey's Equation (USBR expanded, 1984)

$$d_m = 0.47\left(\frac{Q}{f}\right)^{1/3}$$

$$d_s = z.d_m$$

Where,

d_m = mean flow depth at design discharge (ft)

Q = design discharge (ft^3/s)
f = Lacey's silt factor
d_s = scour depth below bed level (ft)
z = multiplication factor, for max scour depth

ii) Blench Curve (1969)

$$d_{fo} = \frac{{q_f}^{2/3}}{{F_{bo}}^{1/3}}$$

$$d_s = z.d_{fo}$$

Where,
d_{fo} = depth for zero bed sediment transport (ft)
q_f = design flood intensity ($ft^3/s/ft$)
F_{bo} = Blench's zero bed factor (ft^2/s)

iii) Neill's Equation (1973)

$$d_f = d_i\left(\frac{q_f}{q_i}\right)^m$$

$$d_s = z.d_m$$

Where,
d_f = scour depth below design flood water level (ft)
d_{fo} = avg. depth at bankfull discharge in incised river reach (ft)
q_f = design discharge intensity ($ft^3/s/ft$)
q_i = bankfull discharge intensity in incised reach ($ft^3/s/ft$)
F_{bo} = Blench's zero bed factor (ft^2/s)
m = exponent varying from 0.67 for sand to 0.85 for coarse gravel

The values of multiplying factor 'z' for Lacey's, Blench and Neill's equations are tabulated below in table 5.3:

Table 5.3: Multiplying factor 'z' values for Lacey's, Blench and Neill

Condition	'Z' value		
	Neill	Lacey	Blench
Equation Type A and B			
Straight reach	0.5	0.25	0.6
Moderate bend	0.6	0.5	0.6
Severe bend	0.7	0.75	0.6
Right angled bends	-	1.00	1.25
Vertical rock bank or wall	-	1.25	1.25
Equation Type C and D			
Nose of piers	1.00		0.5-1.0
Nose of guide bunds	0.4-0.7	1.5-1.75	1.0-1.75
Small dam or control across rivers	-	1.5	0.75-1.25

The equation types specified in above table 5.2 are classified for structure design in table 5.4 provided below:

Table 5.4:Classification of scour estimation equations w.r.t structure design

Equation Type	Scour at	Design for
A	Natural channels for restrictions and bends	Syphon crossings, natural bank and waterway for single span bridge
B	Bank line structures	Abutments, bank slope protection, spurs/groynes, dykes, guide banks, etc. pumping plants and canal regulators
C	Mid channel structures	Piling of bridge, piers, power line footings and river bed intake structures
D	Hydraulic structures across channels	Dams, barrages, bank erosion controls, rock cascade drops, gabion controls, weirs and outfall structures

Browns Formula

This formula is applied for the determination of scour at straight rivers or torrent approaches in hilly areas. The width of incised river can be determined using following formula,

$$W = 2.4\, Q^{1/4}$$

Where,

W = width of incised river (ft)
Q = design discharge (ft^3/s)

a. Rivers subjected to sustained floods

$$D = 1.32 \left(\frac{Q}{C_r}\right)^{3/10}$$

Maximum scour depth = 1.7 D

b. Rivers and torrents subjected to short lived spates

$$D = 1.0 \left(\frac{Q}{C_r}\right)^{3/10}$$

c. Rivers subjected to fluctuating flood discharge throughout the year

$$D = 0.78 \left(\frac{Q}{C_r}\right)^{3/10}$$

d. Rivers and torrent in gorges with inerodible banks and deep alluvium bed

$$D = 1.32 \left(\frac{Q}{C_r}\right)^{3/10} \text{for gorge width} \geq 2.4 \left(\frac{Q}{C_r}\right)^{1/2}$$

$$D = 2.25 \left(\frac{Q}{W_r.C_r}\right)^{3/5} \text{for gorge width} < \mathit{2.4} \left(\frac{Q}{C_r}\right)^{1/2}$$

Where,

D	=	maximum scour depth from design flood level (ft)
C_r	=	Kennedy's velocity ratio
W_r	=	width of gorge (ft)

Kennedy's velocity ratio for different bed materials is provided in table 5.5.

Table 5.5: Kennedy's velocity ratio for different bed materials

Bed Material	Value of 'C_r'
Fine Sand	0.80
Medium Sand	1.00
Coarse Sand	1.25
Fine Gravel, or Bajri	1.50
Medium Gravel	1.75
Coarse Gravel	2.00
Small Boulders and Gravel	2.50
Medium Boulders and Gravel	3.00
Coarse Boulders and Gravel	4.50

5.5 Stone Pitching

Stone pitching is to be provided where there are chances of severe wave wash damage.A layer of filter material comprising of six inches of sand and gravel is to be provided below the stone pitching. It prevents the washing away of fine particles from embankment.The practice of laying filter material under stone pitching is recommended to be continued.

The procedure for design of stone pitching for a typical flood embankment is given below.

$$\text{Wetted Perimeter} = P = 2.67\ (Q^{1/2})$$

$$\text{Dischrage Intensity} = q = \frac{Q}{P}$$

Assume Lacey's Silt factor (f) value depending upon the bed material gradation.

$$\text{Lacey's Scuor Depth} = R = 0.9\ \left(\frac{q^2}{f}\right)^{1/3}$$

Consider a factor (x) for worst scour to get worst scour depth (R').

$$\text{Worst Scour Depth} = R' = x.R$$

$$\text{Depth of Flow} = d = \text{HFL} - \text{Average RBL}$$

$$\text{Scour below bed level} = d_s = R' - d$$

$$\text{Thickness of pitching on slope} = t = 0.06\ (Q)^{1/3}$$

$$\text{Length of Stone Apron} = L = (\text{Apron length factor}).(d_s)$$

$$\text{Slope of Launched Apron} = 1V: zH$$

$$\text{Thickness of Launched Apron} = t_s = 1.25\,t$$

$$\text{Volume of Stone} = \text{Vol.} = (d_s\,.\,t_s)\sqrt{1 + z^2}$$

$$\text{Thickness of Unlaunched Apron} = T = \frac{\text{Vol.}}{L}$$

The Hydraulic Design Charts sheet 712-1, by US Army Engineer Waterways Experiment Station, provides a relation for the stability of stone considering flow velocity and stone size. Stone diameter can be determined using the following equation by Isbash [16].

$$d_{50} \text{ for Stone Apron} = \frac{v^2 \gamma_w}{2g\, C^2 (\gamma_s - \gamma_w)}$$

Where,

g	=	acceleration due to gravity	=	32.2 ft/s^2
v	=	flow velocity (ft/sec)		
γ_s	=	specific density of material (lb/ft^3)		
γ_w	=	specific density of water	=	62.4 lb/ft^3
C	=	Isbash Coefficient (0.86, when failure is by sliding; 1.2, when failure is by rolling or overturning)		

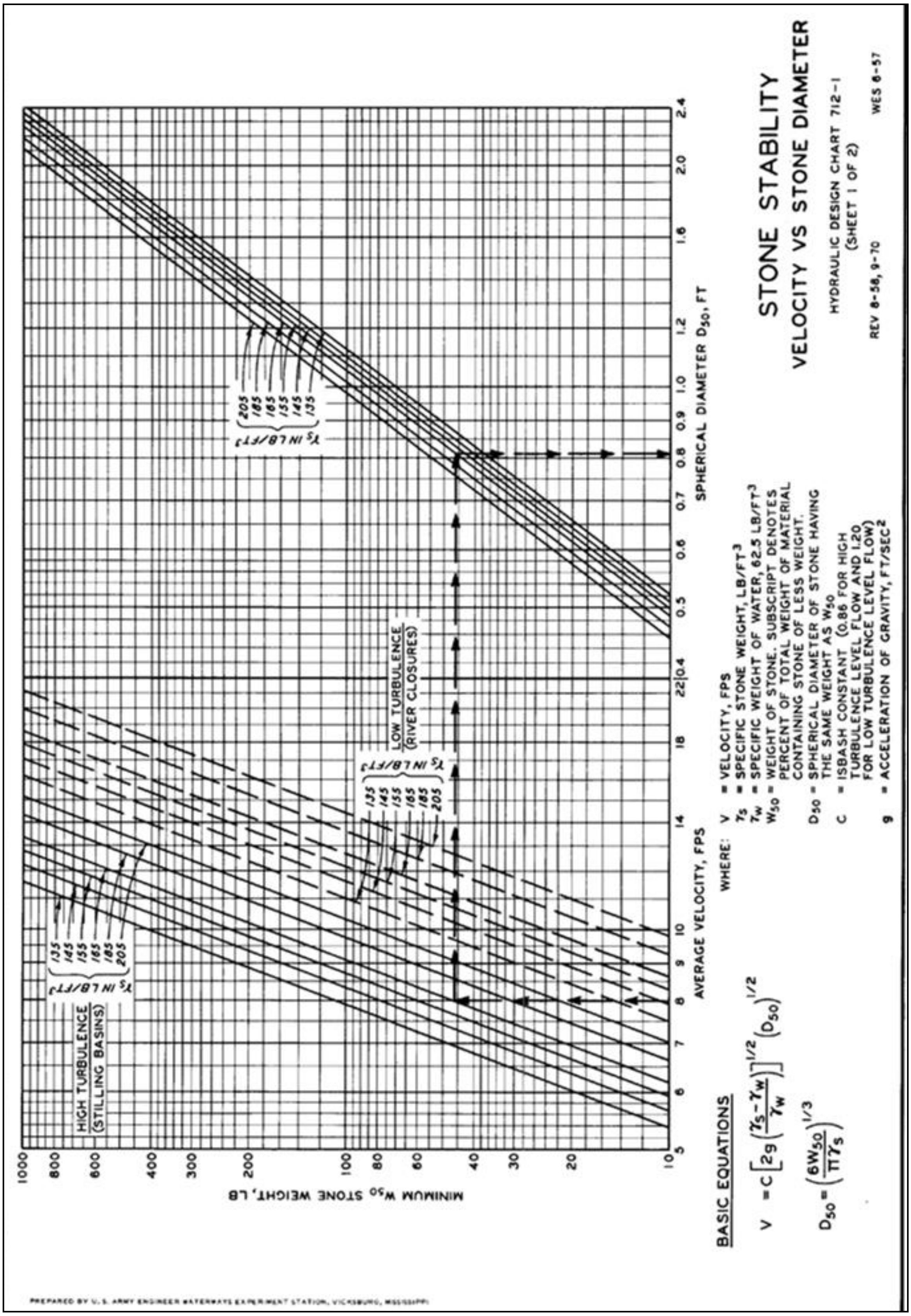

Figure 5.9: Isbash Curve - Stone Stability, Velocity v/s Stone Diameter (Sheet 1 of 2)

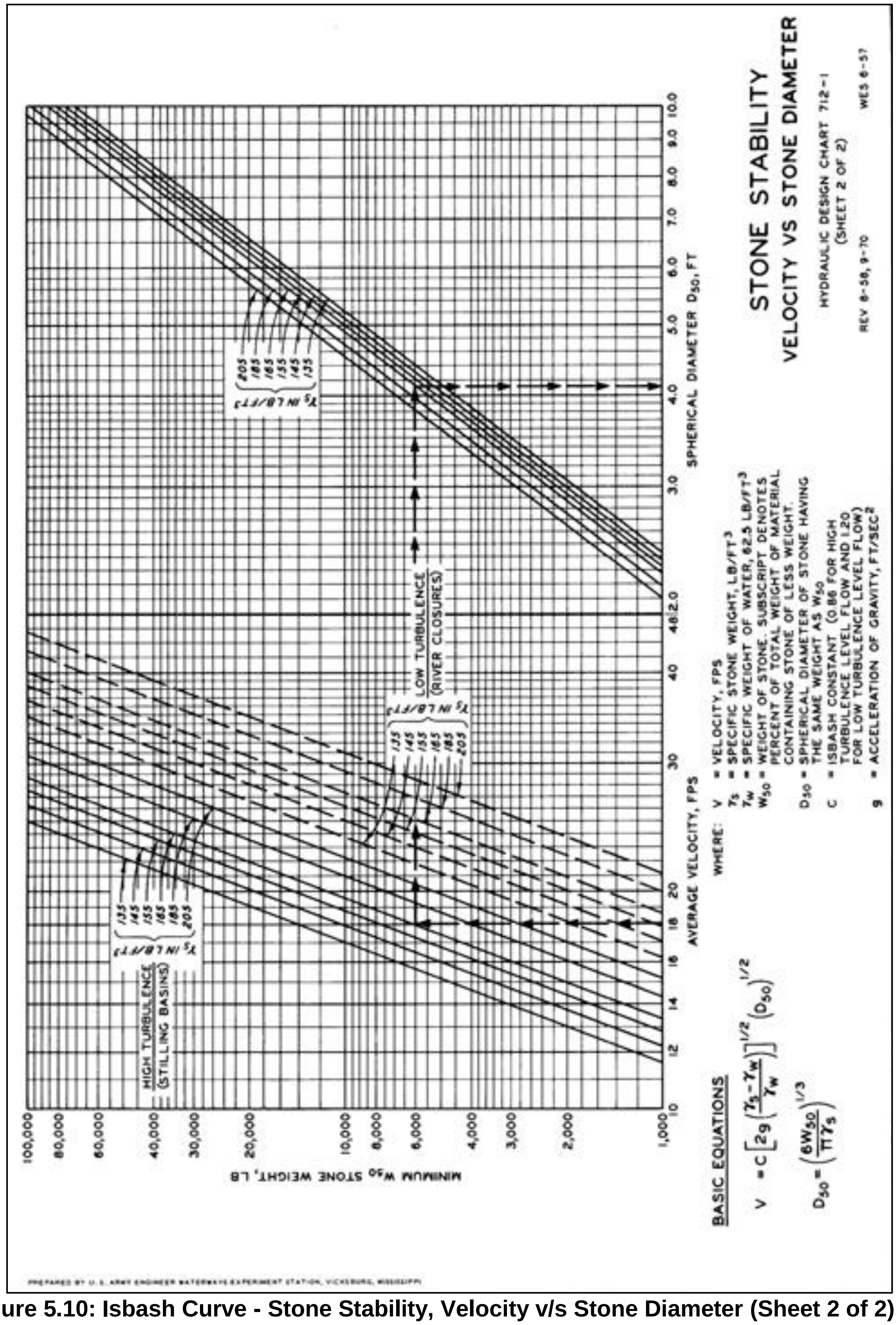

Figure 5.10: Isbash Curve - Stone Stability, Velocity v/s Stone Diameter (Sheet 2 of 2)

5.6 Geotechnical Explorations and Design

5.6.1 Collection, Review and Analysis of Existing Data and Reports

To review and evaluate the existing structures condition with respect to geology and geotechnical aspects, all the available and existing data like geological maps, geological profiles, boreholes logs, test pit logs, field and laboratory testing data will be collected for the assessment of the site conditions. All reports concerning the design, construction and operation and maintenance of embankments and other appurtenant structures will be collected from client. The available data will then be thoroughly reviewed to assess the geotechnical design. The geotechnical investigations carried out before will be reviewed with reference to the following aspects in particular:

- Adequacy of field and laboratory testing for design of new structures and evaluation of existing ones
- Validation of the geotechnical design parameters
- Geological mapping of the project area and geologic sections prepared
- Foundation geologic conditions assessed at the dam site and other structures
- Properties of construction material used in dam, barrages and spate breaker and other appurtenant structures

In addition, geological design data of flood protection and river training structures will be assessed to check the stability and safety thereof and to propose any remedial measures/ treatments if required.

5.6.2 Reconnaissance Visits

After detailed and comprehensive desk studies of all the available data, maps, reports; performa for the field visit will be prepared which will help to formulate the additional geotechnical investigation and further geotechnical and geological studies in light of scope of work if required. Geologist and geotechnical engineers / experts will visit the site to perceive, but not limited to, the following features of the project area:

- General terrain, geology, and topography of the area
- General soils/rock conditions in the study area
- Existing structures condition in the area, if any
- Need for additional geotechnical investigations
- Used construction material and its applicability

Collection, review and analysis of existing data/reports/drawings and findings of reconnaissance visit will help in evaluating existing structures health and setting up design guidelines for new structures.

5.6.3 Geological and Geotechnical Investigations/Geophysical Studies

Based on the review of previous studies and site visit observations, geological and geotechnical investigations will be proposed to be carried out for flood protection and river training structures. Geotechnical investigation/geophysical survey or field geological mapping if required will be carried as per site requirements.

Geotechnical investigations will include excavation of testpits, drilling of boreholes along withfield and laboratory tests. These will be performed as per project requirements and site condition. These investigations will be carried out to assess foundation conditions and construction material availability.

5.6.4 Subsurface Exploration

Subsurface exploration will consist of boreholes and testpits. Geophysical surveys (seismic and resistivity) will be carried out as per site requirements, i.e. if investigation is not possible or to check the subsoil condition under the existing structure these surveys must be conducted. In special cases field pumping tests and piezometers for pore pressure observation may be required.

