LIST OF CONTENTS

LIST OF TABLES

1 DESIGN STANDARDS

1.1 DESIGN CRITERIA FOR STRUCTURES

1.1.1 Definition of Structures

Culverts: Cross drainage structures having length ≤ 6 m shall be classified as culverts.

Major and minor bridges: Bridges having length upto 60m shall be classified as minor bridges and bridges having length greater than 60m shall be classified as major bridges

1.1.2 Design Standards-Structures

The design shall cover all aspects of preliminary design pertaining to various parts of Bridges / Grade separators / ROBs, foundations, protective works etc. The design shall generally be based on relevant IRC codes of practice, MoRT&H circulars. However, where the IRC codes are not applicable or silent, appropriate BIS or other international Codes of Practice, such as, British / American / Australian Codes based on sound engineering practice shall be used.

This section below outlines the standards to be adopted for the design of the structures which include Flyovers and Interchanges, Major Bridges, Minor Bridges, SVUPs and Culverts.

The IRC codes/Standards/Act, MoRT&H Publications, IS & BIS codes shall be followed in the project. Design of all proposed structures shall be done in accordance with the provisions of the following Latest IRC Codes:

List of IRC Codes

Manual	Description
IRC: 5	Section I- General Features of Design (Eighth Revision)
IRC: 6	Section II- Loads and Stresses
IRC: 22	Section IV-Composite construction for Road Bridges (Second Revision)
IRC: 24	Section V-Steel Road Bridges (Second Revision)
IRC: 78	Section VII- Foundations and Substructure (Second Revision)
IRC:83(Part-II)	Section IX- Bearings, Part II: Elastomeric Bearings
IRC: 83(Part-III)	Section IX-Bearings, Part III: POT, POT-CUM-PTFE, Pin and Metallic Guide Bearings
IRC: 87	Guidelines for the Design and Erection of False work for Road Bridges
IRC: 89	Guidelines for Design and Construction of River Training and Control Works for Road Bridges (First Revision)

Manual	Description
IRC:112	Code of Practice for Concrete Road Bridges
IRC: SP13	Guidelines for Design of Small Bridges and Culverts
IRC: SP:40	Techniques for strengthening & rehabilitation of bridges.
IRC: SP:64	Guidelines for the Analysis and Design of Cast-in-Place Voided Slab Superstructure
IRC: SP:66	Guidelines for Design of Continuous Bridges
IRC: SP:69	Guidelines & Specifications for Expansion Joints
IRC: SP:70	Guidelines for the Use of High Performance Concrete in Bridges.
IS:14593	Design and Construction of Bored cast in situ Piles founded on Rocks.
IS: 2062	Hot rolled medium and high tensile structural steel specifications.
IS:14268	Uncoated stress relieved low relaxation seven Ply strand for pre stressed concrete specifications
IS-2911 (Parts/Section-I,II,III,IV)	For Pile foundations
IRC:SP:114	Seismic design for Road Bridges

1.1.3 Special Design Requirements

The complete structure shall be designed to be safe against collapse and to maintain at all times an acceptable serviceability level. These shall be also designed to be durable to withstand the deteriorating effects of climate and environment.

All new bridges shall have independent superstructures for each direction of travel. Choice of single or independent structure for culverts shall be decided based site condition. Width of median in structural portion will be maintained same as that in the approaches.

In cases where median is kept open to sky, suitable provision will be made for retaining the earth likely to spill from median portion of immediate embankment.

All new bridges will be provided for carriageway width as per 2 Lane Manual IRC: SP:-73.
Bearing of new bridges shall be easily accessible for inspection and maintenance.

Reinforced Earth/R.C.C Retaining wall type can be provided for high fill/embankment with aesthetically pleasing appearance. Design life of reinforcing elements for earth retaining structures shall be 100 years minimum. Structure with viaduct shall be provided for ensuring unhindered local cross movement of pedestrians and local vehicular traffic.

1.1.4 Design Basis

The design would be carried out using the limit state design philosophy satisfying the requirements of IRC-112 2011. The structure would be designed to meet both the ultimate and serviceability requirements of the code.

Ultimate limit state: This cover static equilibrium and failure of structural element or structure as a whole when acted upon by ultimate design load.

Serviceability limit state: This deals with the condition of structure subjected to serviceability design loads. These conditions include level of internal stress, fatigue failure, deflection, cracking and discomfort by vibrations.

Load Combination shall be adopted as per table B.1 to B.4 of Annex-B of IRC: 6-2017 as given below:

- Table B.1 for Verification of Equilibrium.
- Table B.2 for Verification of Structural Strength
- Table B.3 for Verification of Serviceability
- Table B.4 for Base Pressure and Design of Foundation

At present the combination of loads shown in Table B.4 shall be used for structural design of foundation only. For checking the base pressure under foundation, load combination given in IRC: 78-2014 shall be used. Table B.4 shall be used for checking of base pressure under foundation only when relevant material safety factor and resistance factor are introduced in IRC: 78-2014.

Table 1-1: Design Parameters for Structures

S. No.	Design Figure	Standard
1	**Service life (Years)**	
	Foundations	100
	Piers	100
	Deck	100
	Bearings i. for MJBR, VUPs and Rail road structures ii. for other structures	50 25
	Expansion Joints	10
	Parapets(Concrete)	100
	Parapets(Metals)	20
	Exposure Condition:	
	As the general environment condition is Moderate	
2	Minimum clear cover to reinforcement is given below. According to Exposure condition clear cover is provided for structural components as per IRC: 112-2011.	

S. No.	Design Figure		Standard
	i	Superstructures	50mm
	ii	Crash Barrier	50mm
	iii	Substructures	50mm
	iv	Pre-stressing cable duct	75/90mm
	v	Pre-cast elements	50mm
	vi	Foundations	75mm
	vii	Earth Face of Abutment, return wall, retaining wall, box side wall	75mm
	viii	Non Earth Face	50mm
3	**Grade of Steel**		
	i	HYSD bars	Fe500D (Section 1600 of Specifications: High yield strength deformed bars Fe500 conforming to IS: 1786)
	ii	Structural Steel	Fe490(IS 2062)
4	**Grade of Concrete**		
	I	Precast RCC Girder with RCC Deck	M40
	ii	RCC Box type structure-MNB	M40
	iii	Pier and Pier Cap	M40
	iv	Bearing Pedestal	M40
	v	RCC Abutment, Abutment cap, Return Wall, Dirt wall.	M40
	vi	Open foundation, Pile and Pile cap	M40
	vii	Crash Barrier	M40
	viii	Approach slab	M40
	ix	Box Culverts	M40
	x	Leveling course	M15
	xi	Head Wall	M20
	xii	Parapet wall	M30

S. No.	Design Figure		Standard
5	**Dead load-(unit wt.)**		
	i	Pre-stressed Concrete	2.5 T/m³
	ii	Reinforced Concrete (RCC)	2.5 T/m³
	iii	Plain Cement Concrete (PCC)	2.5 T/m³
	iv	Steel	7.854 T/m³
	v	Wearing Coat	2.2 T/m³
6	**Live Load**		
	i	Footpath	400 Kg/m² (Rural area) 500 Kg/m²(Urban area)
	ii	c/w 5.3m to 9.6m	One lane of class 70R or Two lane of Class A
	iii	c/w 9.6m to 13.1 m	One lane of class 70R for every two lanes with one lane of class A on the remaining lane or 3 lanes of class A
	** Live Load shall be considered at inner edge of the carriageway for stage 2 constructions and Girder shall be capable to take care the same.		
7	**Impact factor**		
	Concrete Bridges		
	i	for Class A	4.5/(6+L)
	ii	for Class 70 RT and 70RW	**Upto-9m**
			For Tracked-25% for span up to 5m and linear reducing to 10% for span up to 9m.
			For Wheeled-25% for span up to 9m
			More than 9 m-
			For Tracked-10% for span between 9m to 40m.As per curve for span more than 40m.
			For wheeled-25% for span up to 12m and as per curve for span more than 12m
	Steel Bridges		
	i	for Class A	9/(13.5+L)
	ii	for Class 70 RT	**Up to -9m**