Standard Penetration Tests (SPTs) will be performed in boreholes and disturbed and undisturbed samples taken for visual examination and testing in the laboratory.

Cone Penetration tests (CPTs) will also be performed in sandy soils to determine the subsoil characteristics.

5.6.4.1 Drilling of Boreholes

The location, actual number and depth of boreholes will be determined and finalized on the basis of review of previous data and site visit observations. Tentatively boreholes will be spaced about 100 to 300 m with additional boreholes at critical locations. The depths of boreholes are equal to the maximum height of bund but not less than 3 m. After drilling, the site will be restored to its original position and boreholes will be backfilled by using cement sand bentonite mix.

5.6.4.2 Excavation of Testpits

Test pits will be excavated to explore the engineering properties of overburden/rock at the site, and at borrow areas/quarries for construction materials. The location, actual number and depth of testpits will be determined and finalized on the basis of review of previous data and site visit observations. Ideally testpits will be excavated at those structures where boreholes will not be drilled, e.g. small bunds and embankments, approach ramps for connecting roads(if any). The depth of testpits varies from 1.5 to 3m or upto the bedrock whichever met earlier. After excavation the testpits will be backfilled by compacting in layers.

5.6.4.3 Collection of Disturbed, Undisturbed/ Core Samples

For a realistic evaluation of subsurface characteristics of soils/rock, samples (undisturbed, disturbed/core) from boreholes and test pits will be collected. The collected samples will be preserved, properly packed, marked and transported to an approved laboratory.

5.6.4.4 Water Sampling

Water samples, from the boreholes and test pits in the project area, must be collected for testing of sulfates, chlorides, total dissolved salts and pH values to determine the type of cement and aggregate specifications to be used in the construction.

5.6.5 Field Investigation Tests

The following field tests will be performed in boreholes and testpits during the course of investigations, where necessary [17]:

i) Permeability test, (ASTM D 2434)

Permeability test is performed to determine the coefficient of permeability ‘k’ utilized in carrying out seepage analysis.

ii) Standard Penetration test (ASTM D 1586)

SPT is widely used to assess the in-situ denseness of the subsurface materials. The SPT blow counts are recorded for 45 cm total penetration of split barrel sampler. The number of blows required to drive the sampler through the last 30 cm viz. 'N' values is shown on the respective borehole log sheets.

iii) Assessment of % age core recovery and RQD assessment (if rock encountered), (ASTM D 5434)

RQD is an index or measure of the quality of rock mass. RQD is computed from recovered core samples as length of intact core pieces greater than 4inches to the total length of core advancement.

iv) Field density test (ASTM D 1556)

To evaluate the in-situ dry density 'γ_d' of the subsurface soils, density tests is performed in the testpits at selected horizons below NSL. Sand replacement method is used to perform the density tests.

5.6.6 Laboratory Testing

Routine laboratory tests such as Grain Size Analysis, Atterberg Limits, and Water-Content Tests etc. are performed on most samples.The test results are correlated to estimate permeability, consolidation and shear strength characteristics. Water content is an important test. If it is observed in test results thatsample is close to the liquid limit the soil is too wet for compaction. Grain size analysis may be used to estimate the permeability of the soil using correlations.

On a few samples specialized tests such as Triaxial Compression and Consolidation tests are carried out. Compaction tests are required to be carried out for partially compacted bunds.

The samples collected during the boreholes and testpits will be sent to an approved testing laboratory for testing the index, strength, and chemical properties of the materials.

Following laboratory testing on selected soil/rock material samples is tests is to be carried out in accordance with the actual field conditions and design requirements established after review of available data and site requirement.

i) Sieve Analysis and Hydrometer Analysis (ASTM D – 421, 422 or BS 1377 Part 2)
ii) Atterberg Limits, (ASTM D – 4318 or BS 1377 Part 2)
iii) Bulk Density and Dry Density Tests, (ASTM D – 2216 or BS1377 Part 2)
iv) Specific Gravity and Water Absorption, (ASTM D-854 & C-127)
v) Consolidation Test, (ASTM D – 2435)
vi) Triaxial Compression Test, (ASTM D 4767)
vii) Direct Shear Test, (ASTM D 6528)
viii) Unconfined Compression Test, (ASTM D – 2166 & BS 1377 Part 7)
ix) Aggregate Shape Test, (ASTM D3398)
x) Los Angeles Abrasion Test, (ASTM C-535)
xi) Sand Equivalent, (ASTM D2419)
xii) Soft and Friable Particles, (ASTM C 142)
xiii) Petrography Analysis, (ASTM C-295)
xiv) Sodium Sulphate Soundness Test, (ASTM C-88)
xv) Complete Chemical Analysis of Water Samples i/e TDC, CI, SO4 & pH, (BS 1377 Part 3)
xvi) Complete Chemical Analysis of Soil Samples. (BS 1377 Part 3)

Sieve Analysis, ***Atterberg's Limits***and ***Hydrometer Analysis***are done to determine the index properties of subsoil. Subsoil classification will be used to determine the scour depths.

Specific Gravity test is also an index property test and will be utilized in the liquefaction analysis.

Bulk and dry density tests determine the in-situ moisture content, specific unit weight etc., which is utilized in the stability analysis.

Triaxial Compression tests, ***Direct Shear test*** and ***Unconfined Compression tests*** are required to determine the shear strength parameters i.e. cohesion c, and angle of internal friction Φ, modulus of elasticity along with drained soil parameters c' and Φ'.

Consolidation test is carried out to determine the rate of settlement, modulus of volume compressibility, compression index C_c, recompression index C_r, initial and final void ratios of pre-consolidation pressure p_c'. These parameters will be utilized to determine the total settlements.

Petrography test is carried out to determine the physical and chemical characteristics of the concrete or aggregate material that may be observed by petrographic *methods and that have a bearing on the performance of the material in its intended use.*

Sodium Sulfate Soundness test provides a procedure for making a preliminary estimate of the soundness of aggregates for use in concrete and other purposes. This is accomplished by repeated immersion in saturated solutions of sodium or magnesium sulfate.

Complete Chemical Analysis of Soil and Water Samplesare carried out to determine the sulfate and chloride content in soil and water samples. Amount of organic material and total salt are also determined in soil samples, however pH of water samples are also determined.

5.6.7 Seismic Studies

In order to determine the seismic design parameters for the flood protection and river training structures, the seismic hazard evaluation must be carried out at the site. The evaluations include through study of regional geological and tectonic information collected from the available literature and maps and collection of historical and instrumental earthquake records. On the basis of this data, the critical tectonic features affecting the project sites are identified and seismic hazard evaluation can be conducted accordingly [18].

Probabilistic Seismic Hazard Analysis for all important and critical flood protection and river training structures are carried out, and peak horizontal ground acceleration 'g' is determined for onward use in structural designs of these structures.

It is recommended that the project structures should be designed after deciding the seismic zone as per the Seismic Provisions (2007) of Building Code of Pakistan after giving due consideration to the foundation material at site.

5.6.8 Seepage Analysis

Quantity of water passing through a porous media such as soil is known as seepage. It is considered to be all movement of water from the reservoir through the embankment, abutments and foundation that includes porous media (inter-granular) flow, flow in fractures and concentrated flow through defects such as cracks, loose lifts, etc.

The flow of water through a porous medium like soil can be represented by the Laplace equation, which forms the mathematical basis for most models or methods of seepage analysis.

5.6.8.1 Associated Problems with Seepage

According to US Army Corps of Engineers [19], all earth and rock-fill embankments are subject to seepage through the embankment, foundation and abutments. Seepage control is necessary to prevent excessive uplift pressures, instability of downstream slope, piping through the embankment and/or foundation and the erosion of material by migration into open joints in foundation and abutments.

In order to evaluate new or existing embankments with respect to safety against seepage and design defensive measures to mitigate the effects of seepage, it is important to understand the various modes of failure that can occur due to reservoir seepage acting on an embankment or its foundation. According to Design Standard No. 13 of U.S. Department of Interior Bureau of Reclamation [20] the problems associated with seepage are:

- Excessive Exit Gradients and Uplift Pressures
- Surface Erosion
- Piping and Undermining
- Internal Migration
- High Pore Pressures
- Excessive Seepage Flows
- Hydraulic Fracturing
- Dissolution of soluble rocks and Karst formation (Karst topography is a landscape formed from the dissolution of soluble rocks such as limestone, dolomite, and gypsum. It is characterized by underground drainage systems with sinkholes, dolines, and caves)

5.6.8.2 Seepage Control Measures

Some seepage control measures are as follows:

a) Embankment Internal Filter or Drain

Drainage features and internal filter for an embankment usually include a chimney filter and/or drain placed instantly downstream of the core of the embankment, linked to a horizontal filter and/or drainage blanket that stretches to the downstream toe of the embankment. This filter and/or drain system is contained of two isolated zones to confirm both filter compatibility and suitable drainage capacity.

b) Toe Drains

Toe drains normally function as the collection system for the internal drainage arrangement in the embankment, as well as a drainage source for foundation seepage. Toe drains need to be carefully designed to completely satisfy filter criteria for both embankment and foundation soils. Toe drains typically comprise of perforated or slotted pipe surrounded by a gravel or small rock envelope which, in turn, is surrounded by filter sand or gravel.

c) Drainage Trenches

Downstream drainage trenches running parallel to the toe of the embankment can be used when downstream drainage of the foundation is needed beyond what is normally provided by a toe drain. The deeper trenches provide release of pressures and a filtered passage for seepage layers that are situated at a larger depth than would be met with a typical toe drain. Trenches are excavated and filled with filter/drainage materials of indicated gradation to avoid piping of nearby foundation soils into the trench.

d) Relief Wells

Relief wells are used to decrease undue pore pressures in permeable foundations to a bearable level. Relief wells offer safety against high exit gradients or uplift pressures. Frequently, relief wells are used to decrease artesian pressures in confined aquifers. Cautiously designed “filter packs” are employed around the well screen to confirm that foundation materials are not piped into the wells.

e) Horizontal Drains

Horizontal or semi-horizontal drains can be bored into foundations (normally in abutment areas) to release excessive pore pressures or capture seepage. Horizontal drains have been built in both rock and soil materials. Vigilant attention to screening and filtering is vital to stop the potential for internal erosion into the drains.

5.6.8.3 Seepage Reduction Measures

There are a number of different seepage reduction measures, with nearly all of them basically reducing seepage by means of lengthening the seepage path through the use of vertical or horizontal barriers. This extending of the seepage path outcomes in a lowering of the hydraulic gradient and, thus, a decrease in seepage flows. Some seepage reduction measures are as follows:

a) Embankment Core and Location

The efficiency of a wide embankment core acting as a seepage barrier should not be miscalculated. Due to low gradients through wide cores, seepage is minimized. Wide cores have been a feature of Reclamation embankments for decades and may help explain why the older embankments designed without chimney filters or drains do not experience internal erosion through the embankment. Wide cores of relatively impervious soils lead to significant head losses, as the seepage traverses long path. In addition, a wide core reduces the chance that any defect in an embankment will create a seepage path that is continuous. For that reason, past Reclamation guidance typically has been to limit the width of the core to no less than one-fourth to one-third the reservoir head. Thinner cores can be used; however, thinner cores lead to higher gradients through the core and place an even greater reliance on the filter compatibility of adjacent filter or drain and transition zones.

b) Cutoff Trenches

A well-constructed cutoff trench located beneath the core of an embankment and backfilled with impermeable soils is a very reliable means of minimizing seepage through pervious foundation soils. In addition, since the excavation of this feature enables complete view of foundation conditions, it enables a designer to gain first-hand knowledge of the foundation materials, provides the ability to adjust the design (for example, filter gradations) if needed and permits foundation treatment at the bottom of the excavation and filter protection along the downstream face of the excavation.

c) Slurry Trench Cutoff Walls

Cutoff walls constructed by slurry trench methods can effectively cut off seepage in the embankment and/or foundation of embankments. For new embankments, slurry trench cutoff walls have been used as the impermeable water barrier for an embankment (instead of an impervious earth core) or as a foundation cutoff when the bedrock (or other suitable impermeable layer) is relatively deep, making a traditional cutoff trench excavation very costly. On existing embankments, slurry trench cutoff walls have been used to reduce seepage through embankments, soil foundations, and rock foundations. These features are constructed by excavating relatively narrow trenches, typically 2 to 5 feet in width, with bentonite slurry pumped into the excavation to support the trench side walls and prevent collapse during construction.

d) Other Types of Walls

In addition to slurry trench cutoff walls, there are several other types of walls that can be designed and constructed to serve as vertical seepage barriers in embankment embankments. These wall types include sheet piles, secant pile walls, walls constructed of stiff geomembrane panels, and jet grouted or soil mixing walls. Early pile walls in embankments consisted of timber; occasionally, an older embankment with one of these walls will be encountered. However, timber pile walls are rare. Rolled steel is the most typical type of sheet pile wall, while vinyl and composite (such as fiber reinforced polymer) sheet piles are a relatively new development.

e) Grout Curtains

Grout curtains have often been used to reduce seepage through foundation and abutment rock but, as a seepage cutoff feature their effectiveness varies greatly depending on geologic conditions. Although grouting can be dependable for reducing total seepage flow through the foundation, a single “window” in the curtain can allow a shorter flow path with concentrated seepage. The effectiveness may be increased by use of multiple grout lines.

f) Upstream Blankets

Upstream blankets are a horizontal extension of the embankment water barrier (usually an earthfill core) typically used at a site underlain by high permeability foundation materials that are too deep to allow economical construction of a fully penetrating cutoff. As with the seepage reduction measures discussed previously, this feature is geared towards lengthening the seepage path in the foundation. Relatively impermeable soil materials are frequently used in an upstream blanket, although geomembranes can be an economical alternative. Because a high gradient will typically occur across an upstream blanket, it is important to ensure that blanket materials cannot pipe into the underlying foundation. This can be accomplished by designing a transition or filter material beneath the impermeable soil that meets filter criteria for the blanket and the foundation. The use of a geomembrane instead of low permeability soil will usually eliminate the need for an underlying filter, although a bedding layer and a protective cover will be needed to protect the geomembrane both during construction and throughout future operation. Since an upstream blanket is constructed of low permeability materials, it does not have to be particularly thick. The length to which the blanket extends upstream is generally more important and can be assessed by numerical seepage analysis.

g) Flat Slopes and Berms

The use of flat outer embankment slopes and berms can be an effective way of lengthening the seepage path through an embankment or its foundation and, thus, reducing seepage. In addition, downstream berms provide a means of increasing safety factors against uplift or instability due to high pore pressures in the foundation. Downstream berms can also function as seepage control measures when filters and drains are incorporated into their design.

5.6.8.4 Seepage Design Criteria

The under seepage exit gradient at the landside toe of the bund should not exceed 0.5 using steady state analysis. For bunds with a landside blanket layer a factor of safety of 1.6 for under seepage is required at the landside toe.

The under seepage exit gradient is required to be 0.8 or less at the toe of a seepage berm less than 100 m wide using steady state seepage analysis. A minimum factor of safety 1.0 for under-seepage is required at the toe of the seepage berm.

In calculating the factor of safety for underseepage, the following equations are applied.

$$FOS = \frac{I_c}{I_e}$$

$$I_c = \frac{(\gamma_s - \gamma_w)}{\gamma_w}$$

Where,

FOS	=	Factor of Safety
I_c	=	Critical hydraulic gradient
I_e	=	Calculated exit gradient
γ_s	=	Saturated unit weight of blanket layer
γ_w	=	Unit weight of water

If relief wells are constructed for seepage control, the above criteria must be achieved midway between relief wells.

5.6.8.5 Embankment Seepage Analysis Using SEEP/W Software

The manual procedures for seepage analysis are very complex, laborious and less precise. Numerical analysis, coded into computer programs, is an extensively used technique to analyze seepage issues. Embankments and other structures can be effectively analyzed for steady state conditions using Seep/W-GeoStudio(2007).

SEEP/W uses the soil permeability coefficient which is determined through field permeability tests. Permeability is the ease with which water flows through soils and/or rocks. Some soils are relatively impervious while others are pervious. A soil will be pervious when it offers the lowest resistance to the flow of water such as gravels and sands. These soils have permeability in the range of 10^{-2} to 10^{-5} m/sec. Soils which offer extreme resistance to the flow of water are called impervious having permeability value less than or equal to 10^{-8} m/sec.

For further detailsregarding the software and its analysismethodology, '*Manual on Seepage Modeling with SEEP/W, 2007*' must be referred to.

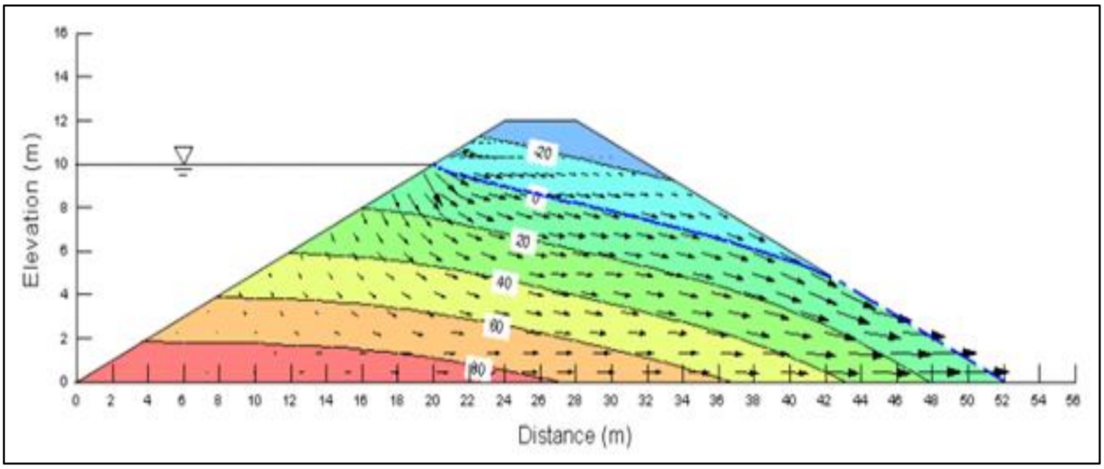

Figure 5.11: Piezometric line and pressure head contours in an Embankment using Seep/W-Geostudio (2007)

5.6.9 Stability Analysis and Design

Slope stability analysis is performed to assess safe design of human-made or natural slopes (e.g. embankments, road cuts, excavations, etc.) and the equilibrium conditions. Slope stability is the resistance of inclined surface to failure by sliding or collapsing. The main objectives of slope stability analysis is to findthe endangered areas, investigation of potential failure mechanisms, slope sensitivity determination to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilization.

The stability of slopes for different river training and flood protection structures such as embankments, studs and spursetc. are analyzed using computer software SLOPE/W-GeoStudio (2007) which is based on limit equilibrium methods. The conventional limit equilibrium methods investigate the equilibrium of the soil mass tending to slide down under the influence of gravity in terms of moment and force equilibrium factor of safety equations. Different limit equilibrium methods available in for analysis of slope stability in the software are given in Table 5.6 [21].

Table 5.6: Limit equilibrium methods for slope stability analysis

Sr. #	Method	Moment Equilibrium	Force Equilibrium
1.	Ordinary or Fellenius	Yes	No
2.	Bishop's Simplified	Yes	No
3.	Janbu's Simplified	No	Yes
4.	Spencer	Yes	Yes
5.	Morgenstren-Price	Yes	Yes
6.	Corps of Engineers - 1	No	Yes
7.	Corps of Engineers – 2	No	Yes
8.	Lowe-Karafiath	No	Yes
9.	Janbu Generalized	Yes (by slice)	Yes
10.	Sarma – Vertical Slices	Yes	Yes

Two-dimensional sections are analyzed assuming plain strain conditions (assuming that strain is zero in the direction perpendicular to the 2D plane). These methods assume that the shear strengths of the materials along the potential failure surface are governed by linear (Mohr-Coulomb) or non-linear relationships between shear strength and the normal stress on the failure surface.

The slope stability analysis provides a factor of safety, defined as the ratio of available shear resistance (capacity) to that required for equilibrium. If the value of factor of safety is less than 1.0, slope is unstable.

It is important to understand that the results (factor of safety) of particular methods can vary because these methods differ in assumptions and satisfied equilibrium conditions as given in table 5.6.

The most common limit equilibrium techniques are methods of slices where soil mass is divided into vertical slices as shown in figure5.12. Generally, all methods of analysis are very similar, differingonly in some respects. The differences between these analysis methods are as follows:

i. the equations of statics included and satisfied in analysis
ii. the inter-slice forces that are included
iii. assumed relationship between inter-slice shear and normal forces

Figure 5.12 illustrates a typical sliding mass divided into slices and the possible forces acting on the slice. Normal and shear forces act on the slice base and on the slice sides.

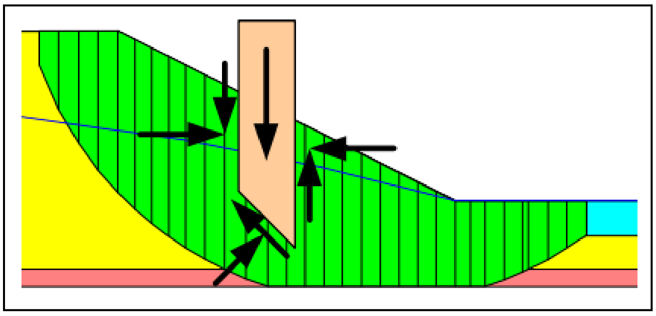

Figure 5.12: Method of slices for slope stability analysis

Table 5.7 provided below gives the inter-slice force characteristics and relationship for different methods to perform slope stability analysis [21].

Table 5.7: Inter-slice force characteristics and relationship

Sr. #	Method	Inter-slice Normal (E)	Inter-slice Shear (X)	Inclination of X/E Relationship and X-E Relationship
1.	Ordinary or Fellenius	No	No	No inter-slice forces
2.	Bishop's Simplified	Yes	No	Horizontal
3.	Janbu's Simplified	Yes	No	Horizontal
4.	Spencer	Yes	Yes	Constant
5.	Morgenstren-Price	Yes	Yes	Variable, User function
6.	Corps of Engineers - 1	Yes	Yes	Inclination of line from crest to toe
7.	Corps of Engineers – 2	Yes	Yes	Inclination of ground surface at top of slice
8.	Lowe-Karafiath	Yes	Yes	Average of ground surface and slide base inclination
9.	Janbu Generalized	Yes	Yes	Applied line of thrust and moment equilibrium of slice
10.	Sarma – Vertical Slices	Yes	Yes	$X = C + E \tan\phi$

Sarma, Spencer and Morgenstern-Price method are called as rigorous methods because they satisfy all three conditions of equilibrium i.e. force equilibrium in horizontal and vertical direction along with moment equilibrium condition. Rigorous methods can provide more accurate results than non-rigorous methods. Bishop simplified, Fellenius and others are non-rigorous methods satisfying only some of the equilibrium conditions and making some simplifying assumptions. Comparing Spencer and Morgenstern-Price method; the assumed relationships between the inter-slice shear and normal forces in case of Morgenstern-Price method are variable and user function as opposed to constant. Therefore, Morgenstern-Price method will be used here [21].

The computer program locates the failure surface i.e. critical slip surface where the factor of safety has lowest value. Stability analyses of generally layered soil slopes, mainly embankments, earth cuts and anchored sheeting structures are made. Earthquake effects, external loading, groundwater conditions, stabilization forces (i.e. anchors, geo-reinforcements etc.) can be also included in the analysis.

It must be understood that flood retaining walls cannot be designed based on this software (SLOPE/W) however, with understanding; their sliding stability can be checked.

The input parameters required for the analysis are unit weight and shear strength parameters (i.e. cohesion and angle of internal friction) of the material which will be acquired through density tests and direct shear tests/triaxial tests respectively.

Pore water pressure conditions can be specified in different ways using SLOPE/W such as linking it with SEEP/W analysis, drawing a piezometric line, input of Ru-coefficient, etc. (for detailed information 'Manual on Stability Modeling with SLOPE/W' can be referred). Pore water pressure (PWP) refers to the pressure of groundwater held within a soil or rock, in gaps between particles is measured below the phreatic level through piezometers.

5.6.9.1 Loading Conditions

Failure of a flood embankment can occur due to instability of either upstream or downstream slopes. The failure surface may lie within the embankment or may pass through the embankment and the foundation soil. The critical stages in an upstream slope are at the end of construction and during rapid drawdown. The critical stages for the downstream slope are at the end of construction and during steady seepage when the reservoir is full.

The slopes for flood embankments and other structures can be analyzed for different stages and conditions that are expected to occur during or after construction. The embankment stability evaluation requires determination of applicable and critical loading conditions. These loading conditions for different stages and conditions are discussed below:

a) *End of Construction Stage*

In this stage significant pore pressure development is expected either in the embankment or foundation during construction of the embankment.The endofconstruction loading condition is usually analyzed for new embankments that include finegrained soils and are constructed on finegrained saturated foundations that may develop excess pore pressures from the loading of the embankment.

The embankment is constructed in layers with soils at or above their optimum moisture content that undergo internal consolidation because of the weight of overlying layers. Embankment layers may become saturated during construction as a result of consolidation of the layers or by rainfall. Because of the low permeability of finegrained soils and the relatively short time for embankment construction, there is little drainage of the water from the soil during construction resulting in development of significant pore pressures.Soils with above optimum moisture content will develop pore pressures more readily when compacted than soils with moisture contents below optimum.

Both the upstream and downstream slopes of the flood embankment are to be analyzed for this condition.

b) *Steady State Seepage Condition*

This condition develops when the long-term phreatic surface within the embankment has been established.

- ***Normal Pool Condition***

  After prolonged reservoir storage, water percolates through embankment establishing a steadystate seepage condition. The upper surface of seepage is called the phreatic line.

  It is general practice to analyze the stability of the downstream slope of embankment for steady state seepage conditions with reservoir at its normal operating pool elevation. This loading condition will be experienced the most in embankments.

- ***Flood Surcharge Condition***

  When the maximum flood storage elevation is significantly higher than the normal pool elevation,the effect of raised reservoir level, or flood surcharge, on the stability of the downstream slope is generally analyzed. The flood surcharge is considered as a temporary condition causing no additional saturation of the flood embankment.Therefore, steady state seepage conditions developed from the normal operating pool elevation are used for this analysis.

- ***Partial Pool Condition***

  When reservoir is maintained at an intermediate level such as during the filling of a reservoir, the analysis of partialpool loading condition may be required by the review agencies. This condition assumes that steadystate seepage has been established at the lower reservoir level. In addition to the downstream slope, the upstream slope is analyzed for this condition to determine the pool elevation that results in the lowest factor of safety.

c) *Drawdown Pore Water Pressure Condition*

This condition develops during rapid reservoir drawdown such that drawdown is faster than the dissipation of pore pressures within the embankment after the establishment of corresponding steady state seepage conditions.

- ***Rapid Drawdown from Normal Pool***

  This loading condition assumes that steadystate seepage conditions have been established within the embankment due to maintenanceof reservoir at normal pool elevation and that the embankment materials beneath the phreatic surface are saturated. Rapid reservoir drawdown,faster than the dissipation of pore pressures within the embankment materials, results in reduced factor of safety. This loading condition is the normal operating case for pumped storage reservoirs where the drawdown of reservoir (up to 5-10 ft per hour) occurs daily. This loading condition is analyzed for the upstream slope of the embankment.

- ***Rapid Drawdown from Maximum Pool***

  When the maximum flood storage elevation is significantly higher than the normal pool elevation, an analysis of the effect of the rapid drawdown of reservoir on the stability of the upstream slope may be required. The maximum pool is considered a temporary condition causing no additional saturation of the embankment; therefore, the steady state seepage conditions developed from the normal operating pool elevation are used for this analysis.

d) Earthquake (Pseudostatic Analysis)

The earthquake causes an additional horizontal force in the direction of failure. This force is equal to a seismic coefficient times the weight of the sliding mass. The pseudostatic method of analysis is normally applied to those critical failure surfaces determined by the longterm static loading conditions, such as steadystate seepage resulting from normal reservoir pool elevation. The pseudostatic method of analysis is not usually applied to shortterm to temporary static loading conditions such as end of construction, flood storage pool, or rapid drawdown, except when this condition is the normal operating case. Where reservoir drawdown occurs on a daily cycle, such as for a pumped storage project, the earthquake loading is recommended in combination with rapid drawdown.

5.6.9.2 <u>Potential Failure Surfaces</u>

Failure surfaces within embankments fall into the following three categories:

a) Circular
b) Non-circular, or Wedge
c) Infinite Slope

In general, fill embankments will most often be analyzed using circular failure surfaces. Slope stability computer programs can quickly search for the most critical failure surfaces within a fill embankment.

Non-circular failure surfaces are used where there are weak zones in either the embankment or foundation. Examples include rock foundations with horizontal or nearly horizontal weak clay seams, alluvium underlying an embankment, the interface between embankment zones with significant strength differences, or potentially liquefiable layers within the embankment.

The Infinite Slope method is generally used to evaluate the nearsurface stability of saturated slopes with seepage. This is usually a concern for granular materials with low cohesion.

5.6.9.3 <u>Location of Potential Failure Surface for Stability Analysis</u>

The location of potential failure surfaces within an embankment takes consideration and experience. These factors will include embankment material zoning, fill strengths and the location of phreatic surface. SLOPE/W can easily perform these analyses after consideration of necessary factors.

5.6.9.4 <u>Slip Surface Criteria</u>

US Bureau of Reclamation (USBR) lists out slip surfaces that should be examined during the stability analysis of embankments [20].

(i) Slip surfaces that may pass through either the fill material alone or through the fill and the foundation materials and which do not necessarily involve the embankment crest.

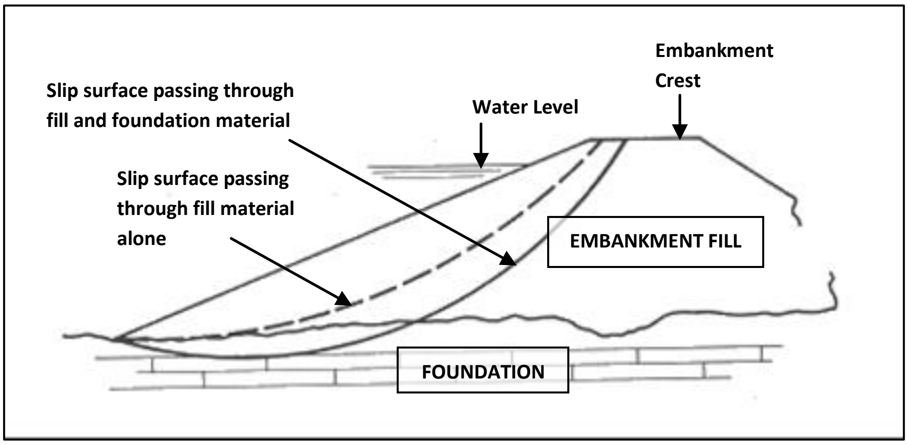

Figure 5.13: Slip Surfaces through Fill Embankments

(ii) Slip surfaces, as in preceding paragraph, that do include the embankment crest.

(iii) Slip surfaces should be examined which pass through major zones of the fill and the foundation.

(iv) Slip surfaces that involve only the outer portion of the upstream or downstream slope. In this case, the infinite slope analysis may be appropriate for cohesionless materials.

For Zoned Embankments, the slip surfaces should be examined when they pass through the following locations:

- within rockfill
- through rockfill and foundation
- through core and foundation

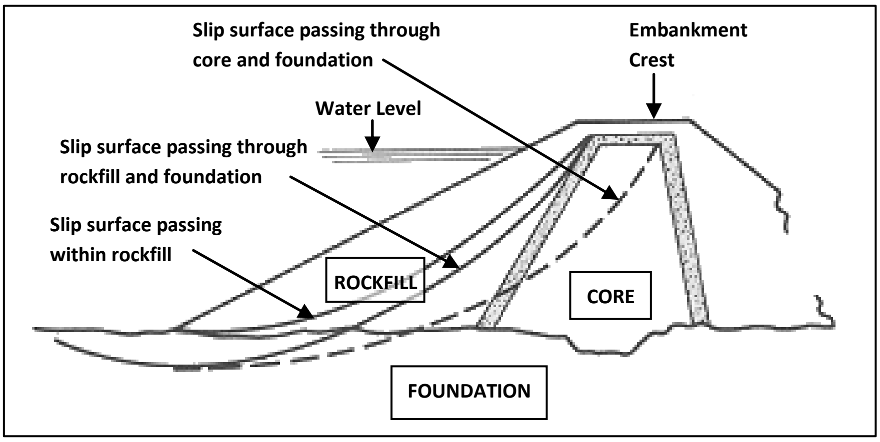

Figure 5.14: Slip Surfaces through Zoned Embankments

5.6.9.5 <u>Recommended Factor of Safety</u>

The factor of safety (FOS) is calculated as the ratio of total available shear strength or available resistance'S' along a failure surface to the total stress, or driving force mobilized 'T' along the failure surface.

$$FOS = \frac{S}{T}$$

For embankment stability analyses, the recommended factor of safety varies with the loading conditions. Long term loading conditions i.e. steady seepage, require higher factor of safety while short term loading conditions i.e. rapid drawdown, requires lower factor of safety. Factor of safety of embankment analysis for different loading conditions as given by USACE is provided in Table 5.8.

Embankment embankments for pumped storage projects may require special consideration since upstream slope frequently experiences rapid drawdown loading conditions.