S. No.	Design Figure		Standard
		& Class 70 R W	For Tracked-25% for span up to 5m and linear reducing to 10% for span up to 9m.
			For wheeled-25% for span up to 9m
			More than 9m-
			For Tracked-10% for all spans. For wheeled-25% for span up to 23m and as per curve for span more than 23m.
8	**Wind Load**		
	i	As per basic wind speed 47m/s as per IRC:6-2017, clause 209.	
9	**Horizontal Forces due to water current**		
	i	Case-I	Parallel to pier
	ii	Case-II	At inclination of (20±θ) to the pier
10	**Longitudinal forces**		
	i	Case-I	In case of single lane and two lane 20% of first train load plus 10% of load of succeeding train or part thereof
	ii	Case-II	In case of bridges with more than two lane braking force for two lane plus 5 % of the loads on the lanes in excess of two
11	**Buoyancy**		
	i	100 % buoyancy for stability check	
	ii	15 % buoyancy for design	
12	**Temperature (as per IRC:6-2017 clause 215)**		
	For bridge having difference between max and min air shade temperature-		
	>20^0 C	Mean of Maximum and Minimum air shade temperature ,- 10˚C whichever is critical	
	<20^0 C	Mean of Maximum and Minimum air shade temperature ,- 5˚C whichever is critical	
	The nonlinear temperature gradient for design of superstructure shall be considered as per clause 215.3 of IRC: 6-2017.		
13	**Seismic force (as per IRC:6-2017, clause 219)**		

S. No.	Design Figure		Standard
	i	Zone V	Culverts and MNB up to 10m span are not designed for seismic forces. For all other cases seismic forces shall be considered as per Clause 219 of IRC 6- 2017
14	**Expansion Joints**		
	i	Filler type	For span up to 10m(Section 2600 of the specification)
	ii	Strip Seal Type	For Span >10m and movement up to ±80 (Section 2600 of specification)
	iii	Modular Type	movement more than ±80 (Section 2600 of specification)
15	**Bearing**		
	i	Elastomeric	As per design requirements
	ii	Pot cum PTFE	As per design requirements
	iii	Pin and Guided Bearing	As per design requirements
	iv	Spherical Bearing	As per design requirements
16	**Wearing Coat**		65 mm thick
17	**Pre stressing (**IS: 14268, section 1800 of Specifications): • Uncoated stress relieved low relaxation steel • Type of Strand-Stress relieved multiply strands of low relaxation Ultimate Stress in Cable -1861 Mpa • Maximum pre stress jacking force-0.783(90% of 0.1% of proof load) 1) The maximum force applied to a tendon at active end during tensioning, shall not exceed 90% of 0.1% proof stress 2) The analysis of pre stressed section would be as per the stress strain properties given in clause 6.3.5 of IRC-112. 3) Maximum pre stressing force applied to structure immediately after transfer shall not be greater than 75% of characteristic tensile strength of pre stressing steel or 0.85 of 0.1% of proof load whichever less is. 4)For serviceability limit state the section would be checked for 10% lower (Inferior) and 10% higher (Superior) values of pre stressing force as per IRC -112		
18	**Sheathing**		HDPE
	Time Dependent material properties		
19	**Shrinkage:** Total shrinkage is auto-generous shrinkage and drying shrinkage		
20	**Creep:** Creep to be calculated with time and stress as per IRC112		

S. No.	Design Figure	Standard
21	**Coefficient of thermal Expansion-**12×10^{-6}/degree C	
22	**Modulus of Elasticity-**Modulus of Elasticity to be calculated as per short term and long term creep and shrinkage	
23	**Minimum Bar Diameter**	10 mm (refer Table 5.1,IRC112)
	Diameter if any reinforcing bar including transverse ties, stirrups etc. shall not be less than 10 mm. Diameter of any longitudinal reinforcement bars in columns/ vertical member shall not be less than 12 mm. However diameter of the reinforcing bars shall not exceed 25 mm in slabs and 32 mm in other member.	
24	**Margin in Material (FOS)**	
	All critical sections shall be checked for stresses under various load combinations. A suitable margin (preferably 5%) shall be there between maximum stress and allowable stress in concrete as well as reinforcement in the final design.	

1.1.5 Design Load and Stresses

Design loads shall be as per IRC: 6-2017, appropriate for the proposed carriageway width, type and properties of stream, location, altitude, etc.

Dead Load (DL)

The dead load i.e. the self-weight of the superstructure, substructure and foundations, backfill will be considered as per the Cl. 203 of IRC: 6 -2017 and are summarized as below:

Wet concrete including reinforcement - $2.6\ t/m^3$ (IRC: 87 – 2011)
Concrete (Cement Reinforced) - $2.5\ t/m^3$
Concrete (Cement Prestressed) - $2.5\ t/m^3$
Concrete (Asphalt) - $2.2\ t/m^3$
Earth (Compacted) - $2.0\ t/m^3$
Concrete (Cement - plain with plum) - $2.5\ t/m^3$

Superimposed Dead Load (SIDL)

SIDL comprises of the following items

Crash barrier without Hand Rail - 0.8 t/m
Crash barrier with Hand Rail - 1.0 t/m
Wearing Course - $0.246\ t/m^2$
Railing - 0.6 t/m
Footpath Load

Crash barrier is adopted as per IRC: 5-2015.

Live Load Combinations

Live load combinations mentioned in IRC: 6-2017 Table-6 shall be followed as per relevant carriageway width. In general for Bridges and Flyovers following combinations shall be used:

i) Class A 3-Lane Loading
ii) 1 Lane of 70R + 1 Lane of Class A Loading
iii) IRC Class SV Loading special Multi Axle Hydraulic Trailer Vehicle (Prime Mover with 20 Axle Trailer – GVW =385Ton shall be considered to ply for single carriageway or Dual Carriageway Bridge with a maximum eccentricity of 300mm from centre of carriageway.

Minimum clear distance between the two vehicles shall be 1.2m in transverse direction.
The loads which are not mentioned in this clause shall be as per IRC: 6-2017.

Where ever footpath is provided in the bridge Footpath live load is taken and bridge is also designed for without footpath case.

Live Load shall be considered at inner edge of the carriageway for stage 2 constructions and Girder shall be capable to take care the same.

Reduction in the longitudinal effect on bridges having more than two traffic lanes due to the low probability that all lanes will be subjected to the characteristic loads simultaneously shall be in accordance with the Table shown below:

Table 1-2: Lane Reduction Factor for Live Load

Number of lanes	Reduction in longitudinal effect
For two lanes	No reduction
For three lanes	10% reduction
For SV Loading, during passage on bridge, no other vehicle shall be considered to ply on the bridge. No wind, seismic, braking forces and impact on the live load need to be considered as SV shall moves at a speed not exceeding 5kmph over the bridge.	

Notes:

1) However, it should be ensured that the reduced longitudinal effects are not less severe than the longitudinal effect, resulting from simultaneous loads on two adjacent lanes. Longitudinal effects mentioned above are bending moment, shear force and torsion in longitudinal direction.

2) The above Table is applicable for individually supported superstructure of multi-laned carriageway. In the case of separate sub-structure and foundations, the number of lanes supported by each of them is to be considered while working out the reduction percentage.

Special vehicle loading

Structure need to be checked for special vehicle also. The total load 385T of special vehicle is the load considered to act at 300mm from center of carriageway. No other load is considered to moving on structure when special vehicle is moving.

Longitudinal forces

In all road bridges, provision shall be made for longitudinal forces arising from anyone or more of the following causes:

a) Tractive effort caused through acceleration of the driving wheels;
b) Braking effect resulting from the application of the brakes to braked wheels.
c) Frictional resistance offered to the movement of free bearings due to change of temperature or any other cause. The braking effect on a simply supported span or a continuous unit of spans or on any other type of bridge unit shall be assumed to have the following value:

a) In the case of a single lane or a two lane bridge: twenty percent of the first train load plus ten percent of the load of the succeeding trains or part thereof, the train loads in one lane only being considered. Where the entire first train is not on the full span, the braking force shall be taken as equal to twenty percent of the loads actually on the span or continuous unit of spans.

b) In the case of bridges having more than two-lanes: as in (a) above for the first two lanes plus five per cent of the loads on the lanes in excess of two.

Construction Live Load

Construction load wherever applicable may be considered as 0.36 t/m^2 of the form area to be considered as per IRC 87-2011.This load include load due to mobile construction plant or equipment and temporary loads.

A minimum dynamic amplification of 50% of the loads during normal lifting operations is to be assumed. When Pre cast segmental construction is done consequence to stability to the structure to be determine due to sudden loss of segment. Dynamic amplification of 100% is to be considered.

Differential Settlement

If the riding quality permits, clause 706.3.2.1 of IRC: 78-2014 specify that the calculated differential settlement between the foundations of simply supported span shall not exceed L / 400 of the distance between the foundations, where L is distance between two foundations. In case of structure sensitive to differential settlement such as continuous structures the total value of differential settlement shall be taken as 12mm out of which $2/3^{rd}$ value to be considered for immediate settlement.

Temperature Gradient

Effective bridge temperature shall be estimated from the isotherms of shade air temperature given in fig 15 and fig 16 of IRC: 6-2017. Difference in temperature between the top surface and other levels through the depth of the structure, where ever applicable shall be taken in accordance with clause :215.3 of IRC:6-2017.