A recommendation of minimum factors of safety of 1.5 for an upstream slope under rapid drawdown conditions and 1.1 for an upstream slope under rapid drawdown conditions with earthquake loading[22].

The USACE recommends minimum factor of safety of 1.4 to 1.5 for upstream face where rapid drawdown is a routine operating condition [23], and goes on to recommend that if the consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered.Factor of safety for stability analysis of flood embankments are given in Table 5.8.

Table 5.8: Factor of Safety for Embankment Stability Analysis

Agency	Loading Condition	Stress Parameter	FOS
USACE (United States Army Corps of Engineers)	During Construction and End of Construction	Total and Effective	1.3
	Long term (Steady seepage, max. storage pool, spillway crest or top of gates)	Effective	1.5
	Max. Surcharge Pool	Effective	1.4
	Sudden Drawdown for Max. Surcharge Pool	Total and Effective	1.1
	Sudden Drawdown for Max. Storage Pool	Total and Effective	1.3
	Sudden Drawdown when Routine Operating Condition (Pumped Storage Facility)	Total and Effective	1.4-1.5

5.6.9.6 Interpretation of Slope/W Results

The critical slip surfacesat upstream and downstream face of an embankment having factor of safety of 3.086 and 1.895 are shown in figure 5.15 and figure 5.16. This incorporates the piezometric or phreatic line adopted from Seep/W. For further details,Manual on Stability Modeling with SLOPE/W, 2007 should be referred.

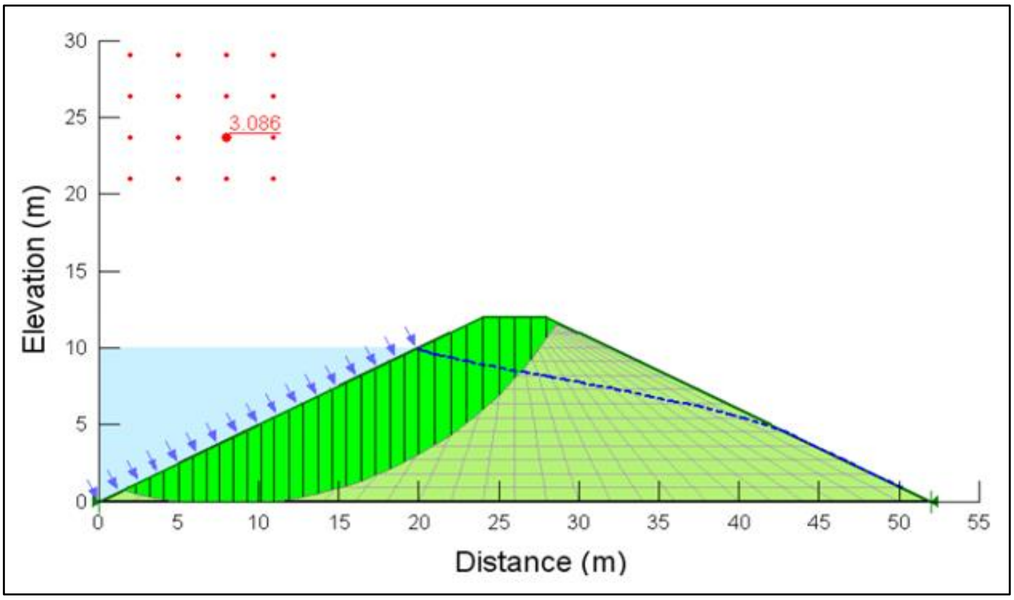

Figure 5.15: Upstream stability analysis of an embankment (Slope/W-Geostudio-2007)

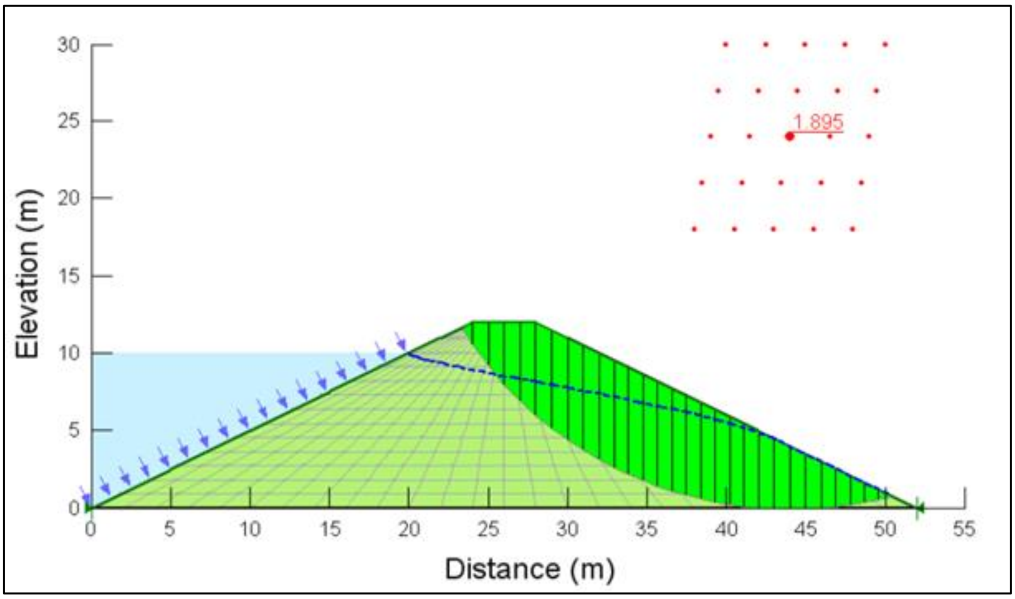

Figure 5.16: Downstream stability analysis of an embankment (Slope/W-Geostudio-2007)

5.6.10 Geotechnical Design of Cut Slopes/Excavation

Safe design of cut slopes is based either on past experience or on more in-depth analysis. A slope stability study is essential and the information required for the carrying out the analysis must include an accurate cross section showing topography, proposed grade, soil unit profiles, unit weight and strength parameters (c', φ'), (c, φ), or Su (depending on soil type and drainage and loading conditions) for each soil unit, and location of the water table and flow characteristics. Slop/W-GeoStudio(2007) can be employed to design the cut slopes [24].

Where,

c', φ' = drained shear strength parameters
c, φ = undrained shear strength parameters
S_u = undrained shear strength

Excavations and soil/ rock support will be designed to ensure that the overall slope and local inter-berm slopes meet the specified factor of safety for sliding and toppling. Factors of safety for cut slope design are given in Table 5.9 [24]. Cut slopes are generally not designed for seismic conditions unless slope failure could impact adjacent structures. These factors of safety should be considered as minimum values.

Table 5.9: Factors of safety for cut slope design

Case	Description	Factor of Safety
Usual	Normal Static Loadings	1.25
Extreme	Earthquake *OBE horizontal seismic coefficient = 0.17g for slope design*	1.1

There are different options that can be used to design an excavation. The techniques included are as follows [24]:

- Flattening slopes
- Benching slopes
- Lowering the water table
- Structural systems such as retaining walls or reinforced slopes

Changing the geometry of a cut slope seems an adoptive and economical option. Moreover, it entirely depends upon the soil type and its characteristics, and availability of space. Cut in purely dry cohesionless soils will depend on the slope angle, while the height of the cut is often the most critical parameter for cohesive soils. Thus, flattening slopes usually proves more effective for granular soils with a large frictional component. Benching will often prove more effective for cohesive soils. Benching also reduces the amount of exposed face along a slope, thereby reducing erosion.

Figure 5.17 shows the typical configuration of a soil benched slope. Structural systems are generally more expensive than the other techniques, but might be the only option when space is limited [24].

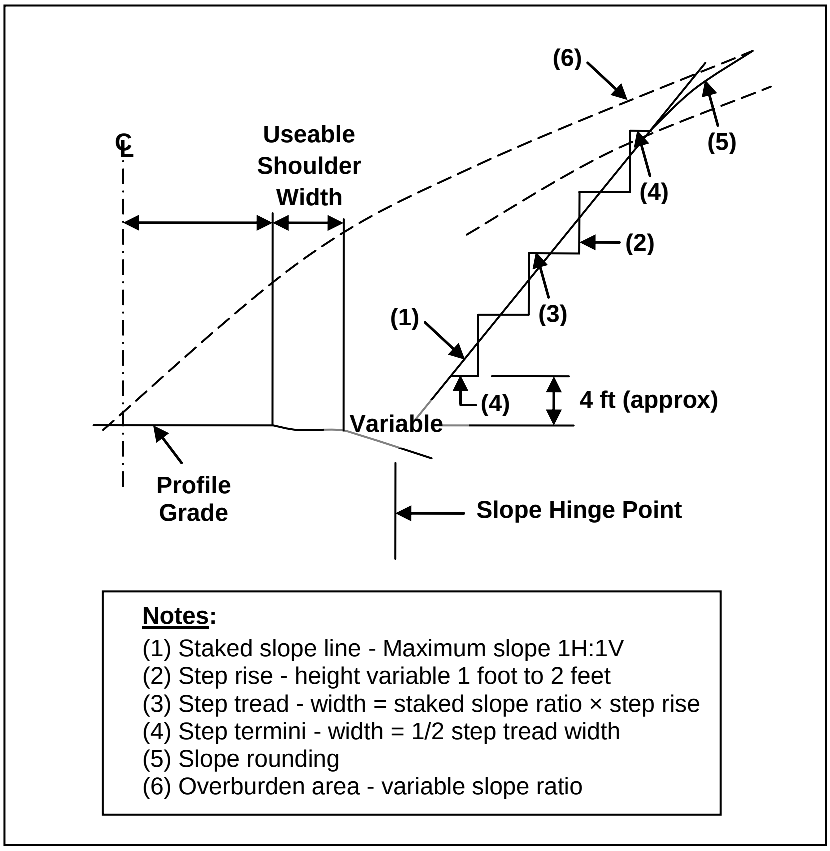

Figure 5.17: Typical Configuration of a Benched Slope (Soil)

The figure 5.18 provided below figuratively explains the concept of interberm and overall slope angles for an excavation to be carried out in rock.The berms must be provided in rock cut, where necessary, in a similar manner [24].

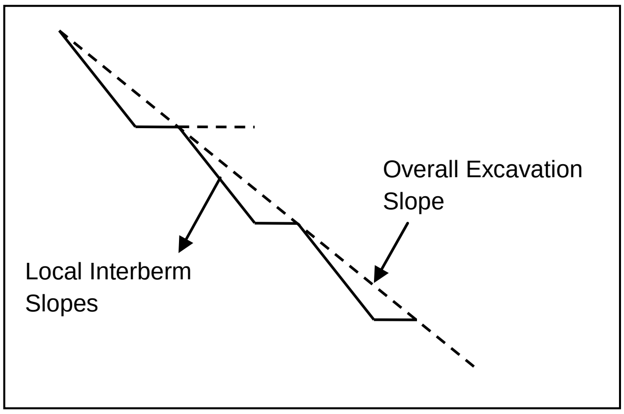

Figure 5.18: Illustration of interberm slopes and average overall excavation rock slope

5.7 Model Studies

The model studies for flood bunds and embankments can be divided in following two categories:

5.7.1 Physical Model Studies

The physical hydraulic model studies for flood bunds and river training works are carried out to:

i) assess the flood plain area for different flood conditions
ii) assess the water levels at flood bunds and embankments for different flood conditions
iii) optimizethe hydraulic design of flood bunds and embankments
iv) observe fluctuationof flood levels with respect to discharge
v) observe any back water effects or rise of water level due to flow path constriction or sudden increase in discharge
vi) observe flow depths and velocities in river reachand verify with performed calculations
vii) observe scour at river/channel bed and banks
viii) observe local scour pattern along structures for confirming the optimum level and sufficiency of flexible stone launching apron
ix) verifyif there will be overtopping of bunds or training works during extreme flood events
x) observe flow behavior at inner and outer curves of bunds

Model Types

There are two types of physical hydraulic modelsbased upon scalingratios.

i) ComprehensiveModel

This model is constructed at a suitable geometric scale for the optimization and refinement in design of flood bunds and river training works. The results obtained from the model are transferred in a quantitative manner with simple laws of similitude to the prototype.

ii) Distorted Model

This model is developed at a distorted scale i.e. horizontal and vertical scales used for scaling the model are different. Appropriate horizontal and vertical scales are selected to construct the required river reach such that necessary hydraulic parameters and phenomenon can be properly observed and recorded. The distorted scale model is used for the optimization of model configuration, scour protection, velocity, flow depth and other hydraulic aspects of the flood bunds, river training works and other embankments.

5.7.2 Numerical Model Studies

For construction, rehabilitation or refinement of a flood embankment or a spur, use of numerical models is very essential. Depending upon the scale and accuracy of topographic data, numerical models are capable of producing reasonable accurate design flood levels and design velocities for an embankment.

For planning level studies, one-dimensional hydrodynamic model can be used to obtain ready information on flood depths and velocities along a flood embankment using coarse topographic details along floodplains. For detail design, fine topographic details in models may be helpful to obtain accurate hydraulic parameters for design of embankment.

An advantage of using numerical models is that it can quickly estimate impact of a proposed embankment on water surface profiles upstream and downstream of proposed location.

Limitation of one-dimensional hydraulic models is that they provide a unique value of velocity across any location which is basically averaged over cross section extent. In contrary, flow velocities are relatively less near embankments compared to center of floodplains.

Latest innovations and developments in numerical models have provided a great support for designers in taking decisions for selecting design parameters. Use of 2-D and 3-D models virtually eliminates the need of physical models, provided they are good calibrated with the field conditions.

To accurately estimate magnitude of velocity, streamline behavior, flood depths and various other hydraulic parameters along a flood embankment/ spur, use of 2-D or 3-D hydraulic models is highly recommended.

5.8 Construction Material

The sourcesof construction material for the construction of flood protection structures and river training works must be selected appropriately to meet the site and design requirements since these have a major impact on durability, performance and quality of the structures. The identification of suitable material sources require systematic investigation to be planned and carried out. These sources are to be identified and confirmed for their appropriateness in accordance with the site requirements.

For this purpose a comprehensive reconnaissance of the surrounding area and project locality is to be made for identificationof potential sources for different natural materials available for construction. Various potential borrow areas and quarry sites will be investigated. The final choice will depend on the availability of suitable materials, slope stability analysis along with haulage distance of borrow area to the project site, which must be cost effective.

Following major aspects must be considered for investigation of potential construction material sources:

5.8.1 Evaluation of Existing Material SourceSites

The evaluation of existing construction material sourcesites and quarries will require the review of any existing material data and available quarry information. The following data must be reviewed:

- Site Geology from existing mapping, aerial photographs, onsite testing, reports etc.
- Past quality testing and production history of the material source sites
- Surface and subsurface drainage atconstruction site
- Seasonal fluctuations in the water table, including water wells located at adjacent land

5.8.2 Geological Field Investigations

Geological field investigations must be carried out to ascertain the appropriateness of the contraction material source so that obtained material complies with the design requirements and site conditions. In order to begin with these investigations, a reconnaissance survey must be carried out at identified and potential material sites so that the specific geology at that site can be understood.

Some major aspects considered during the initial site reconnaissance shall include the following:

- Site topography
- Geology

- Test pits
- Test holes
- Representative photographs of the site
- Geologic mapping of existing exposures
- Mega and microscopic study for the assessment of mineral composition of material source
- Laboratory testing of materials

5.8.3 Detailed Site Exploration

The detailed site explorationswill include the following:

- The test pits and boreholes are made at suitable locations and logged for appropriateborrow material and water table.
- The selection of representative material samples from test pits and boreholes is to be made for necessary material quality testing.
- For quarry site investigation, wet rotary rock coring methods are to be used to determine subsurface conditions and acquire samples for testing.
- For riprap sources, careful measurement of fracture spacing is to be carried out for assessment of rock block sizes that can be produced by blasting.
- For geophysical explorationof the material source, geophysical methods are employed which include seismic refraction surveys and electrical resistivity surveys.
- Surface drainage at the site must be determined noting areas of ponding water, flooding possibilities or surface flow after periods of heavy rainfall.

5.8.4 Material Source Report

The report developed for the identified construction material sources and quarries must include the following:

- Assessment for quantity of material available in the potential material source.
- Assessment of largest size cobble or boulder observed during the investigation along with any glacial irregularities and variations.
- Qualitative evaluation of all material sources for sand and aggregates will be carried out in accordance with the site requirements and specific potential of each source, which shall include:
  - (i) assessment of mineralogical characteristics of the rock mass
  - (ii) assessment of physical engineering characteristics
  - (iii) evaluation of results to establish appropriateness of sources to be used for construction material acquisition.
- Identificationfor the presence of deleterious minerals with respect to alkali-silica and alkali-carbonate reactions and establishmentof their percentages.
- Accessibility and haulage
- Suitability of construction materials with reference to physical strength and petrographic properties
- Study for the availability of other manufactured materials such as cement, steel, bricks and admixtures, to be used in bulk quantities, for construction of flood retaining structures.
- Petrographic analysis to ensure the potential of ASR (Alkali Silica Reaction)in the selected fine and coarse aggregate samples collected during investigations along with other properties of rock, soil and aggregate.

5.9 Construction Practices and Procedures

5.9.1 General

The construction planning for works envisaged in any flood management/river training works is a vital component for the timely completion of the works avoiding time and cost overrun. Time is of high essence of flood management works as the same has to be completed in available non-monsoon season. Construction Planning becomes part of the overall activity starting from off-setting of monsoon which include (i) vulnerability/damage assessment; (ii) Type of measures to be taken, a Model Study is performed to adopt proper measures (iii) Design of structures, construction planning and preparation of drawings and estimate, (iv) Administrative approval of drawings and estimate and (v) Implementation of the works while keeping sufficient time for each activity. Time to accommodate unforeseen issues should also be kept in mind.

Implementation of flood management/river training works include invitation of tenders for various works, site survey like latest river configuration, site clearance etc, mobilization of resources like men, material at the site in pre-organized manner for various works.

Construction methodology includes proper construction of the works as per the approved design and drawings. This also includes the in-time procurement, mobilization and proper installation of the construction materials being used for the works.

5.9.2 Construction planning

It is understood that construction planning is the key for in-time completion of the flood management and river training works. It is seen from the past experience that most of the projects are delayed in completion due to lack of proper construction planning.

For a proper completion of a project, the storage and installation of new innovative material for construction of embankments, revetment, spurs etc. for project specific problem need to be executed under well trained guidance and accuracy. The planning for the same needs to be done considering all the situations like working season, monsoon season, land acquisition, site survey and clearance, procurement of materials etc. The Implementation of project may involve following steps.

5.9.3 Invitation of Tenders

Model tender documents for procurement of materials include stone, filter materials, wire/wire-mesh etc. for various civil works including earth work, stone pitching and launching of apron.

5.9.4 Site Preparation

Soundings are taken in river water before commencing work for accurate measurement /quantification of payable work. Any loose material or slush is removed from the site. Vegetation or other types of deleterious material is removed from the site.

5.9.5 Layout of Structures

Layout of the structures is made as per Model study and Design. Layout with lime is done outside river while poles are erected in the river Control points are established at various locations to ensure the alignment of the structures as per recommendations of the Model Study. To avoid any conflict or ambiguity, expert/ research officer under whose supervision Model Study was performed, is sometimes consulted to finalize the location and layout of the structure.

5.9.6 Procurement of Construction Materials

Construction materials, required frequently in large quantities including boulders, sand, stone, wire-mesh etc. should be procured well in advance preferably during monsoon season to save time.

5.9.7 Storage of Construction Material at Site

There should be proper space/shed for the storage of construction material. The storage space/shed should be such that, there is no risk of theft of the construction material till the works are over. The location of storage should be easily accessible from the site of work.

5.9.8 Testing of Construction Material

There should be arrangement of testing of the construction material before the start of the work. Provision for standard testing along with procedure of testing should be made a part of the tender document. All the construction material should possess qualifying standards before construction.

5.9.9 Construction Methodology

The construction methodology and equipment planning for various works must be based on the site conditions that prevail at the project area. Construction activities should be planned in such a manner that the project is completed in shortest possible time period. The activities can be divided into pre-construction and construction (during construction) activities.

The pre-construction activities such as land acquisition, infrastructural works and procurement, transportation and testing of materials are to be completed prior to commencement of construction activities. All the construction activities for the project have to be executed in phases while certain construction works can be executed simultaneously.

i) Construction Methodology for Earthwork

- The borrow area should be approved by the Engineer-in-charge after satisfying standards before starting transportation of fill materials. Extensive testing should be performed on materials to check its suitability for intended purpose. Plant roots, vegetation, plastic bags or other deleterious materials that may interfere with the quality of work may be sorted out carefully and discarded from the site. Lead of the borrow area is measured.

- The fill materials are spreaded at the location of structure in layers of specified thickness. Water is added and mixed thoroughly up to optimum moisture content. A no. of passes of roller of 8-10 tonne capacity are then applied. A vibratory roller may be a better option. Compaction tests are performed to ensure the compaction of the fill. At least one test per layer per 500 ft length or as specified will be required. The practice is performed until the desired level is achieved.

- Extra fill material from the slope of structure is removed and the slope of the structure is trimmed to design so that it can be prepared for other activity.

ii) Construction Methodology for Stonework (Stud, Spur, Stone Pitching and Launching of Apron)

- The natural surface should be excavated to the required level as per the drawings for formation of the base of the structure. The surface of formation should be free from any deleterious material and unwanted foreign objects. Loose pockets if any, should be excavated and filled with suitable granular or backfill material.

- The excavated surface should be compacted by using the roller of 8 to 10 tonne capacity. The design requirements with respect to bearing capacity should be achieved and verified before proceeding further. Density of compacted formation should be as per specifications.
- The excavated surface prepared should be leveled without ruts and undulations.
- Stone of specified weight (size) and quality should be offloaded at site.
- The apron should be excavated to the desired level and size. This activity should be performed as quickly as possible because in many cases in river works the soil may be wet that may collapse inward.
- Stone should be dumped in apron but great care should be taken to avoid loose pockets that should be filled with smaller size of stones. Care should be exercised that no earth materials enters the apron.
- The top layer of stone apron should be hand packed.
- Earth filling should be started in layers. Each layer should be moistened up to optimum moisture content and should be compacted up to specified standard. Care should be exercised to remove any deleterious materials like plastic bags, vegetation etc.
- In next step, the filter materials of specified gradation and quality should be spread over already prepared slope in specified thickness.
- Then stone profiles should be placed at an interval not greater than 100 ft. upto required height so that the overall thickness of stone pitching between profiles of specified thickness can be maintained. Any loose hole should be avoided in pitching and should be filled with smaller size stone.
- In case of studs, stones will be dumped above apron level in specified shape. Care should be taken to avoid loose pocket which should be filled with smaller size stone.

iii) Construction Methodology for Gabion/Crated Structures

The following sequence may be followed in the construction of Gabion/crates structures.

- The natural surface should be excavated to the required level as per the drawings for formation of the base of the structure. The surface of formation should be free from any deleterious material and unwanted foreign objects. Loose pockets if any, should be excavated and filled with suitable granular or backfill material.
- The excavated surface should be compacted by using the roller of 8 to 10 tonne capacity. The design requirements with respect to bearing capacity should be achieved and verified before proceeding further. Density of compacted formation should be as per specifications.
- The excavated surface prepared should be leveled without ruts and undulations.
- The crates may be placed over the leveled surface and should be connected with each other.

- The top and bottom ends of the panels may be stretched along the longitudinal direction. This arrangement will keep the front and back panels in tension during the rock filling operation.

- Boulders, for filling the gabion, may be obtained from nearby quarry. The boulders may be selected in such a way that at least one face is flat-shaped. Boulders of smaller size (40 mm to 100 mm) may be used for packing the voids and reducing the porosity.

- After the filling of gabion, the same may be tied securely so that boulders filled inside the gabions are kept intact. It is also recommended to slightly overfill the baskets by 25 to 50 mm to allow for settlement of the boulders.

6. FLOOD RETAINING WALLS

6.1 General

A retaining wall is any wall that retains material to maintain a change in elevation. The principal function of a flood retaining wall is to prevent flooding or inundation of the adjacent land that is to be protected against floods [25].

A flood retaining wall is subjected to water force on one side which is generally greater than the earth resisting force at the opposite side. Hence, this wall may act as a retaining wall for one loading condition and as a flood retaining wall for flood loading condition. The flood loading (surge tide, river flood, etc.) may either be from the same or the opposite direction as the higher earth elevation [25].

6.2 Factors for Selecting Flood Retaining Wall or Embankment

The selection of flood retaining wall or embankment as a flood protection structure depends on various factors including site conditions. Main factors for choosing between flood wall and embankment are summarized in table 6.1 below [26].

Table 6.1: Factors for selecting a flood retaining wall or embankment

Factor	Flood Retaining Wall	Embankment
Space	Ideal when space available for constructing a flood retaining structure is limited.	Requires a lot of space and a wide foot print.
Environment	Ideal for urban conditions where the designed structure is to blend in with local infrastructure.	Ideal for rural locations but an be provided at urban places if space permits.
Foundations	Weak and permeable foundations can complicate the structural design and stability.	A weak and permeable foundation threatens the stability of the embankment.
Seepage	It requires a cutoff for safety against seepage action.	A pushta or back berm is provided to prolong flow path for safety against seepage which increases the foot print (base width) of the designed embankment.
Inspection	Inspection of critical elements should be done before and after floods.	It requires regular inspection.
Maintenance	It requires less maintenance.	It requires careful maintenance including control of unwanted vegetation control, burrow holes and repair of any damages to embankment of protection works.
Cost	The cost depends upon construction	Cost depends mainly upon fill

Factor	Flood Retaining Wall	Embankment
	materials, construction methods and foundation condition and treatment.	material. If local material is allowed for fill by the Engineer, costs are significantly reduced.

6.3 General Failure Modes of Flood Retaining Walls

In general, the flood retaining walls can experience following failure modes [26]:

i) Failure due to Overtopping

The flood retaining walls are generally capable of sustaining an overtopping flood event and do not failure under it. However, the collapse or failure during overtopping can result in severe consequences. It will result in breach of the flood protection system.

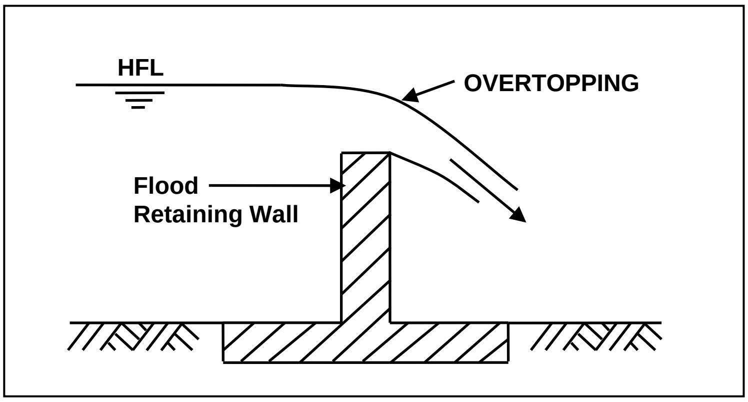

Figure 6.1: Overtopping Failure

ii) Failure due to Overturning/Rotation

The floodwall can overturn or rotate under the effect of hydrostatic loads which may include uplift pressures under the wall foundation. A partially rotated wall may remain stable for a while but can lead to collapse under further exerted loads at any time without further warning.

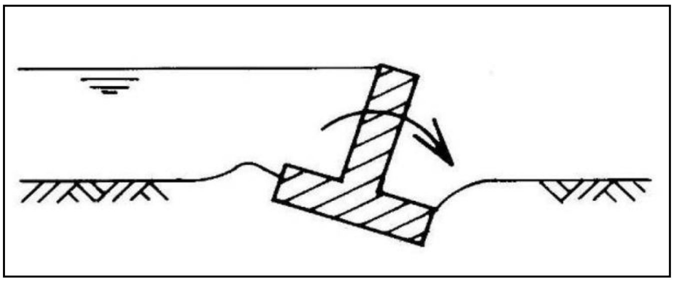

Figure 6.2: Overturning Failure

iii) Failure due to Sliding

Failure due to sliding can result in opening up or development of cracks between adjacent parts of the flood retaining structure and may lead to compromise of flood retaining wall foundations.

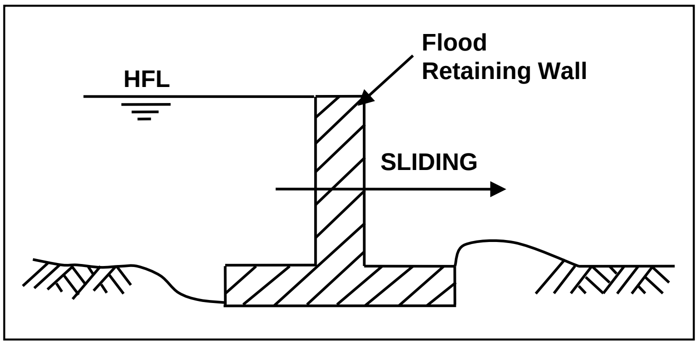

Figure 6.3: Sliding Failure

iv) Failure due to Seepage

The amount of seepage is usually very low but can be severe. Excessive seepage problem can lead to local flooding and cause damage to foundations of flood retaining walls.

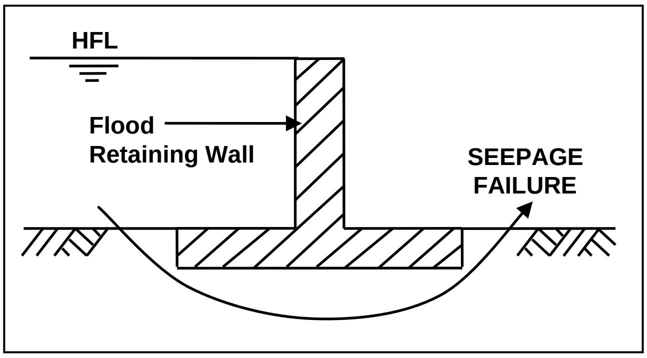

Figure 6.4: Seepage Failure

v) Failure due to Undermining/Piping

The undermining or piping leads to washing away of fine particles from the base of the structure and results in formation of voids. It can result in sliding or rotational wall failure.

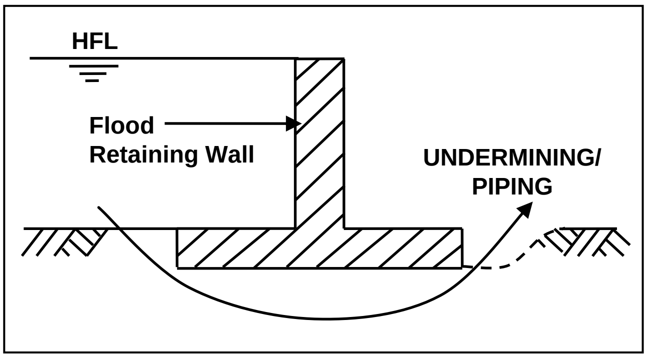

Figure 6.5: Undermining or Piping Failure

vi) Structural Failure

A structural failure of flood retaining wall relates to the inability of the structure to retain the exerted hydrostatic and hydrodynamic loads. Shear failure to the structure can occur. A sudden collapse or structural failure is dangerous and may lead to rapid inundation of the surrounding area without any warning.

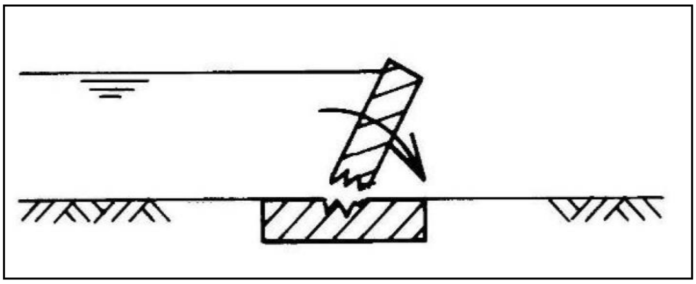

Figure 6.6: Structural Failure

6.4 Freeboard Estimation and Wall Top Level

The flood retaining walls must be high enough to avoid overtopping so that there is no spillage of flood flow beyond the flood retaining structures and the area to be protected is safe under floods.

Freeboard must be provided above the highest flood elevation to avoid overtopping. The height and run-up of generated waves must be considered in design. The formulas and criteria given in section 5.2.1 for calculation of wave height and wave run-up for embankments can also be used to fix freeboard for flood retaining walls. The top level of flood wall is to be fixed by adding estimated freeboard to the design flood level.

In case the flood wall is overtopped due to any unforeseen event or extreme flood scenario, the structure design must be such that no collapse or failure occurs. Generally, the flood retaining walls made up of concrete have the ability to sustain flood overtopping.

6.5 Geotechnical Design Criteria

6.5.1 Foundation Design

The foundation design is made keeping in view the type of flood retaining structures, topography of the area and subsoil characteristics. For safe and economical design, foundations of all structures should meet the following design criteria:

a. Foundations should be safe against shear failure of the supporting ground. A factor of safety of 3.0 is adopted for this purpose.

b. Foundations should not settle excessively under the service loads. A limit of 25 mm has been put on the total settlement of individual foundations and 50 mm on the total settlement of mat foundations. Similarly, the angular distortion between the edge and the centre of the foundations should not exceed 1/500.

6.5.2 Foundation Bearing Capacity[17],[24], [27], [28]

Bearing capacity of subsoil for shallow and deep foundations is evaluated by in accordance with the guidelines given in USACE manuals *EM1110-1-1905* and *EM1110-1-1902*. The ultimate bearing capacity calculated by these methods is divided by a factor of safety of 3.0 to obtain allowable bearing capacity to be used in design.