Centrifugal Forces

Centrifugal forces are considered for spans in curved portion as per IRC 6-2017 Centrifugal forces shall be determined from following formula:

$$C = WV^2/127R$$

Where,

C =Centrifugal force acting normal to the traffic. W = Live load (tons/m)
V= Design speed of vehicles (Km/ hour)
R = Radius of curvature (m)

It is considered to be acting at 1.2m above deck level**.**

Earth Pressure

1. All earth retaining structures like Abutment and Other Earth Retaining Structures designed to retain earth fills shall be proportioned to withstand pressure calculated in accordance with any rational theory. Coulomb's theory, subject to the modification that the center of pressure exerted by the backfill, when considered dry, is located at an elevation of 0.42 of the height of the wall above the base instead of 0.33 of that height.

2. For RCC Box Structure-Active Earth pressure / Earth pressure at rest will be considered to be acting on the vertical walls of the RCC Box. The Co-efficient of such Earth pressure will be taken as 0.5.

3. Surcharge Pressure-All Earth retaining wall is designed for a live load surcharge pressure equivalent to 1.2 m earth fill as per IRC 6-2017.

4. Dynamic Earth Pressure-All Earth retaining wall is designed for the pressure from earthfill behind the wall during an earthquake calculatedas per IRC 6-2017. The static component of the total pressure shall be applied at an elevation h/3 above the base of the wall. The point of application of the dynamic increment shall be assumed to be at mid-height of the wall.

Wind forces

1. The superstructure shall be designed for wind induced horizontal forces (acting in the transverse and longitudinal direction) and vertical loads acting simultaneously. The assumed wind direction shall be perpendicular to longitudinal axis for a straight structure or to an axis chosen to maximize the wind induced effects for a structure curved/skewed in plan.

2. The substructure shall be designed for wind induced loads transmitted to it from the Super structure and wind loads acting directly on the substructure. Loads for wind with Live Load and without Live load shall be envisaged.

3. The longitudinal force on bridge superstructure (in N) shall be taken as 25% and 50% of the transverse wind load as calculated as per Clause 209 for beam/box/ plate girder bridges and truss girder bridges respectively.

Water Current Forces

Any part of a road bridge which may be submerged in running water shall be designed to sustain safely the horizontal pressure due to the force of the current.

On piers parallel to the direction of the water current, the intensity of pressure shall be calculated from the following equation:

$$P = 52KV^2$$

Where,

P = intensity of pressure due to water current, in kg/m^2
V = the velocity of the current at the point where the pressure intensity is being calculated, in meter per second, and
K = a constant having the following values for different shapes of piers illustrated in Fig.6-1

i) Square ended piers (and for the superstructure)	1.50
ii) Circular piers or piers with semi-circular end	0.66
iii) Piers with triangular cut and ease waters, the angle included between the faces being 30° or less	0.50
iv) Piers with triangular cut and ease waters, the angle included between the faces being more than 30° but less than 60°	0.50 to 0.70
v) -do- 60 to 90°	0.70 to 0.90
vi) Piers with cut and ease waters of equilateral arcs of circles	0.45
vii) Piers with arcs of the cut and ease waters intersecting at 90°	0.50

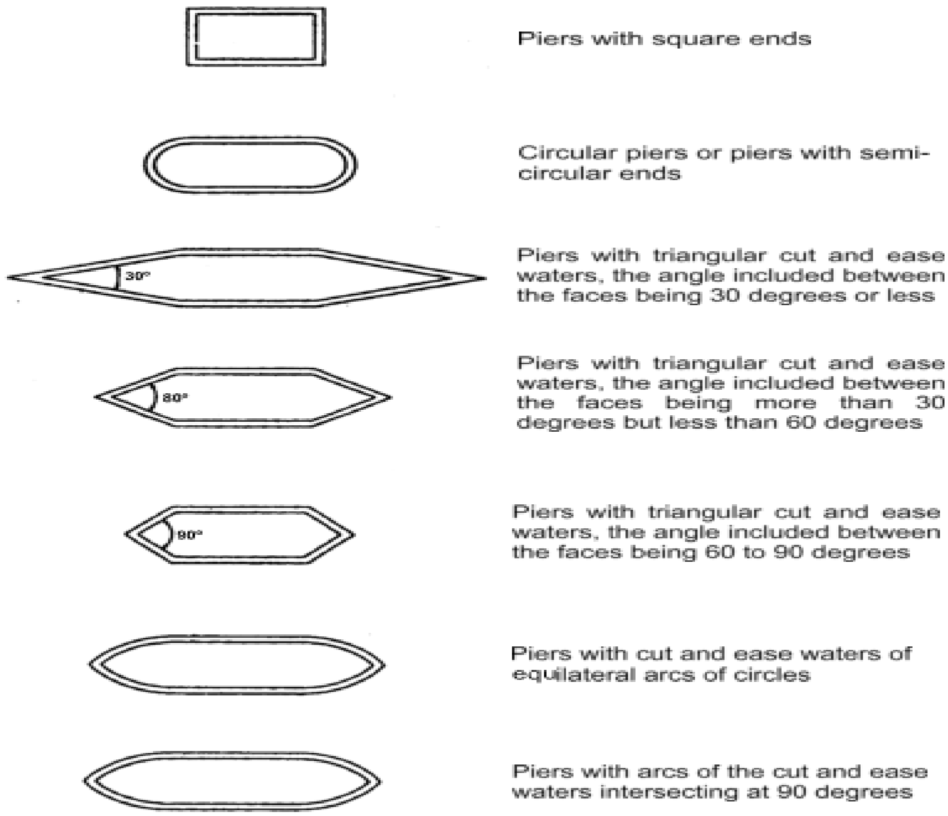


Figure 1-1: Shape of Bridge Piers

Buoyancy

1. In the design of abutments, especially those of submersible bridges, the effects of buoyancy shall also be considered assuming that the fill behind the abutments has been removed by scour.

2. To allow for full buoyancy, a reduction shall be made in the gross weight of the member affected by reducing its density by the density of the displaced water.

   The density of water may be taken as 1 .0 t/m^2.

   For artesian condition, HFL or actual water head, whichever is higher, shall be considered for calculating the uplift.

3. In the design of submerged masonry or concrete structures, the buoyancy effect through pore pressure may be limited to 15% of full buoyancy. 213.4 In case of submersible bridges, the full buoyancy effect on the superstructure shall be taken into consideration.

Seismic Forces

The project corridor falls under seismic zone-V which is Severe seismic zone. Seismic design is carried out as per zone and as per codal provisions along with provision of ductile detailing and seismic arrest or Block.

All bridges supported on piers, pier bents and arches, directly or through bearings, and not exempted below in the category (a) and (b), are to be designed for horizontal and Vertical forces as given in the following clauses.

The following types of bridges need not be checked for seismic effects:

a) Culverts and minor bridges up to 10 m span in all seismic zones.

b) Bridges in seismic zones II and III satisfying both limits of total length not exceeding 60 m and spans not exceeding 15 m

The effect of Vertical component may be considerd for all elements in Zone-V for following cases as applicable for said project.

a) Bearing and Linkage
b) Horizontal cantilever structural element
c) For stability

Combination of component motion

The seismic force assumed to be coming from any horizontal direction. For this purpose two separate analyses shall be performed for design seismic forces along two orthogonal directions. The design seismic forces resultants at any cross-section of a bridge component resulting from the analysis in two orthogonal horizontal directions shall be combined as below:

a) $\pm r1 \pm 0.3r2$
b) $\pm 0.3r1 \pm r2$

Where,

r1 = Force resultant due to full design seismic force along x-direction
r2 = Force resultant due to full design seismic force along z-direction

To improve the performance of the Bridge during earth quake, the bridges in seismic zone III may be specifically detailed for ductility for which IRC: 112 shall be referred.

Seismic Analysis

The Seismic Analysis of the bridges shall be carried out using following method as per applicability defined in Table 5.3, of IRC :SP:114 – 2018,depending upon the complexity of the structure and the input ground motion.

a) Elastic seismic acceleration method (Static load)
b) Elastic Response Spectrum Method
c) Time history Method

Accidental Load

Bridge piers of wall type, columns or the Frames built in median or in the vicinity of the carriageway supporting the superstructure shall be design to withstand Vehicle collision loads as per clause 222.1 of IRC: 6- 2017.

The effect of collision load shall not be considered on abutments or on the structures separated from the edge of the carriageway by a minimum distance of 4.5m and also not be combined with principal live loads on the carriageway supported by the structural members subjected to such collision loads as well as wind or seismic load as per cl 222.1.2 of IRC: 6 – 2017.

Where pedestrian/cycle track bridge ramps and stairs are structurally independent of the main highway spanning structure, their support need not be design for vehicle collision loads.