6.5.3 Liquefaction Analysis

Liquefaction phenomenon occurs when saturated loose sand deposit is subjected to a load of very short duration, such as during earthquakes or blasting etc. The loose sand deposit is densified during shear and that tends to squeeze the water out from the pores. Liquefaction analysis is carried out to investigate the liquefaction-induced deformation of embankments.

The parameters are developed based on SPT-N values and the analysis is carried out depending upon the substrata and peak ground acceleration 'g' factor. The peak ground acceleration is to be selected from Building Code of Pakistan(2007), Seismic Provisions, updated after 2005 earthquake.

The liquefaction potential of a facility can be screened using the Seed and Idriss Method, considering the following criteria [29]:

6.5.3.1 Fines Content and Plasticity Index

Seed and Idriss(1982) reported that liquefaction potential in a soil layer increases with decreasing fines content and plasticity of the soil. Cohesionless soils having less than 15% by weight of particles smaller than 0.005 mm, a liquid limit less than 35%, and in situ water content greater than 0.9 times the liquid limit may be susceptible to liquefaction.

6.5.3.2 Saturation

Although soils with low water contents have been reported to liquefy, at least 80% to 85%saturation is generally considered to be a necessary condition for soil liquefaction. The highest anticipated temporal phreatic surface elevations should be considered when evaluating saturation.

6.5.3.3 Depth below Ground Surface

If a soil layer is within 15.2 m (50.0 feet) of the ground surface, it is more likely to liquefy than deeper layers. For deeper layers,liquefaction is not allowed to occur due to the presence of overburden pressure.

6.5.3.4 Soil Penetration Resistance

Seed et al (1985) stated that soil layers with a normalized corrected SPT blow count [$(N_1)_{60}$] less than 22 have been known to liquefy. Marcuson et al (1990) suggested an SPT value of [$(N_1)_{60}$] less than 30 as the threshold to use for suspecting liquefaction potential.

Where,

$(N_1)_{60}$ = corrected SPT blow count

If three or more of the above criteria indicate that liquefaction is not likely, the potential for liquefaction can be dismissed. Otherwise, a more rigorous analysis of the liquefaction potential at a facility will be required.

The following factor of safety should be used, unless superseded by rule, when demonstrating that a facility will resist failures due to liquefaction.

Liquefaction analysis FOS > 1.0

6.5.4 Liquefaction Analysis Methodology

A liquefaction analysis should, at a minimum, address the following:

a) Developing a detailed understanding of site conditions; the soil stratigraphy, material properties and their variability, and the areal extent of potential critical layers. Developing simplified cross sections amenable to analysis. SPT procedures are widely used in practice to characterize the soil (field data are easier to obtain on loose cohesionless soils than trying to obtain and test undisturbed samples). The data needs to be corrected as necessary, for example, using the normalized SPT blow count [(N1)60]. The total vertical stress (so) and effective vertical stress (so') in each stratum also need to be evaluated. This should take into account the changes in overburden stress across the lateral extent of each critical layer, and the temporal high phreatic and piezometric surfaces,

b) Calculation of force required to liquefy the critical zones, based on the characteristics of the critical zone(s) (e.g., fines content, normalized standardized blow count, overburden stresses, level of saturation),
c) Calculation of the design earthquake's effect on each potentially liquefiable layer should be performed using the site-specific in situ soil data and an understanding of the earthquake magnitude potential for the facility, and
d) Computing the factor of safety against liquefaction for each liquefaction susceptible critical layer.

For liquefaction potential analysis, the "Simplified Procedure," developed by H. B. Seed & I. M. Idriss will be used. Details of this procedure are given in RCRA Subtitle D(258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities by U.S. Environmental Protection Agency [30].

6.5.5 Stability Concerns[30]

The primary concerns for the design of flood retaining wall structures are as follows:

a) Stability against overturning by adopting an acceptable factor of safety
b) Stability against sliding by adopting an acceptable factor of safety
c) The stresses developed within the components of flood retaining wall, i.e. stem and footing,may not cause the stress failure of the structure by resistingthe imposed vertical and lateral loads

The concrete flood retaining walls should be designed using the design procedure given by Nilson and Darwin in Design of Concrete Structures. The safety factors (FOS) applied to the flood retaining wallsto meet above design concerns are given below[31]:

6.5.5.1 FOS against Sliding

The factor of safety against slidingfor flood retaining walls is taken as 1.5. Since the passive resistance of the cutoffs is ignored, this safety factor is deemed to be adequate.

6.5.5.2 FOS against Overturning

The flood retaining wall is considered to be safe against overturning provided that the soil reaction is within the middle third of the base and that the soil bearing pressure does not exceed the allowable pressure 150 kN/m^2.

6.5.5.3 FOS against Stress Failure

In order to ensure flood retaining wall safety against stress failures, the tension and bending stresses within the mass concrete base should not exceed 2.0 N/mm^2 (for concrete with a characteristic strength of 20 N/mm^2). The factor of safety on allowable tensile stress is 1.5, from design literature, which is quoted as 3.0 N/mm^2. The compressive strength within the concrete is to be limited to the characteristic strength.

6.5.6 Coefficients of Lateral Earth Pressure

The cohesive material identified below natural surface level(NSL) at the project sites should not be used as the backfill material behind the retaining walls, if required. It is recommended to use granular material as the backfill material behind the retaining walls. The sands should be compacted to at least 90% Modified Proctor density.

The lateral earth pressure coefficients for active (K_a), at rest (K_o) and passive (K_p) conditions, (for $\Phi=30^0$), using sand backfill, are recommended as follows:

$$K_a = \frac{(1 - sin\emptyset)}{(1 + sin\emptyset)} = 0.33$$

$$K_p = \frac{(1 + sin\emptyset)}{(1 - sin\emptyset)} = 3.0$$

$$K_o = (1 - sin\emptyset) = 0.5$$

The lateral earth pressures to be used in design should be increased for the additional residual earth pressures to be induced by the effect of compaction, as per provisions of Naval Facilities Engineering Command (NAVFAC) Design Manual [24].

6.6 Structural Design Criteria

This section focuses on the structural design requirements of flood retaining walls. The technical data, design assumptions, codes of practice, methods and procedures to be adopted in the structural design of flood retaining walls are given in following sections.

During the structural design, various design parameters such as loads, foundation substrata characteristics, environmental data etc. are to be decided and fixed. Also, the type and quality of materials to be used in construction, as well as the allowable factors for safety, stability and stress levels have to be finalized.

6.6.1 Measurement Units

System International (SI) Units shall be used in design.

6.6.2 Codes and Standards

The Codes and Standards used for structural design are provided in table 6.2.

Table 6.2: Codes and Standards for Structural Design

Sr.#	Code and Standard	
1.	**BCP (2007)**	Building Code of Pakistan Seismic Provisions - 2007
2.	**EM 1110-2-2502**	Engineering and Design of Retaining and Flood Walls by US Army Corps
3.	**PCPHB (1967)**	Pakistan Code of Practice for Highway Bridges
4.	**ACI 318-11**	Building Code Requirements for Reinforced Concrete American Concrete Institute
5.	**ACI 301-95**	Specifications for Structural Concrete American Concrete Institute
6.	**ASTM A615**	Specifications for Deformed and Plain Billet Steel Bars for Concrete Reinforcement
7.	**ASCE 7-10**	Minimum Design Loads for Buildings and other Structures
8.	**ASTM C150**	Specifications for Portland Cement
9.	**ASTM C33**	Specifications for Concrete Aggregate
10.	**UBC 1997**	Uniform Building Code of USA
11.	**FEMA P-259 (2012)**	Engineering Principles and Practices for Retrofitting Flood Prone Residential Structures
12.	**FEMA 274**	NEHRP Commentary on the guidelines for the seismic rehabilitation of buildings
13.	**ACI 562-13**	Code requirement for evaluation repair and rehabilitation concrete buildings and commentary

6.6.3 Loads

The designed structures must be capable to withstand and sustain the following when loads are applied collectively or separately:

- Dead Load
- Earth Pressure
- Hydrostatic Forces
- Hydrodynamic Forces
- Impact Forces
- Wind Forces
- Seismic Forces

A brief description of these loads is provided below:

a) Dead Loads

Dead Loads will be computed from the unit weight of materials as specified in the Building Code of Pakistan.

b) Hydrostatic Forces

Hydrostatic pressures and forces will be exerted on the flood retaining structures due to still and slow moving water. The hydrostatic forces will include lateral hydrostatic pressure, equivalent hydrostatic pressure due to flow velocities, combined water and soil pressure and vertical (buoyancy) water pressures.

c) Earth Pressures

Lateral earth pressures due to backfill will be computed by the classical theories taking into account the effect of submergence and seismicity of the area. In earth pressure computations the saturated earth pressure is to be considered alone as well as in combination with earth pressure due to saturated soil and standing water due to hydrostatic pressure.

d) Hydrodynamic Forces

The floods apply hydrodynamic forces on the flood retaining structures. The applied hydrodynamic loads are a function of flow velocity and structural geometry. Two types of hydrodynamic forces can act on a flood retaining structure depending upon the magnitude of flow velocity.

Low Velocity Hydrodynamic Forces	(flow velocity < 10ft/sec)
High Velocity Hydrodynamic Forces	(flow velocity > 10ft/sec).

e) Impact Forces

Any impact force and loads due to objects carried by flowing water will be considered in design. The impact force depends upon the objects or debris striking the structure and can be categorized as follows:

- Normal Impact Force
- Special Impact Force
- Extreme Impact Force

f) Wind Forces

In structural design, the wind loads acting upon flood retaining walls and other structures have been considered in accordance with ASCE 7-10.

g) Earthquake Forces

The earthquake forces will be considered in design using earthquake acceleration (pseudo-static) in accordance with BCP (Seismic Provisions-2007).

6.6.4 Loading Combinations

Load combinations must be considered as per AASHTO code and design should be carried out for the worst load combination.

6.6.5 Materials

The following materials are used in the construction of the flood retaining structures:

a) Concrete

Concrete class to be used for design of flood retaining structures is given in table 6.3.

Table 6.3: Concrete for structural design of flood retaining structures

Description	Use
Minimum compressive cylinder strength at 28 days equal to 28 MPa (4,000 psi)	RC Wall
Minimum compressive cylinder strength at 28 days equal to 10.5 MPa (1500 psi)	Plain/Blinding Concrete

b) Reinforcement Steel

The reinforcement steel to be used in reinforced concrete works shall conform to ASTM A615 Grade 60 with minimum yield strength of 414 MPa (60,000 psi) or AASHTO M-31 Grade 60.

6.6.6 Foundation Parameters

The Geotechnical design parameters such as bearing capacity, lateral earth pressures, foundation depth, Use of Sulphate Resisting Cement etc. will be used for carrying out structural design of flood retailing structures. These parameters shall be based on actual geotechnical investigations at site.

6.6.7 Stability Criteria

The following stability criteria will be followed for safe and stable design of flood retaining structures against sliding and overturning.

Factor of Safety for Sliding = 1.50

Factor of Safety for Overturning = 2.00

The resultant of all forces must fall within the middle third of the base width of structure.

6.7 Construction Methodology

The construction methods evidently vary with the type of flood retaining wall under construction. Following are the steps involved in construction of a stone masonry wall.

- **Spray Paint Line for Wall Dig/Excavate Ground**

  Begin building the wall by spray painting a line as to where the wall will be constructed. Then begin digging/ excavating the ground so that you will have enough room for 6” of compacted gravel plus ½ the first course block buried and 3” in front and behind the wall for room to work and adjust.

- **Add Base Material For Wall and Compact**

  After you done excavating the footer, start compacting 6” of gravel or pour a 1’ of concrete base (for walls higher than 4’) at the lowest spot. This is the most important step; you must have a solid base for your wall to last, and not collapse.

- **Laying First Course at lowest point, keeping it level**

  The first course of stone must be laid at the lowest point making sure that it is kept level. Build a section of wall at a time, beginning with the first course until you need to step up the wall. Keep building the first course for the entire wall, keeping everything level. Use a torpedo level to check to make sure each block is level, and a 4' level to make sure the last 2 or 3 blocks are level.

- **Adding Additional Wall Courses**

  After the first course is built, then stack additional blocks creating 2nd and 3rd, etc. courses of wall units until you reach desired height.

- **Backfilling and Drainage**

  After the wall is at desired height, you will need to backfill behind the wall. Begin by laying fabric against the ground to separate subsoil from the wall. The fabric lets water through, but prevents soil from mixing with gravel, which will be used as backfill. After the fabric is in place, lay a 4” drainage tile in the bottom of the pit between the fabric and the wall for drainage. This is essential so that no extra force and freeze/ thaw will happen behind the wall. When the drainage tile is completed and properly exiting from behind the wall; begin backfilling with suitable materials. Sufficient fill materials should be placed to allow for settlement.

- **Cutting Block for Curves, Snug Fit**

  Some stone will be needed to be cut to create curves and so forth. It is recommended using a dry brick saw that has a diamond tip blade. This makes the cutting process much easier, while saving time. You can use tapered block, half blocks, and corner blocks which allow you not to have to cut, and still get curves and desired look from your wall.

- **Coping to finish top of wall**

  The next step is to apply the coping or capstone. This is used to finish the top of the wall and give it a clean look. Coping is only 3” thick, and is held in place with adhesive chalking. The actual wall is held together by the lip design into the actual wall unit.

- **Placing of Backfill**

  The back fill should be placed in layers so that desired compaction can be achieved. The backfill should be of approved quality of granular nature. Proper drainage should be ensured during backfill. In case of MSE walls, reinforcement should be embedded in backfill.

For concrete flood retaining walls, following steps may be added;

- All necessary equipments and skilled labour for concrete should be arranged
- Steel form work should be erected at site and should be watertight
- Water stopper should be placed after each step of concrete
- Proper curing arrangements should be made

There will be some variations in construction steps depending upon the type of retaining wall.

6.8 Evaluation of Existing Flood Retaining Structures

Evaluation of existing structures shall be ascertained by detailed analytical evaluation as per above listed Design Criteria supplemented by load testing wherever considered necessary. In general, the structural evaluation for existing flood retaining structures will include following activities:

- The collection of design data and documentation must be done for the structures available with the Client. The missing but essential data must be identified and listed down. A checklist should be prepared to ensure that the subsequent site visits and field testing covers all missing parameters and details required to design an effective retrofitting solution.
- The Condition Survey must be carried out by visual inspection of existing structures by qualified engineers to observe and check for any signs of structural deterioration.
- Mapping and photography of all the deteriorated areas must be done.
- Preparation and supervision of testing programme must be done to indentify the areas and components of flood retaining structures for Non Destructive Testing (NDT) along with extraction of cores to evaluate present material strength.
- Preparation of Geotechnical Investigation requirements, supervision of field work and laboratory tests for determination of soil design parameters.
- Evaluation and review of all available data, design criteria, documents, investigation reports, surveys, test results to familiarize with the structural system and to establish various options for retrofitting.
- Design verification of structures for all anticipated static and dynamic loadings as per design criteria considered in the existing design.
- Selection of most prudent retrofitting option based on the above said evaluations and preparation of Assessment Report for submission to the Client.
- Development of Repair/Retrofit Designs

For repair and maintenance of flood retaining walls it is important that routine pre-flood and post flood inspections may be carried out to ascertain their ability to resist and withstand high floods and stability after flood recession. Prior to floods season, routine inspections must be carried out to observe signs for initiation of problems that might require repair and maintenance prior to floods. The routine inspectors must be fully familiar how to identify problems in such flood retaining structures. During flood season, monitoring will be required to observe the satisfactory performance of these structures. When floods have receded, a post flood inspection must be planned and carried out to assess the condition of the structures and to identify areas for maintenance and repair.

6.8.1 Routine Inspection and Maintenance

In typical routine inspection and maintenance of flood retaining walls, following points should be observed:

- clear accumulated debris from drainage channels and slope surfaces
- repair cracked or damaged drainage channels or pavement
- repair or replace cracked or damaged slope surfaces
- clear obstructions in weep holes and outlet drain pipes
- repair missing or deteriorated pointing in masonry walls
- remove any vegetation causing severe cracking of slope surfaces and drainage channels
- in case vegetation is provided, replant vegetation in areas where it has withered
- remove loose rock debris and undesirable vegetation from rock slopes or around boulders
- investigate and repair buried water carrying services in or adjacent to slopes or retaining walls where signs of possible leakage are observed
- Frequency and timing of routine maintenance inspections
- Routine Maintenance Inspections should be carried out at least once a year
- Any maintenance works, if required, should preferably be completed before the wet seasons
- Owners should arrange to inspect the drainage channels and clear any blockages after heavy rainstorm or a typhoon

6.8.2 Post Flood Evaluation of Existing Flood Retaining Walls

Immediately after the recession of flood water, the site should be visited by technical staff to look for condition of floodwalls and observed damages if any. Following types of failures and damages may come across during post flood evaluation of flood retaining walls:

- Overtopping of the wall by flood water
- Failure of wall due to sliding
- Failure of wall due to overturning
- Failure of wall due to settlement and decreased bearing capacity
- Structural failure of the wall. This also includes reinforcement failure embedded in the backfill in case of reinforced stabilized wall.
- Poor drainage from the drains of retaining wall. The backfill may lose strength and the active pressure on the wall may increase.