Material factor of safety under collision load, reference shall be made to the provision in IRC: 112 for concrete and IRC: 24 for steel. For permissible overstressing in foundation, refer provision of IRC: 78.

Collision Load

The normal loads given in Table 22 of IRC: 6 2017 shall be considered to act horizontally as Vehicle Collision loads. Supports shall be capable of resisting the main and residual components acting simultaneously. Load normal to the carriage way below and loads parallel to the carriageway below shall be considered to act separately and shall not be combined.

The loads in Table 22 indicated in clause 222.3.1 are assumed for vehicles plying at velocity of about 60km/hours. In case of vehicles travelling in lesser velocity, the load may be reduced in proportion to the square of the velocity but not less than 50 percent.

The bridge supports shall be designed for residual load component only, if protected with suitably designed fencing system taking in to account its flexibility, having a minimum height of 1.5m above the carriageway level.

1.2 Hydrological and Hydraulic Study

1.2.1 Main Objective

i. The main objective of the hydrological and hydraulic study is to determine the required size of drainage structures to allow the estimated design flow of the streams to cross the road safely
ii. To check whether waterway of proposed structures are sufficient to transmit the flow without risk so that appropriate decisions could be taken concerning their span arrangement.
iii. To estimate the peak discharge for 100-year return period flood
In order to achieve these objectives above mentioned, the work flow of hydrological & hydraulic analysis has been given in schematic diagram below:

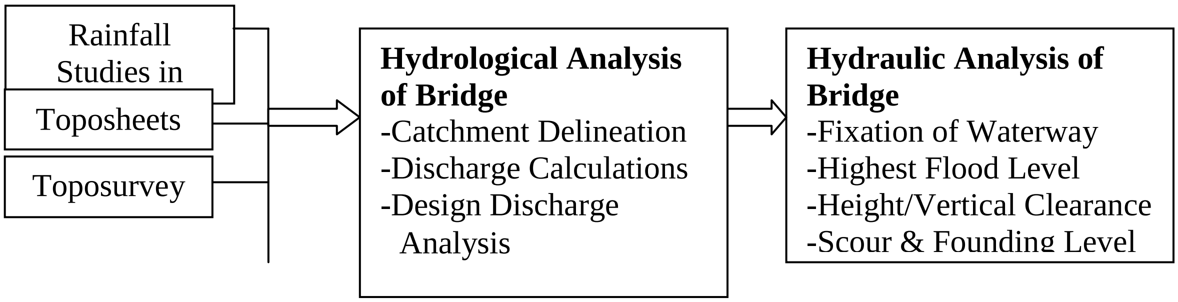


Figure 1-2 : General studies of Hydrological and Hydraulic Analysis

1.2.2 General Description of the Project area

The bridge sites lie in Kohima to Nagaland/Manipur border road in the state of Nagaland. All part of the road section exists in hilly nature of Naga Hills. The Naga Hills rise from the Brahmaputra Valley in Assam to about 2,000 feet (610 m) and rise further to the southeast, as high as 6,000 feet (1,800m). At Mount Saramati the Naga Hills merge with the Patkai Range which forms the boundary with Burma. Rivers such as the Doyang and Diphu to the north, the Barak River in the southwest, dissect the entire state. Most of the part of project road is covered with forest. The evergreen tropical and the sub-tropical forests are found on the road.

Nagaland has a largely monsoon climate with high humidity levels. Annual rainfall averages around 70–100 inches (1,800–2,500 mm), concentrated in the months of May to September. Temperatures range from 70°F (21°C) to 104°F (40°C). In winter, temperatures do not generally drop below 39°F (4°C), but frost is common at high elevations. The state enjoys a salubrious climate. Summer is the shortest season in the state that lasts for only a few months. The temperature during the summer season remains between 16°C (61°F) to 31°C (88°F). Winter makes an early arrival and bitter cold and dry weather strikes certain regions of the state. The maximum average temperature recorded in the winter season is 24°C (75°F). Strong north-west winds blow across the state during the months of February and March.

1.2.3 Data Collection and Data Analysis

1.2.3.1 Requirements of Data collection for Hydrological Study

Toposheets, Rainfall Data and topographical survey data are the primary basis of hydrological studies. The data requirement and their applications are summarized in the table as given below:

Table 1-3: Toposheets, Rainfall Data & Topographical Survey Data

S. No	Title	Category (Primary/Secondary)	Source	Relevant Information
1	Toposheets (1:50,000) & 1:2,50,000)	Primary	Survey of India & U.S. Army Map Service	Mark the catchment area, length of the stream and fall in elevation from originating point to the point of crossing & slope of the stream could be determined to calculate the design discharge of the

S. No	Title	Category (Primary/Secondary)	Source	Relevant Information
				proposed bridge location
2	Rainfall	Primary	IMD Rainfall Map from CWC Subzone reports	100-yr 24-hour Rainfall is used to calculate the design discharge
3	Topographical survey data	Primary	Survey at field/ site along the alignment	To make cross section and longitudinal slopes of the Nala / stream used in computation

Source: Survey of India & U.S. Army Map Service, CWC subzone reports and field data

1.2.4 Hydrological and Hydraulic Study for Bridges

For performing the hydrological and hydraulic analysis which essentially need the design flood of a specific return period for fixing the waterway vis-à-vis the design High Flood Level (HFL) of bridges depending upon their size and importance to ensure safety as well as economy. As per IRC 5-2015-Section I General Features of Design specify that the waterway of a bridge is to be designed for a maximum flood of 100 years return period.

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

- Empirical Formulae
- Rational Method

The above methods have been discussed in detail as indicated below in subhead of Hydrological Aspect and Hydraulic Aspect.

1.2.4.1 Empirical Formulae

The empirical formulae used for the estimation of the flood peak are essentially regional formulae based on the statistical correlation of the observed peak and important catchment properties. Dicken's & Ryve's method (empirical formulae) most popular method as given in IRC: SP-13-2004 and the formula for assessment of peak discharge only from the catchment area.

a. Dicken's Method

$Q = CA^{3/4}$

Where,
Q = Peak discharge in cumecs
A = Catchment area in sq km
C= Dicken's coefficient
= 11-14 where the annual rainfall is 600 mm to 1200 mm
= 14-19 where the annual rainfall is more than 1200 mm
= 22 in Western Ghats

b. Ryve's Method

$Q = CM^{2/3}$
Where,
Q = Peak discharge in cumecs
M = Catchment area in sq km
C = Coefficient
Coefficient C varies with the region as given below:
C = 6.8 for areas within 25Km of coast.
=8.5 for area between 25 Km to 160 Km of the coast.
=10 for limited areas near the hills

1.2.4.2 Rational Method

Discharge Estimation for the Catchment Areas Less than 25 sq km

This is a well-known method as given in IRC: SP-13-2004 and the formula for assessment of peak discharge from project catchment takes into account rainfall, runoff under various circumstances time of concentration and critical intensity of rainfall.

Here, 100 year Peak Discharge is calculated by following formula
$Q_{max} = 0.0278\ P\ f\ A\ I_c$

Where,
Q_{max} = design flood (m^3/s) for 100 year return period
f = Areal Distribution/spread Factor
C = Coefficient of runoff for the catchment characteristics
A = catchment area (Ha)
I_c = Critical intensity of rainfall in cm/hr during the time of concentration.

Time of concentration has been taken from IRC: SP: 13-2004 Equation no. 4.9
$t_c = \left(0.87 \times \frac{L^3}{H}\right)^{0.385}$

Where,
t_c = time of concentration (hours)
L = the distance from the critical point to the structure (km)
H = the fall in level from the critical point to the structure (m)

Intensity of rainfall has been determined from formula $I_c = \frac{F}{T}\left(\frac{T+1}{t_c+1}\right)$ in cm/hr
F = Total Precipitation (cm).
T = Duration of Strom (Hours).

The value of runoff coefficient (P) depends on the porosity of the soil, vegetation cover, surface storage initial state of wetness of soil, area , shape & size of the catchment and may be taken from the below given Table.

Table 1-4: Maximum Value of P (Table 4.1: IRC: SP: 13-2004)

S. No	Description of Catchment	Value of P
1	Steep, bare rock and also city pavements	0.90
2	Rock, steep but wooded	0.80
3	Plateaus, lightly covered	0.70
4	Clayey soils, stiff and bare	0.60
5	Clayey soils, stiff and bare and lightly covered	0.50
6	Loam, lightly cultivated or covered	0.40
7	Loam, largely cultivated	0.30
8	Sandy Soil, light growth	0.20
9	Sandy soil, light growth covered, heavy brush	0.10

The value of areal distribution factor depends on catchment area as shown in below graph given in IRC-SP-13.

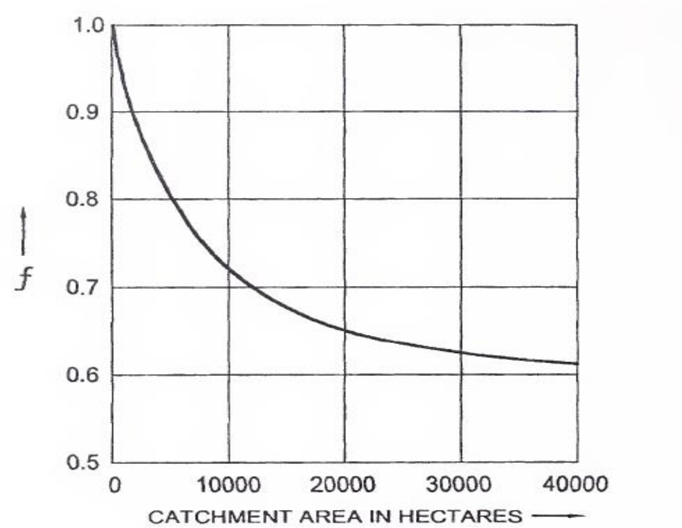


Fig. 4.2 'f' curve

Figure 1-3 : f -Curve

Thus after calculating the above parameters, the 100-year return period peak discharge has been calculated for all the bridges using the formula as given above.

1.2.5 Afflux Calculation

Afflux calculations are carried out as procedures given in IRC: 5-2015 and following Mole's worth formula have been used for calculation purpose.

Afflux $h = [(v^2/17.88)+0.015] \times [(A/A1)^2-1]$
Where, v - Average velocity of river prior to obstruction in m/sec
A - Unobstructed sectional area of river in Sq m
A1 - Sectional area of river at obstruction in Sq m

1.2.6 Scour Depth Calculation

Scour calculation for pier & abutment or box type bridges has been carried out using IRC SP: 13-2004 and IRC: 78-2000. Scour depth has been calculated using following formula:

$$\mathbf{d_{sm} = 1.34 \times (D_b^2/K_{sf})^{1/3}}$$

Where,

d_{sm}	=	Mean scour depth below HFL
D_b	=	(Q/L)
Q	=	Discharge adopted for scour depth calculations (after increasing the design discharge as per clause 703.1.1 of IRC 78: 2000)
L	=	Clear waterway after making deductions for obstruction upto HFL
D_b	=	The design discharge for foundation per metre with effective linear waterway
K_{sf}	=	Silt factor for a representative sample of bed material obtained upto the level of anticipated deepest scour
HFL	=	Highest flood level

Mean scour depth at abutment	=	1.27 * d_{sm}
Maximum scour level at abutment	=	HFL- 1.27 * d_{sm}

Similarly,

Mean scour depth at pier	=	2 * d_{sm}
Maximum scour level at pier	=	HFL - 2 * d_{sm}

The founding level shall be fixed on the basis of calculated scour level and bearing capacity of soil & rock.

1.2.7 Summary and Recommendations

Flood discharge estimated in three different ways should be compared with existing bridge conditions and watermarks of HFL & highest of these should be adopted as design discharge in general. However, the general condition laid down in IRC SP-13 has been used to fix the design discharge, that is, if the discharge obtained by one method is greater than 1.5 times the discharge obtained from the other, the design discharge should be limited to 1.5 times of the smaller one. Accordingly, the design discharge has been established for all the bridges. Based on the highest discharge HFL shall be reassessed using slope area method for the hydraulic calculation, provision of spans and scour levels.

1.2.8 Roadside Drains

Inadequate cross drainage on a hill road causes softening of the sub-grade and renders it too weak to take the load of the moving traffic. Roadside drains are necessary on a hill road. Section of side drain shall be taken up as specified in IRC: SP: 48-1998.

1.2.9 References

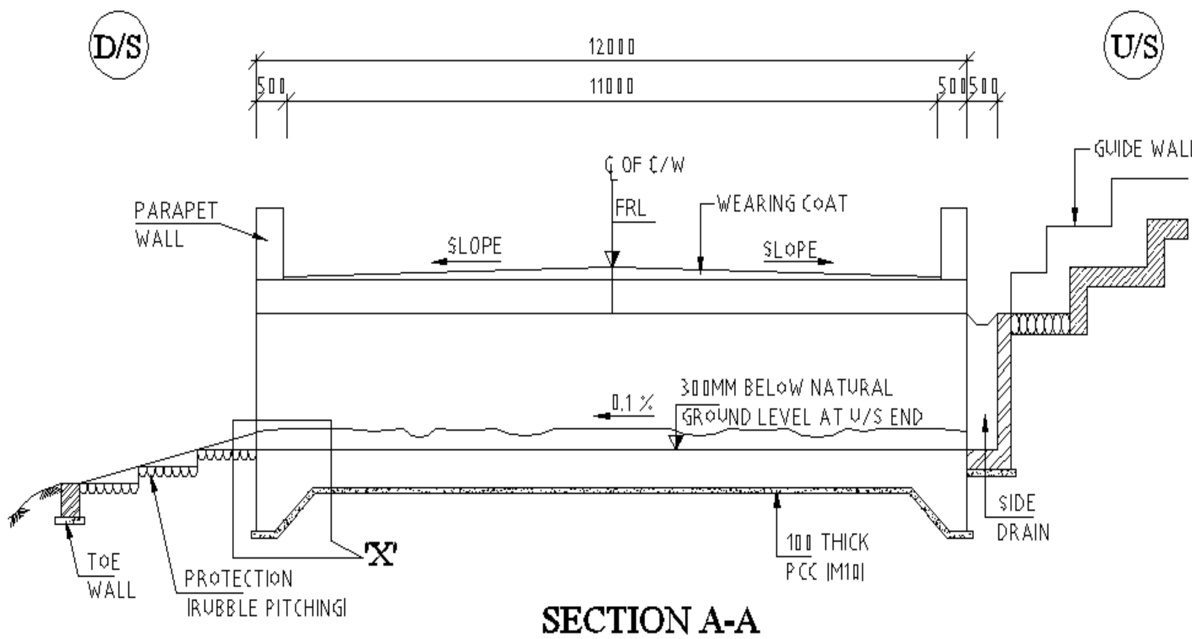
- IRC: SP-13:2004- Guidelines for design of small bridges & Culverts
- IRC:5:2015- Standard specifications and code of practice for road bridges section-I

- IRC:89:1997- Guidelines for design and construction of River Training and control works for road bridges
- IRC-78:2014- Standard specifications and code of practice for road bridges Section – VII
- CWC- Flood Estimation report for South Brahmaputra Subzone 2b
- Journal of IRC - July- Sep 2009, Technical paper No. 551, Determination of waterway under a bridge in Himalayan Region - some case studies By S. K. Mazumdar
- IRC SP-42- 2014- Guidelines of road drainage
- IRC SP-48- 1998- Hill road manual

1.3 PROTECTION WORK

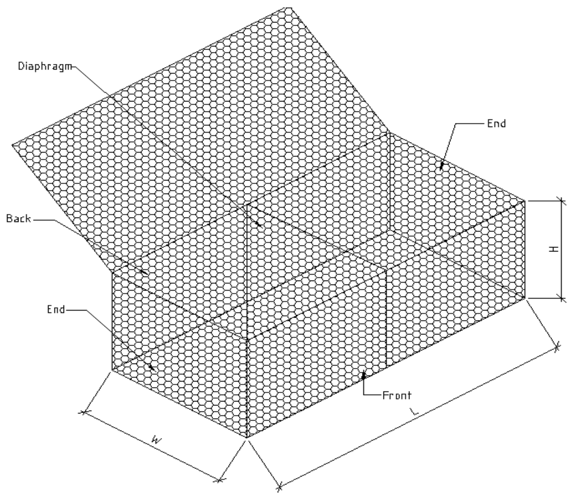
Protection works for various Structures are provided as follows:

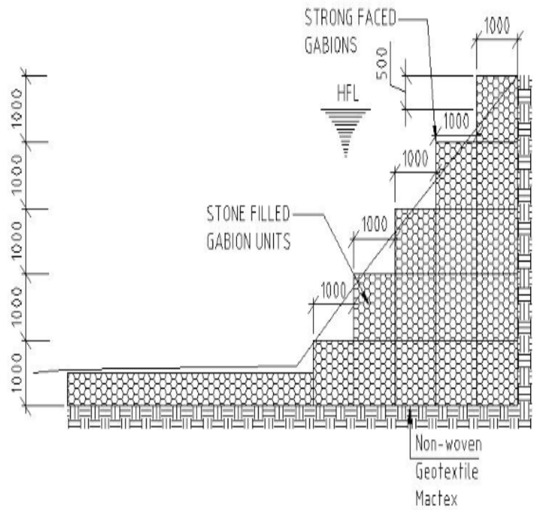
I Box Culverts



SECTION A-A
(SCALE-1:100)