7. EVALUATION OF EXISTING FLOOD PROTECTION AND RIVER TRAINING STRUCTURES

The existing flood protection bunds and embankments throughout the country have been designed and constructed prior to extreme flood event of 2010. The flood event caused some major catastrophes and breaches resulting in loss of life and property. It is required that the existing flood protection bunds and embankments may be evaluated and assessed if they are in a position to sustain extreme events like 2010 flood.

The evaluation of existing embankments and bunds will require verificationof available freeboard, top width, side slopes, scourand erosion problems, slope protection works etc. to be in accordance with the latest design criteria and standards.

The primaryevaluation of existing flood bunds is carried out in field by visiting different flood bund sites, assessing their physical conditions and design conformity to the latest criteria and standards. The site inspection will reveal further level of monitoring, investigation and repair requirements.

7.1 Hydraulic Evaluation Criteria

7.1.1 Freeboard

A minimum freeboard of 6.0 ft has been recommended for designing of new flood bunds and embankments. Inassessment of existing bunds and embankments the highest design flood level must carefully be evaluated. The available freeboard between high flood level and bund crest level must be verified. It should be adequate to avoid overtopping of the flood bunds and in no case should it be less than 6.0 ft.

The existing flood protection bunds are to be raised to the desired level so that minimum required freeboard is available throughout their length. It must be kept in mind that the raising of embankments involves consideration of many factors and can be costly.

7.1.2 Top Width

The flood bunds must have adequate top widths to allow the inspection team to conveniently inspect thesebunds andprovide a passage for any pedestrian traffic.

The increase in top width of a bund will require the expansion of flood bunds either on landside or riverside. If the bund section is increased on landside, the presence of any houses, roads or cultivationland that may have developed over time must be considered. The increase of bund section on riverside will reduce the width of flood waterway and hence raise the flood levels. The situation is site specific and will vary for each location of flood bunds. It must be decided after assessmentof both options and then selecting the most viable alternative considering safety and economy.

A minimum top width of 25.0 ft must be available at the flood bunds and embankments. This top width will also add to stability of the embankment section and prevent slope failure during high floods or after flood recession. If the bund riverside slope is severely scoured and eroded during floods, the provided minimum top width will help prevent bund failure or breachingdue to the presence of a wider bund section. Extra time can be gained to construct a protection bund around that section.

7.1.3 Side Slopes

The existing flood bunds and embankments in Pakistan are constructed providing 1:2 landside slopeand 1:3 riverside slopes. In some areas a flatter riverside slope of 1:4 has also been used.

Embankment raising results in increase of embankment height and a mild slope serves well. When evaluating existing flood bunds and embankments, it must be ensured that a minimum of 1:2 landside slope and 1:3 riverside slopes have been provided.

7.1.4 Hydraulic Gradient

Almost all of the existing flood bunds and embankments have been constructed assuming an arbitrary hydraulic gradient of 1V:6H. During the evaluation and assessment of existing bunds, hydraulic gradient must be determined against highest flood levels.

Seep-W software must be used to model the bund section under evaluation. After section modeling and incorporation of necessary parameters, the phreatic line within the section can be obtained. The practice must be repeated considering different water levels at riverside of flood bund and determining the most critical scenario. The flood recession scenario after which water level at riverside is low but the bund is internally saturated with a hydraulic grade line having a higher level then river water level must essentially be modeled and analyzed. This is an imperative requirement for stability analysis of existing flood bunds and embankments which is carried out using Slope-W software.

7.1.5 Scour Protection

The scour protection of existing flood protection bunds and embankments must be carried out with care considering the highest recorded flood discharge and levels. The design criteria and formulas provided in section 5.2.10 can be used to estimate the scour depths and levels.An appropriate safety factor for calculating maximum scour (or factored scour depths) must be used. Scour protection in the form of riprap or stone pitching must be provided accordingly for these sections.

7.1.6 Stone Pitching

The provision of stone pitching at all flood protection bunds and embankments must be a compulsory practice to prevent riverside bank scour and erosion. The evaluation of existing flood bunds must ensure that proper stone pitching has been provided at the riverside which is able to resist the high flow velocities and wave wash.

The thickness of stone pitching layer and stone size provided at an existing flood bund section must be assessed. The required stone size and volume along with thickness of protection layer can be calculated using the design procedure and criteria provided in section 5.2.11. The comparison will show whether the provided stone pitching layer is adequate or not. The stone size provided must be such that high flow velocities and wave action may not cause its displacement and it stays intact. The shape of stone provided must also be observed whether it is angular or rounded. The highest recorded flood must be used at the flood bund section to determine the stone size, volume and thickness of pitching.

7.2 Geotechnical Evaluation

7.2.1 Geophysical Explorations

Geophysical explorations are an indirect method of obtaining generalized subsurface geological information by using special instruments to make certain physical measurements. Geophysical observations in themselves are not geologic facts, but are statistical and orderly measurements.

The geotechnical investigations for flood protection and river training works are done by visual inspection and material sampling by making test pits and bore holes to assess the subsurface conditions. The geophysical explorations and surveys provide non-destructive insight of the ground and subsurface conditions by complementing core drilling, test pits, boreholes or other direct methods of subsurface exploration, providing rapid evaluation and assessment of certain geological conditions.

During the inspection and evaluation of existing flood embankments and other river training works, the compaction status of existing bunds and presence of any voids or cavities is to be assessed. For this purpose, Seismic Refraction Method of Geophysical Survey can effectively be used to determine the overall compaction characteristics of existing bunds.

Seismic Refraction Method

The purpose of Seismic Refraction Survey is to determine the subsurface conditions, presence of voids or cavities within the body of embankment, thickness of any overburden material, depth to bedrock and strata compaction.

Seismic Refraction Survey is carried out at the top of flood protection embankment or river training works to determine the seismic velocities of embankment material and correlating it with the compaction characteristics of embankment based on the density of the embankment material.

A twenty-four channel Signal Enhancement Seismograph "TERRALOC MARK 6" by ABEM, Sweden is commonly used in field for recording the arrival time of seismic waves. In Seismic Refraction Survey, 12 to 24 geophones (or detectors) are planted on the ground surface, in accordance with the required information and available space, in a straight line at equal distance along the profile line. The seismic waves are generated at the source point by a Sledgehammer. The Reverse Shooting Method is adopted in which the seismic waves produced at other ends of detectors are spread and the travel time for these waves is recorded at each detector from both directions.

The execution of Seismic Refraction Survey involves following tasks:

- Seismic Refraction Survey at different locations in the project area is carried out to ascertain subsurface conditions and strata compaction
- The coordinates at start and end point of each profile are taken with the help of a hand held GPS device
- The interpretation of field data is done in form of thickness and seismic wave velocity of subsurface layers
- On the basis of the interpretation of Seismic Refraction Survey, the thickness of overburden material and strata compaction is determined
- The results are presented in the form of subsurface sections in a report

Actual ground information from boreholes, test pits, site and laboratory testing are essentially required to prevent any misapprehensions. Since these results are not subject to direct visual verification, geophysical exploration requires boreholes or other direct geological exploration for references and control of measurements.

7.3 Field Inspections

To evaluate the capacity and condition of existing structures, detailed inspection of flood protection structures should be carried out and potential problems must be identified for further action. The inspection program cancomprise of different steps. These may include:

7.3.1 Pre-Inspection Activities

Prior to field inspection of flood protection structures and evaluation of their existing condition, following pre-inspection activities must be ensured.

i) Planning

A review of notes from previous inspections, photographs and "as-constructed" drawings, if available, should precede the actual field inspection. This allows comparison of the present condition to the "as-constructed", and also at the time of the previous inspection. Pre-arranging meeting times with key land owners if access is required may save time, and these contacts can prove to be a valuable source of information.

ii) Field Equipment Requirements

Equipment requirements vary depending on the type of inspection to be carried out and the expected conditionat site. Equipment requirements for inspections are provided in table 7.1:

Table7.1: Field Inspection Equipment

Sr.#	Type	Purpose
1.	Inclinometer	Measuring the degree of slope
2.	Measuring tapes	Measuring dimensions of features or abnormalities
3.	Chain	Measuring longer distances
4.	Rock Hammer	Sounding concrete or rock to check quality and checking for pipe corrosion
5.	Shovel	Minor clearing, taking soil samples
6.	Torch Light (flashlights)	For clear observations at dark areas
7.	Probe (steel rod)	Probing wet, soft areas, sinkholes, and voids
8.	Bucket and timer	Measuring seepage and other flow rates
9.	Sounding lines or tapes	Measuring water depths
10.	Camera	Taking photographic records
11.	Water sample containers	Obtaining water samples
12.	Sample bags	Obtaining soil or rock samples
13.	Level, rod and tripod	Obtaining accurate elevations and heights
14.	Clipboard and record keeping material	Recording inspection observations
15.	G.P.S. Device	Measuring position of the object

iii) Personal Safety

For safety reasons it is always a good idea to inform appropriate co-workers, prior to departure, of the basic five "**W**'s"

- **W**ho is going and with whom are they meeting?
- **W**hat is going to be inspected?
- **W**hen is the departure, arrival and estimated time of return?
- **W**here is the location of the inspection?
- **W**hy is the inspection being carried out – reported problems or routine inspection?

A personal safety plan should be in place for all personnel inspecting any or all components of a flood protection system. This plan should include:

- Supplying and using all personal safety equipment required by the Workers.
- Providing appropriate communication devices
- Ensuring that a check-in procedure is in place and is being used
- Supplying and updating all necessary vehicle safety equipment

Ideally, inspection personnel should work in teams of two when in the field, particularly when doing high water or post-earthquake inspections, or at night in severe weather conditions. All safety requirements must be organized according to the applicable requirements.

7.3.2 Inspections Scheduling

Inspections are critical for identifying problems and planning corrective actions. A structure is an active Component, constructed of materials that are subject to various stresses that may lead to deterioration and risk of failure. An inspection program to observe, record and report on the flood protection system should start during construction and continue on a regular basis throughout its operation. Effective management of a structure system involves inspections at several times of the year, coinciding with water events. The program should be maintained in the Operation and Maintenance Manual.

7.3.3 Regular Inspections

These are routine inspections and must be carried out weekly. The staff appointed for carrying out these inspections must be competent enough to manage such inspections and to be able to identify any problems. The advantage of regular inspections is that the problem is identified at a very early stage and is easy to rectify without any severe damage or loss. Regularinspections help identify problems timely and thus save cost for repair and rehabilitation.

7.3.4 Annual Inspections

At least once a year the entire flood protection system should be inspected in detail by the Competent Authority for routine maintenance purposes. This inspection should be scheduled prior to the high flow season, and early enough to allow adequate time for any required work to be completed prior to possible flood events. The inspection should note and record any conditions that might affect the performance of the flood protection system. In areas prone to floods, the inspection should be completed well before the start of monsoon season so that sufficient time for any type of remedial work can be made available.

7.3.5 Low Water Inspections

This inspection should be undertaken each year when the river water levels reach their lowest annual levels. During this period, areas of the structure fill and bank protection that are normally under water, are exposed making inspection much easier.

7.3.6 Special Inspections

Special inspections may be needed at other times of the year to monitor and react to particular situations such as storms, reports of vandalism, or construction activity on or near the structure. Construction activity should be observed to confirm that accepted engineering practices are being followed (compaction, placement of cut-off walls, etc.). A record should be kept of what was done, by whom, and the cost. This information may be needed to provide evidence should structural failures occur in the future.

7.3.7 High Water Patrol Inspections

Patrol inspections should be carried out during high water events to monitor the performance of the flood control works and emphasize early detection of problems resulting from increased hydraulic pressure (seepage, boils, etc.), increased potential for erosion of materials, and increased chance of reduced freeboard (observed from gauge readings). During high water events, local water level gauges should be monitored regularly and the readings recorded to assess changing conditions and for future reference. Structure patrol frequency should increase as flow and/or water levels approach critical conditions, and should be continuous while the level is within about 1.0 m of the structure crest. The patrol crews are to observe, mark, record and submit reports to the competent Authority for any conditions or occurrences that could signal a weakening or malfunction of any component of the flood control system.

7.3.8 Post-Flood Inspections and Evaluations

Inspection of the protective works should be undertaken after flood events. A complete high water profile along the structure should be obtained after significant flow events to assess the structure crest level and the amount of freeboard. Changes in the river channel should be noted not only for the reach adjacent to the flood protection works but for areas both upstream and downstream any new locations of log jams, streambed aggradations (gravel accumulation) and degradation (gravel scour) areas, weakened or damaged areas, and the condition of all appurtenant works.

7.3.9 Post-Earthquake Inspections

This type of inspection should be integrated with local emergency plans. A rapid initial overall assessment of the entire flood protection system should be undertaken to determine what level of protection it still offers and if there is an immediate danger of secondary damage (i.e. next high tide could overtop, causing widespread flooding). When the immediate threat is determined and addressed, a detailed inspection should be undertaken of the entire structure, erosion protection and appurtenant works. The functionality of individual structure should be checked in detail as follow up. A detailed inspection and evaluation of the system by as suitably qualified professional engineer may be advisable because damage may not be readily visible. Identification of areas prone to earthquake damage in advance will aid assessment after an event.

7.3.10 Inspection Methods

Inspections should emphasize complete examination of all aspects of the works.

i) Inspection of Structure Fill

The usual method of inspecting the structure fill is to walk along both crest edges (shoulders) and the landside toe, in order to see the entire surface area clearly. Details can usually be seen for a distance of 3 to 10 metres depending on the roughness of the surface, vegetation, or other surface conditions. The riverside toe is inspected in the same manner as the landside toe for set-back structures, however for riverside structures without erosion protection, walking along the slope just above the current water level can prove challenging and may be more efficiently done from a boat.

ii) Inspection of Erosion Protection Works

The structure of most types of erosion protection works makes walking over them difficult and somewhat hazardous. It may be easier to identify problems from across the river using a boat for inspections when and where closer observations are necessary.

7.3.11 Flood Protection Inspection Report

A written report on the results of the annual inspection should be prepared on a copy of the Flood Protection Inspection Report, as given below. Field notes may be prepared using the form or one similar which clearly identifies the main failure groups (Structure, Erosion, Protection Works), their condition and the work required to correct any noted deficiencies.

7.4 Field Observation and Monitoring

Flood bunds and embankments require a constant observation before start of monsoon season by local people and concerned authorities to ensure that these are free of cracks, burrow holes, piping, sand boiling, creep flow, leakage through cracks, poor drainage of seepage water etc. Any such observations made must immediately be reported to the stationed officer so that repairs and rehabilitation can be carried out.

Post monsoon and past flood inspections should be carried out to ascertain the health of structure and repaired works. Regular patrolling of guide, marginal and flood protection bunds should be done during floods to monitor the behavior of structure and to take necessary protection measures.

The inspection of flood bunds and embankments must becarried out with care in complete detail to gather maximum information for their existing condition. The observations made must be comprehensively recorded in the field report to provide the reader with maximuminformationand site awareness for the inspected flood bunds.

The site inspection of bunds and embankments can be divided into following three areas to facilitate visual field assessment and observation.

i) At Crest

The crest or embankment top provides primary access for inspection and maintenance. Major observations to be made over embankment crests are surface cracking, vegetative growth, animal burrow holes or cave-ins, rutting of embankment top, settlement, deterioration of top surface, human interaction and seepage.

Human activity such as pedestrian movement and light traffic at the embankment top must be observed and recorded in field observations.

ii) At Riverside Slope

The inspection of riverside or upstream slopes must be made for observations regarding vegetative growth, burrow holes or cave-ins, surface cracking, formation of rainwater gullies, seepage, sediment or debris accumulation if any, scour and erosion, slope protection works and removed stones at riverside slope.

The human activity is quite rare at riverside slopes due to its direct contact with river flows most of the year. Inspections made during low flow seasons must incorporate any such activity or observations in field report.

The inspections made during high flood season must also include visual observations for any generated waves and the wave action on riverside slopes. High water levels during floods can be visually experienced and visual observations recorded in field report. The post flood inspection must incorporate visual inspections for flood water marks present at the embankment slope. If measurement is made between water mark and embankment top, it provides a site assessment for freeboard encroachment if any.

iii) At Landside Slope

The inspection of landside or downstream slopes must include observations for vegetation, animal burrows, surface cracking, rainwater gully formation, settlement, sliding and erosion.