II River embankment protection in bridge location





Gabion mattresses are large plan area gabion units in depths generally of 0.15,0.225, 0.3 and 0.5m.Each unit as standard is sub divided into cells normally not exceeding 2m x 1m.The purpose of a gabion mattress is to protect river beds, river embankments and hilly embankments against erosion. Mattresses can be manufactured in any of the welded wire mesh/ wire diameter as specified in IRC 89.The Depth of mattress protection is dependent upon the following criteria Soil type. With fine soils such as sands and silts are generally more erodible than courser grained soils, the use of a geo-textile separator is imperative. The more erodible the soil, the greater the thickness of mattress required. In straight water courses, the flow approaches laminar flow conditions. However, on bends or changes in direction, the flow conditions change to increased water velocity or more turbulent conditions. The greater the water velocity or turbulence, the greater the mattress thickness required

III Selection of Slope Protection Work

➢ Basis of Design of Active and Passive Slope Stabilization System

Introduction

Hazards provoked by released rocks and slides due slope instabilities from loose, disintegrated and weathered surfaces as well as debris flows in potentially risky areas often occur and are able to cause considerable damages project sites – located in areas prone and vulnerable to such natural hazards in particular. This can be observed frequently in fresh cuttings / excavation or untreated slides left for the nature to rescue.

Depending on the particular conditions, the risks provoked (connected by previous indications) can substantially be reduced or eliminated through the application of active system (high performance Slope Stabilization System), suitably placed over the affected slope. Persons and infrastructure have to be protected or through the use of active systems (high-tensile meshes and nets in combination with nailing) which directly act as coverage, impeding the movement of unstable soil and rock.

In odd cases, affected areas may not be treatable by using unique technique, like with deep-seated failure might require in depth analysis of global stability. Such analysis gives a clear indication of potential failure zones, Length of Soil / Rock Anchors or cable anchors.

Therefore, the necessity of a proven and recognized protection system rises, which acts in an efficient and reliable manner, being based on high resistant components and respecting ecological aspects. This, with the objective to prevent severe and expensive consequences as well as questions of liability related to the operation.

Active and Passive Slope Stabilization Systems

Flexible systems anchored to the ground constitute a technique for slope surface stabilization. These systems are formed by membranes, made of High Tensile Wire Meshes (Tensile Strength 1770 N/ mm2), and bolts anchored to the ground. This technique has spread extensively due to its low visual impact and its minimal influence on traffic during installation.

Flexible systems may be classified as either Active or Passive.

Active systems attempt to prevent rock detachment or soil sliding, as they apply a pressure on the ground through an initial pre-tension of the flexible membrane that covers the unstable zone. In general Active System consist of High Tensile Mesh System (Tensile Strength > 1770 N/mm 2) which are nailed at definite pattern on the surface where active forces are likely to cause break out and leads to destabilization of slopes. By adding, High Tensile Steel Mesh to the system, the nail pattern can be adjusted for optimization of the cost of laying the entire system. In this case, the nail depth is decided based on global stability analysis, and nail patterns (which includes distance , Dia of Rock bolts / soil anchors) are decided based on extensive manual and software analysis. . The analysis should take care of Streaming Pressure (Pore Water Pressure), Seismic Loads, live load and dead load (if applicable).

In contrast, passive systems employ rigid membranes which are not pre-tensioned during installation to the surface; so, they are unable to exert any initial pressure on the ground. They are therefore used in the form of Drape, Attenuators , Rock Fall Barriers , Debris Flow Barriers , Avalanche Barriers and Shallow landslide barriers.

Problem Analysis

It is generally true that the overburden material has causes a landslide due to increase in pore water pressure, result of excessive rainfall or dynamic forces on the surface. Whether, Active or Passive Stabilization is to be adopted, it depends on the site. Fresh Slope, fresh Cuts after widening up to 50 m high with berms and drainage arrangement with presence of dynamic forces on the surface, surface inclination will be designed for active stabilization based on the site condition and proposed alignment

To Design and Active Stabilization System , some details are required like Geological Data, Geological Cross – Section, Geotechnical Report, Topography of Site, Global Stability Analysis and Physical Observation

Basis of Slope Stabilization Measures

In order to evaluate the surface stability of the slope cut and to propose adequate measures, analysis to be carried out based on the cohesion & friction values of the in situ soil. The results of the analysis provide information such as nail distance, nail type and appropriate mesh cover.

The superficial instability analysis is based on following system components which are part of the High Tensile Mesh Slope Stabilization System with different diameter, depending on requirement.

The slope surface is intended to stabilize in combination with soil nailing and rock bolting respectively and with a flexible facing (High Tensile Mesh) for active stabilization of the slope. Bore hole diameter shall be 70-90 mm.

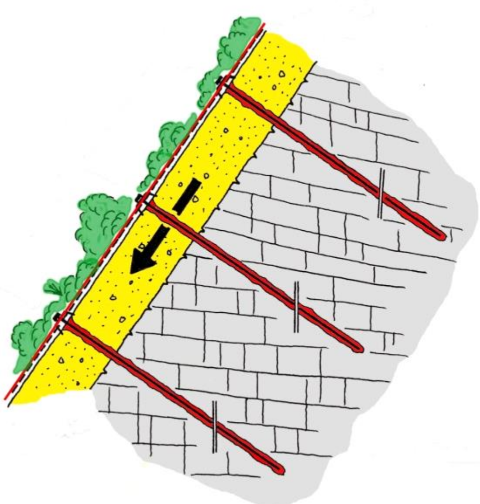
The use of the high-tensile steel wire mesh as a flexible surface stabilization measure has proved its suitability in numerous cases and offers ideal possibilities for an efficient and economical stabilization of slopes.

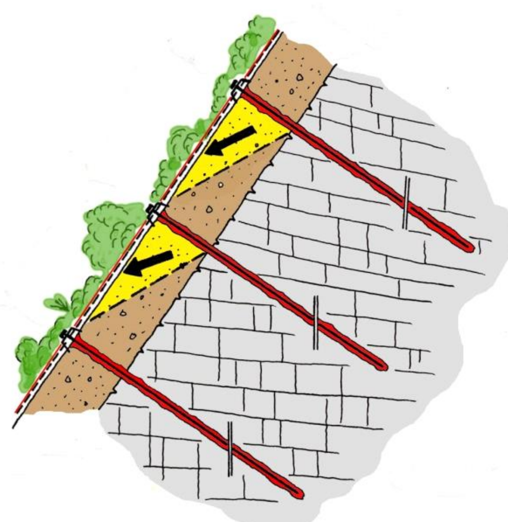
Superficial Stability Calculations

Investigation of Superficial Drawer-shaped bodies liable to slide down

The following proofs have to be calculated for soil slopes and weathered rock slopes.

1. Investigation of superficial slope-parallel instabilities
2. Investigation of local instabilities between single nails





The following three proofs of bearing safety have to be established in the context of the investigation of superficial slope-parallel instabilities:

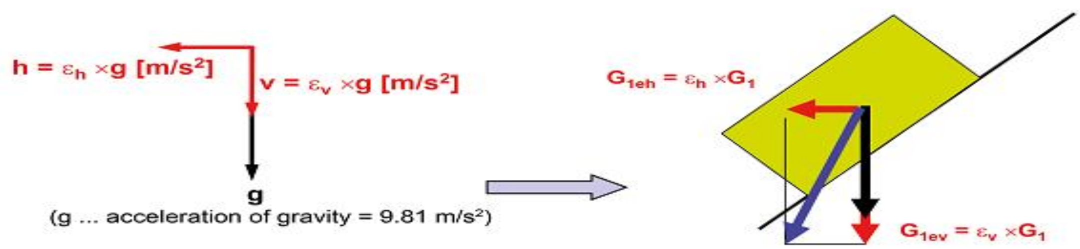
- Proof of the nail against sliding-off parallel to the slope
- Proof of the mesh against puncturing
- Proof of the nail to combined strain

Bearing resistance types to be known:

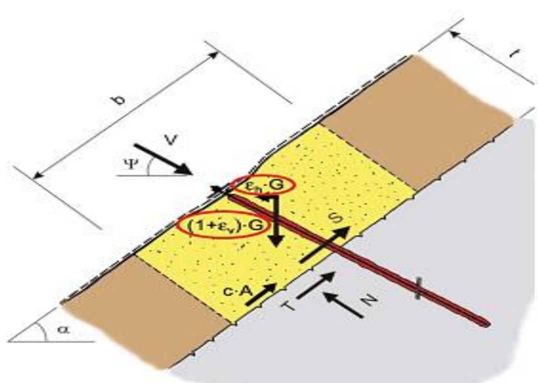
- D_R = Bearing resistance of the mesh to pressure strain
- S_R = Bearing resistance of the nail to shear strain
- T_R = Bearing resistance of the nail to tensile strain