Some of the most common features that a field inspection team must be vigilant in observing during field inspection are provided below:

Vegetation

All sorts of vegetative growth over flood bunds must be observed and included in the field report. Tree and plant roots penetrate in the embankment weakening its structurefrom inside. During wind storms or floods, if tree is detached from the ground, the flood bunds are left exposed to risk of failure. Also, existence of organic matter with the embankment results in deterioration with time.

Burrow Holes

Animal burrow holes are present in majority of existing bunds which are threatening to the embankment condition especially during floods. It is necessary that such holes may be identified during field visits and closed or filled before flood season.

When bund material has high sand content the holes will collapse and close. But when it consists of high clay content,the holes will not close and water will flow through hollow portions and cavities between clay clods. It is a good practice to observe embankment material at site to be sandy or clayey where burrow holes or cave-ins are identified.

Surface Cracks

The cracking of bund surface at top and side slopes can be due to a number of factors. It can be due to drying out of embankment material, settlement or initiation of sliding. Embankment with high clay content used at top layers will show signs of cracking when dry.

These cracks serve as leakage pathsthat must becritically observed and assessed. The width of cracks must also be inspected. A surface crack few inches wide can be almost one foot deep. A scale or thin object can be inserted in wide cracks to get an idea of the crack depth. The observations must be recorded in field report along with photographs.

Rainwater Gullies

The rainwater over the embankment top is to be drained towards the landside slope. This slope is generally unlined and action of drained and falling rainwater results in the formation of rainwater gulliesand rain cuts. These gullies and cuts weakenthe flood bunds and must be identified in field report during field inspection. Any rain cuts formed at the embankment top and riverside slopes must also be reported and corrected before starting of flood season.

Seepage

It is impossible to construct flood protection bunds that are fully watertight. All flood bunds have certain amount of leakage or under-seepage through the embankment material, especially when holding back a flood.

The seepage can take place through both embankment body and under it, often caused by animal burrow holes. A severe seepage problem can cause piping failure and local collapse of the embankment section. The inspection team must be vigilant in observing such locations. The signs for any seepage activity through embankment sections must especially be observed where holes and cracks are present.

Also, seepage problems can be identified at places where the flood bunds come across some structure such as a culvert or bridge. Seepage may take place through abutments for which careful attention must be paid. If seepage is excessive through the embankment body, it will show a damp patch on the landside of the flood bund.

Settlement

Allflood bunds and embankments undergo settlement with time. The settlement may or may not be uniform depending upon the used construction material and degree of compactness. During field visits, the bund sections must be inspected for any excessive or unequal settlements that can be visually identified at crest and side slopes. The embankment settlement at riverside slope will result in damage to the slope protection works. The settlement can also be a result of sliding of some portion of the embankment. All observations for settlements must be critically observed and properly recorded in the field report.

Weathering and Deterioration

The weathering and deterioration of embankment top takes place due to the pedestrian or light traffic that passes over it. Deterioration and weathering of unlined side slopes is due to weather effects, climatic changes, blowing winds or human activity. Recording for such observations must be made in the field report.

It must be understood that the granular oils such as sand, are less resistant to erosion and weathering as compared to cohesive soils such as clay. The removal or weathering away of primary layer provided at embankment top leaves the inner granular layers exposed for erosion and weathering. These observations must not be neglected by the inspection team and recorded in field reports with photographic evidence.

Sliding

The field inspectionteam must observe the flood bund sectionsagainst any signs of sliding. The bulking or bulging out of embankment material or pitching provided along its slope must be looked for. These are also the indications for embankment sliding.In case of sliding, the stone protection is also damaged and at times slides down to the embankment toe. Care must be taken not to ignore signs of any future slides at bunds which must be reported immediately to irrigation department officials so that further damage can be prevented and repairs may begin.

Scour and Erosion

Erosion and scour occurs at flood bunds due to action of river flows and can pose a serious threat if severe. A close inspection of the embankment slope at riverside must be made. During high flow season, riverside slopes are not properly visible and erosion problems become hidden.Evidence for scour at river bed,especially close to embankment toe,and riverside slope must be observed after flood recession. The scour or erosion at toe can result in collapse of flood bund which is not acceptable at any stage. All observations for scour must be well recorded. Photographs of scoured areas must be taken to so that exact visual observations are also preserved in field records. An annual post flood inspection is recommended for underwater portions of the embankment and river bed to assess the scour and erosion in vicinity of the flood bunds and embankments.

Protection Works

Slope protection provided at the riverside slope of flood bunds and embankments can be damaged or washed away due to severe wave action and high flood flows. The inspections for slope protection works must ascertain if the protection stone is intact or detached from its position.

An annual inspection of underwater protection works should be carried out after flood season to asses scour and launching of apron in the vicinity of the structure. Washing away or displacement of stone must be observed and recorded. The inspection team must proceed in flow direction and observe signs of any dislodged stone pitching or apron material away from the displacement location. There is also a possibility that high flow currents would wash away the detached material and no signs of accumulation may be observed by the inspection team.

The lack of proper maintenance and replacement or removal of protection works, such as protection stones from side slopes, in flood risk zones can result in embankment failure and breaches.The inspection team must pay special attention to the conditioning of protection works in high risk flood zones.

Stone Removal

At times stone provided for riprap or slope protection is loosened or detached from its base but not washed away. This is normally due to poor binding mixture used during construction for laying slope protection or apron. The field observations must be made and recorded in the field report so that the stone for slope protection may be repaired.

Human Interaction and Activity

Human interaction and activity is always present on bunds mostly at crest in form pedestrian or any light traffic.At landside of flood bunds the activity is mostly considerable asresidences are made and land is used for cultivation by locals.

The inspection of bunds and embankments will vary from one site to another. Each inspected location must be given individual importance. Apart from the above mentioned observatory checks, other observations and field findings during inspection must be included in the field report. Photographs must be taken during field inspection and included in field report for all inspected sections. The photographs must clearly show the observed problems and findings so that concerned Department officials and technical staff can be well aware ofthe nature of situation and act accordingly.

The problems observed and identified during field inspection have a certain cause of occurrence. The inspection team must be familiar with the root cause of observed problems and effectively address the safety and stability concerns related to these field observations.The Engineer and field inspection team must be able to clearly understand the extent of observed problems and gather maximum information to sort out remedies.

A set of guidelines for the identification of problems in flood protection structures and erosion protection worksalong with their cause and concern are provided in table7.2.

In order to facilitate the field inspection of flood bunds and other flood protection and river training structures, a checklist is providedin table7.3for recording observations during field inspection of existing flood protection and river training structures.

The geometric dimensions are to be measured and recorded at different embankment sections so that the existing embankment dimensions can be recorded and checked if they meet the design requirements. The table7.4liststhe geometric dimensions and levels to be recorded.

Table7.2: Guidelines for Identification of Problems in Flood Protection Structures and Erosion Protection Works

Sr.#	Problem	Observation	Cause	Concern
1.	Overtopping or Loss of Freeboard	- High water surface profile is within the freeboard allowance - Evidence of slumps, sinkholes, slides	- Aggradations of the channel bed - Channel blockages; logs, etc. - Settlement of structure	- Reduced freeboard creating a potential for overtopping
2.	Settlement	- Uneven surface of the crest or slopes - Depressions with gently sloping bowl-like sides	- Internal erosion of the embankment material - Prolonged erosion from wind or water - Poor construction practices, poorly compacted fill, organic material line fill - Foundation consolidation	- Creates areas of structural weakness - Loss of freeboard from settling can create the potential for overtopping
3.	Sinkholes	- Hole in the structure surface - Depression with steep bucket-like sides	- Animal burrows - Internal erosion from seepage piping - Foundation problems such as rotting stumps or other wood debris	- Weakens the structure fill by decreasing the length of the seepage path - Provides an entrance point for surface water - Can pose a danger to vehicular and pedestrian traffic - May signal collapse and/or instability
4.	Seepage/Piping (Wet Areas)	- Turbid (dirty) or cloudy seepage water - Water or wet areas near the toe or on structure slope - Localized or lush vegetation on structure slopes or adjacent to the structure - Increase in seepage flow rates different from past patterns	- Excessive flow of water through the structure fill or through the foundation material - Surface water entering through cracks, sinkholes, animal burrows, along the outside surface of conduit	- May cause slope instability which can lead to failure - Turbid (dirty) seepage water is an indication that piping may be occurring and may result in a piping failure of the foundation and ultimately the embankment
5.	Boils	- Water upwelling on landside of structure, near toe or further away - Upwelling may form cone-shaped 'volcanoes'	- A weak layer of sand or gravel in the foundation material is being charged by hydraulic pressure produced during high water conditions - A concentrated seepage path or pipe has developed through the foundation	- May be an early sign of piping
6.	Desiccation/ Drying Crack	- Random, honeycomb pattern	- Embankment material expands and	- Provides an entrance point for surface water

Sr.#	Problem	Observation	Cause	Concern
		of cracks along the embankment	contracts with alternating wet and dry weather - Embankment fill with high fines content and/or inadequate compaction	which can saturate the crest material - May affect durability of the crest in wet weather
7.	Transverse Cracking	- Cracks extend across the crest perpendicular to the protection work length	- Uneven movement between two adjacent segments of the embankment - Instability of the embankment or foundation material - Differential settlement	- Provides an entry point for surface water - Creates an area of structural weakness which could result in further movements or failure - May create a seepage path and/or a potential piping failure
8.	Longitudinal Cracking	- Cracks extend roughly parallel to the length of structure	- Uneven settlement within the foundation or embankment - Initial stage of a slope failure or embankment slide	- Possible instability - Can lead to future movements or failure (breach) - Provides an entry point for surface water which can promote movement - Often reduces the effective crest width
9.	Slope Instability (earthen)	- Displaced material on structure slope - Bulges along the embankment slope or toe - Area above the bulge shows cracking or scarps - Excessive moisture or softness upon probing the bulge - Arc-shaped crack (beginning of a slide) - Evidence of settlement - Slides (shallow or deep-seated)	- Tree logs and wave erosion creating vertical slopes - Steep slopes left unsupported by erosion - Embankment fill becomes saturated during high water followed by rapid drop in water levels - Slope too steep for type of embankment material to allow freed raining	- Direct threat to the integrity of the structure - possible breaching - Provides an entry point for surface water which can promote movement - Often reduces the effective crest width
10.	Stone Work (Spur, Stud, Stone Pitching and Launching of Apron)	- damaged apron - damaged stone pitching at toe near apron - settlement of stone pitching at slope	- Excessive scour - Wave action - Poor earthwork or stone work	- Stone apron and stone pitching may collapse - Excessive seepage and piping
11.	Surface Erosion and Rutting	- Evidence of material loss from structure surface - Wheel tracks, animal tracks - Scarring of structure surface - Pooling of water on crest	- Livestock or human traffic - Surface runoff over erodible material -	- Encourages further erosion - Can decrease cross-sectional width and weaken the embankment

Sr.#	Problem	Observation	Cause	Concern
12.	Unauthorized Construction or Activities	- Embankment material disturbed or removed - New ponds, holes or foundations dug close to the structure	- Uninformed or illegal construction practices	- Otherwise competent system can be compromised by a single unauthorized action - Can block or hamper access - Often hides defects such as poorly compacted fill around a newly placed or repaired conduit increasing the chance of seepage - Can encourage boils or slumping and reduce top width - Can encourage boils and failure from piping
13.	Uncontrolled Vegetation Growth	- Vegetation obscures ability to detect cracks, seepage or other problems	- Lack of maintenance	- Root systems can provide seepage conduits - Rotting root systems weaken the embankment - May prevent emergency access - Provides a habitat for unwanted burrowing animals - Wind throw or uprooting of trees can create holes and weakness
14.	Animals/ Rodent Activity	- Rodent holes, burrows and tunnels - Animal trails - Fallen trees (beaver activity)	- Burrowing animals including bank beavers	- Can weaken the embankment - cause sinkholes and piping - Potential vehicle access restrictions if unchecked
15.	Toe Scour	- Loss of earthwork or stone work from structure slope - Loss of stone from apron - Eddying at the structure to	- Inadequate apron/material size - Shift in flow impact angle due to formation of log jams, shifting river bed materials or man-made obstacles	- Loss of erosion protection material leaving the embankment materials vulnerable to erosion and possible breaching
16.	Changing River Flow Patterns	- Dramatically altered flow pattern of the river - Areas of impingement on the protection work altered - Channel obstructions in the vicinity	- Landslides - Log jams - Gravel accumulations - Man-made obstructions - Natural meander progression and/or formation of cut-offs	- Additional erosive forces applied against existing bank protection works increasing the chance of its failure - Direct flow against sections of the flood protection system not previously subjected to erosion. If not already armoured, could lead to

Sr.#	Problem	Observation	Cause	Concern
				rapid loss of embankment fill - Outflanking of existing works at upstream end
17.	Bed Degradation	- River channel scouring adjacent and roughly parallel to the erosion protection - Damaged apron and damaged toe of slope	- Changing river currents and high water levels - Deepening of the riverbed in the reach near the structure - Insufficient design or construction of toe protection	- The erosion protection material is vulnerable to undermining and collapse exposing the earthen portion of bank
18.	Outflanking	- River erosion upstream of hard point or key trench	- Erosion protection not extending far enough upstream - Erosion protection not extended to a hard point at the upstream end - Weak upstream key (poor design) - Sudden change in river flow pattern	- Rapid loss of erosion protection material leaving the embankment fill vulnerable to erosion - Exposure of unprotected fill to erosive forces
19.	Overbank Erosion	- Reduced riverbank area - Progressive erosion	- Reduced distance from the structure fill to the river channel due to changing river currents - Natural meander progression - Lack of erosion protection on set-back area	- Threat to embankment stability - Undermining of embankment
20.	Degrading (Weathering)	- Disintegration of stonework - Cracks, spalling, crumbling of stone material - Hollow sound on rock hammer testing	- Chemical or mechanical deterioration of the erosion protection material often accelerated by wave action.	- Widespread weakening of erosion protection material leaving the embankment fill more susceptible to erosion -
21.	Uncontrolled Vegetative Growth	- Vegetation obscuring inspection - Large vegetation and trees on fill - Tree uprooting on riprap - Tree blow down across structure	- Lack of regular vegetation management - Poor maintenance procedures	- Can obscure serious problems which may exist - Tall trees with large root systems can displace large amounts of erosion protection material when forced over by wind, or high water

Table7.3: Checklist for Inspection of Existing Flood Protection and River Training Works

Area	Bund Condition	Satisfactory/ Unsatisfactory	Necessary Action		
			Monitor	Investigate	Repair
Crest	Vegetation				
	Burrow holes				
	Surface cracks				
	Rutting				
	Weathering/deterioration				
	Seepage				
	Rainwater gullies and rain cuts				
	Settlement				
	Human interaction				
Riverside/ Upstream Slope	Vegetation				
	Burrow holes				
	Surface cracks				
	Rainwater gullies and rain cuts				
	Seepage				
	Settlement				
	Sliding				
	Scour and Erosion				
	Protection Works				
	Stone removal				
Landside/Downstream Slope	Vegetation				
	Burrow holes				
	Surface cracks				
	Rainwater gullies and rain cuts				
	Seepage				
	Settlement				
	Sliding				
	Erosion				
	Protection Works (if any)				
	Weathering/deterioration				
	Human interaction				

Table7.4: Geometric MeasurementTable for Flood Bund Sections

Sr. #	Measured Dimensionsand Levels			Comments
1.	Top width		ft	
2.	Riverside slope length		ft	
3.	Landside slope length		ft	
4.	Embankment height		ft	
5.	Top level		ft	
6.	Bed level		ft	
7.	Back berm top level (if provided)		ft	
8.	Back berm width (if provided)		ft	
9.	Sketch of Inspected Flood Bund Section			

SITE INSPECTION REPORT – FLOOD PROECTION STRUCTURES

1.	**Project**					
2.	**Location**					
3.	**Brief Project Description**			Approval Date ____________ Project Start Date ____________ Completion Date ____________ Cost of Scheme ____________		
4.	**Inspection Team Members**			**5.**	**Inspection Date**	
6.	**Name of Inspected Structure**					
7.	**Design Parameters**	**Measurement**	**Units**	**Design Parameters**	**Measurement**	**Units**
	Length			Lining/Pitching Type		
	Top Width			Stone Apron Length		
	Height			U/S Side Slope		
	Freeboard			D/S Side Slope		
	Top Elevation			other: ____________		
	Embankment Toe Elv.			________________		

8. Site Observations

A) Structure (Access vegetation growth, gravel surface, height, slopes, erosion, animal burrows, seepage, trash, berms etc.)

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B) Bank Protection (loss of stone protection, settlement, slumping) ____________________

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Signature		**Page (1 of 2)**

SITE INSPECTION REPORT – FLOOD PROECTION STRUCTURES

C)	**Works Required**
9.	**Additional Information (attachments, sketch, site photographs, etc.)**
10.	**Recommendations**

Signature		**Page (2 of 2)**

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