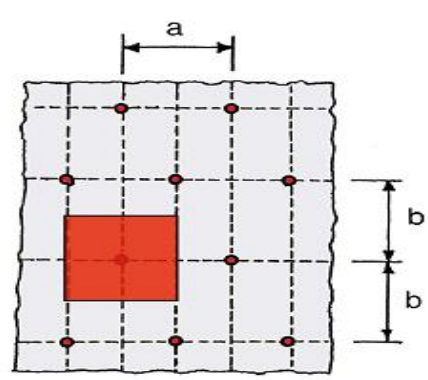
1.3.1 LOAD CASE - Earthquake

Depending on the importance of the structure and the seismological situation, possible additional effects from earthquakes must be investigated when dimensioning slope stabilization systems. Generally this takes place using the substitute force procedure. Here, accelerations acting on a fracture body are converted through the factors εh and εv to additional forces in the horizontal and vertical direction. These additional forces must be appropriately taken into account in the equilibrium considerations. With steeper slopes, in general in solid rock, special investigations are necessary e.g. tilting and sliding individual blocks. Shown below are the additions in the formulas resulting from the earth-quake load case. The corresponding individual proofs remain the same as previously described.



Investigation of instabilities close to the surface and parallel to the slope





Load Case "Streaming Parallel to the Slope"

Described below is the influence of streaming pressure as a result of precipitation water, respectively inflowing ground or slope water in loose rock slopes in the equilibrium considerations. In principle the two types of inflow with precipitation water (from the outside on to the slope) and slope water (from the inside) can be differentiated as shown in Fig. 19. With both cases it is assumed that a streaming parallel to the slope occurs after the saturation of the material.

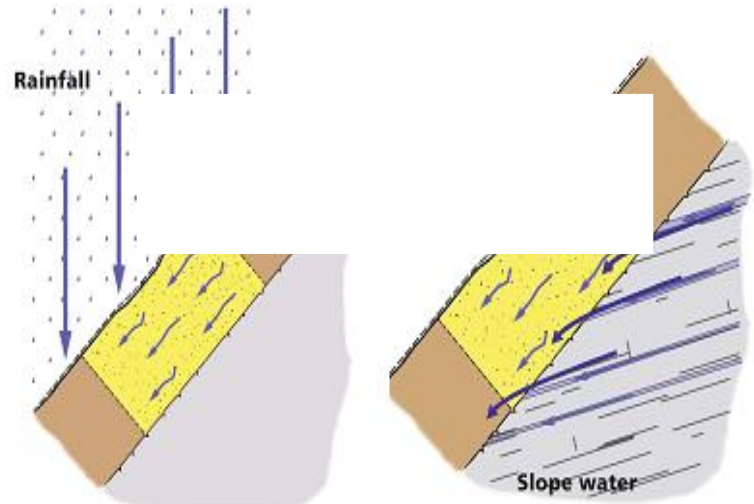
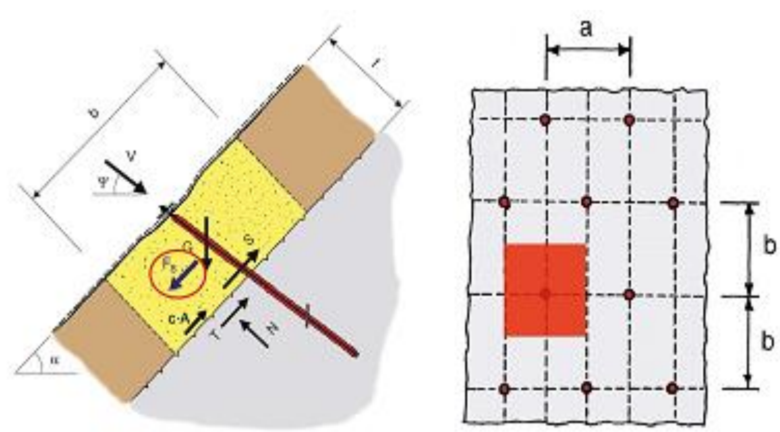


Fig. 19: Streaming parallel to the slope in the case of intensive rain (left) and slope water, e.g. in waterbearing interbeds, clefts etc. (right)

Investigation of instabilities close to the surface and parallel to the slope

The additional force Fs represents the resulting streaming force parallel to the slope and is calculated from the sum of the unit weight of the water (γw), the hydraulic gradient (i = sin α) and the volume of the fracture body (V). In the calculation, buoyancy should also be taken into ac-count with the own weight G of the cubic body. The individual proofs remain the same as previously stated.

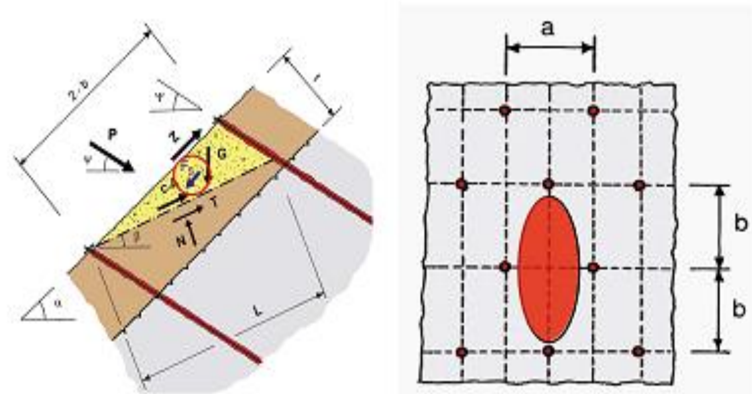
Fig. 20: All forces applied to a cubic body including the streaming load case, $F_s = \gamma_w \cdot i \cdot V = \gamma_w \cdot \sin\alpha \cdot V$



Investigation of local instabilities between the nails

Break mechanism A

Fig. 21: All forces applied with the break mechanism incl. the streaming load case



Passive Protection

In general , a passive protection system is just like arresting the dislodged mass from the parent mass before it hits the area which can be affected . Such system includes Rockfall Barrier , Canopy Barriers , Debris Flow Barriers and Shallow Landslide in broader terms.

Flexible rockfall barriers are proven protection systems to mitigate the hazard of rocks falling on people and infrastructure. To assure that such systems are able to dynamically stop falling rocks in reality, several guidelines with full scale tests were introduced worldwide. These guidelines consider very standardized and repeatable load cases but cannot take possible extreme loads into account.

Physical Examination of Site

The design of rock-fall protection systems is based on detailed investigations by engineering firms, particularly taking into account the following geotechnical aspects to define the range of possible applications. These aspects include the following parameter:

- Previous rock-fall events , wherein the size, density and frequency of falling rocks is noted.
- Condition of the rock-fall breakout zone, to find out the favorable trajectories followed by rolling stones.
- Stability assessment of the entire rock-fall zone
- Rock-fall frequency
- Size of the blocks to be intercepted
- Trajectories and bounce heights of stones
- Calculation of kinetic energies
  (Based on the numerous simulations run to find out the possible trajectories and final kinetic energy of the shooting stone at the time of impact)
- Positioning of the barrier by considering the local topography)
  (Depending on the most rock-fall prone area, the rock-fall barriers are positioned to allow a better catchment of the falling/rolling stones)
- Anchorage conditions

(All the protection and stabilization work only if properly anchored to the ground.

Depending on the local geological and soil conditions, depth and arrangement of the anchors are designed and proposed to offer maximum safety and ensure durable protection.)

Design of Attenuators and Hybrid Barrier Systems

In this case, an impact zone constructed of a high tensile spiral rope net intercepts upslope rockfall. Rock blocks are then guided through a tail drape section and contained to the base of slope (Fig. 1).

Instead they use the boulder impacts with the net and its interaction with the slope to attenuate the velocity and trajectory of rock fall.

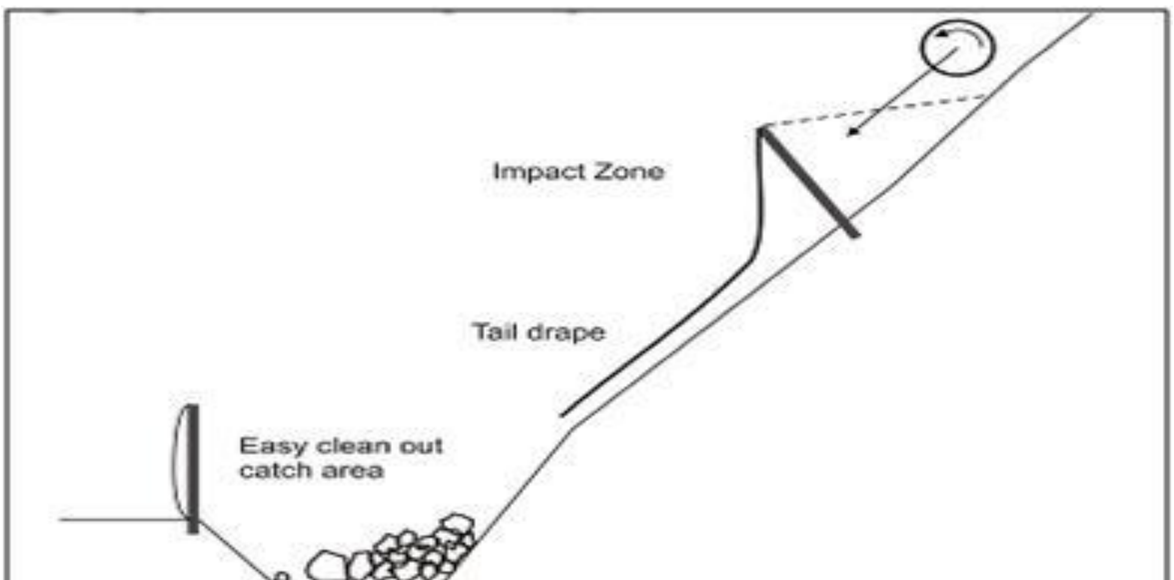


Figure.1. Rockfall attenuator and hybrid concept

For design progressed, a number of alternatives for the design and location of the system are considered. The placement of the system would need to meet the following criteria:

- ✓ The height should be sufficient to catch any of the bouncing rock blocks which may occur,
- ✓ The lateral extent needs to be enough to catch rocks originating from several locations along the high portions of the slope,
- ✓ The “tail” length needs to be long enough to allow the dissipation of energy, yet allow rocks to pass beneath it and fall safely into the ditch,
- ✓ The system energy capacity should be such that rocks representing the maximum expected size that may fall would be effectively contained.

The inputs are almost same as the rock fall barrier. The energy possessed by the moving mass after deflection is assumed and the hybrid is designed based on that impact energy / sqm of mesh.

Design of Debris Flow System

The terrain has numerous number of gorges and nallah's. During precipitation it drags debris with considerable velocity along with it. These slide generally blocks the roads and causes havoc in cases of higher discharged. A Debris Flow system is a passive system made up of high tensile steel wire and accessories that is erected in between the

Debris Flow and habitation to provide adequate protection.

As part of an extensive on-site inspection, it is tasked with planning work determined the corresponding locations of the barriers, including in the catchment area, on the basis of existing visible gullies and corresponding deposits of loose material.

The locations of barriers shall be determined in order to guarantee that sufficient material would be retained in the event of a debris flow, and several smaller systems, known as debris breaks, were planned for the purposes of channel and slope stabilization.

Input Parameters

The design of debris flow protection systems is based on detailed investigations by specialized engineering firms, particularly taking into account the following geotechnical aspects to define the range of possible applications:

- Former debris flow events, wherein the density, volume, and type of debris flow is considered.
- Catchment area (condition of soil, inclination, size)
- Estimated rainfall intensity: to calculate the total volume of the flow
- Debris flow input parameter (volume of decisive surge and total volume, density, middle front velocity)
- Composition of debris flow (debris fraction, water content, density)
- Probability of occurrence
- Calculation of decisive load cases
- Barrier location (Consideration of local topography)
- Anchorage conditions

These condition may then be simulated on in any compatible available Software , to get desired results.

1.3.2 Specifications For Slope Protection System

1.3.2.1 Active System

The active slope stabilization system provides a permanent solution by means of a flexible facing on the basis of high tensile steel wire mesh anchored to the slope face with soil nails or rock bolts. This state of the art solution is applicable for all natural slopes as well as all fresh slope cuts. The suitability of the specified systems and their components shall be proven by full scale field tests under real conditions. Prove of consideration of interaction of the process and system for the particular project-conditions must be given. Respective documentation by an independent institute has to be provided.

The bearing resistances of the surface protection in respect of selective tensile strains parallel to the slope and pressure strains in nail direction have to be proven. The connection of two panels must guarantee at least an equal force transmission as the mesh itself. It is compulsory to submit the static proofs for the stability calculation of the system. For this the calculation concept needs to be validated by an independent institute.

Specification of those Component and System is as below:

Table 1-5: ***Material Requirements and Components af an Active System.***

COMPONENTS & PROPERTIES	SPECIFICATIONS			
WIRE				
Mesh geometry	rhomboid			
Wire coating	95% Zn / 5% Al (based on EN 10244-2 Class B) min. 150 g/m2			
Tensile strength of steel wire	1’770 N/mm2 (based on EN 10264-2)			
Wire diameter	2 mm Ø	3 mm Ø	4 mm Ø	6.5 mm Ø, 1 x 3 à 3mm stranded
Inner circle diameter width in mm	65	65	65	143
Accelerated weathering (salt spray test), according EN ISO 9227	1’508 hrs. until 5% dbr	1’856 hrs. until 5% dbr	1’467 hrs. until 5% dbr	1’856 hrs. until 5% dbr
Minimum bearing resistance of meshes in KN				
- Against puncturing	80	180	280	230
- Against shearing-off	40	90	140	115
- Against slope parallel tensile stress	10	30	50	45
SYSTEM SPIKE PLATE				
Geometry and size	diamond shape: 7 mm (thickness) x 205 mm (width) x 330 mm (length) Or diamond shape: 7 mm (thickness) x 288 mm (width) x 665 mm (length)			
Corrosion protection	Hot dip galvanized based on EN ISO 146, min. layer thickness 55 µm			
CONNECTION ELEMENTS				
wire diameter	4 mm		shackles 3/8”	

COMPONENTS & PROPERTIES	SPECIFICATIONS	
tensile strength of steel wire	1’770 N/mm2 (based on EN 10264-2)	
corrosion protection	95% Zn / 5% Al (based on EN 10244-2 class B)	hot dip galvanized
SOIL NAILS / ROCK BOLTS		
Diameter	Up to Ø 40mm, black, galvanized or duplex coated	
Lenth	subject to design calculations	

1.3.2.2 Passive Systems

Passive systems are those which do not affect the process of rock detachment, example of such systems are Rock Fall Barriers, Drapery Attenuators. The design of the systems shall used based on the property of using high tensile steel wires having minimum tensile strength of 1770 N/mm^2. They have the aim to intercept, retain or guide falling rocks and sliding debris. Types of passive systems are shown in the table below.

Table 1-6: Material Requirements and Components of a Passive System

SYSTEMS , COMPONENTS AND PROPERTIES	SPECIFICATIONS			
TYPE OF APPLICATION	DRAPERY OR ATTENUATORS			
Mesh and net geometry	Rhomboid			
Wire coating	95% Zn / 5% Al (based on EN 10244-2 Class B) min. 150 g/m2			
Tensile strength of steel wire	1’770 N/mm2 (based on EN 10264-2)			
Wire diameter	2 mm Ø	3 mm Ø	4 mm Ø	6.5 mm Ø, 1 x 3 à 3mm stranded
Inner circle diameter width in mm of mesh	65	65	65	143 or 275
Accelerated weathering (salt spray test), according EN ISO 9227	1’508 hrs. until 5% dbr	1’856 hrs. until 5% dbr	1’467 hrs. until 5% dbr	1’856 hrs. until 5% dbr
Component of a drape system	Mesh, wire rope anchors, top support ropes, seam wire rope, lateral boundary rope, bottom support rope, wire rope clips, shackles			
TYPE OF APPLICATION	ROCKFALL BARRIERS			
Energy class in kJ(Kilo Joule)	100, 500, 1000, 2000, 3000, 5000, 8000			

SYSTEMS , COMPONENTS AND PROPERTIES	SPECIFICATIONS			
Approval FOEN Guideline for the approval of rockfall protection kits", Environment in practice. Swiss Agency for the Environment, Forests and Landscape (SAEFL), Swiss Federal Research Institute WSL.	Yes			
Approval ETAG 027 Class A Guideline for European Technical Approval of Falling Rock Protection Kits	Yes			
Mesh or Net geometry	Rhomboid or multiple ringnet (4 times interlinked)			
Wire coating	95% Zn / 5% Al (based on EN 10244-2) Min. 150 g/m2			
Tensile strength of steel wire	1'770 N/mm2 (based on EN 10264-2)			
Wire diameter	2 mm Ø	3 mm Ø	4 mm Ø	8.6 mm Ø, 1 x 3 à 4mm stranded
Innercircle diameter width in mm of rhomboid mesh	80	80	80	143
Accelerated weathering (salt spray test), according EN ISO 9227	1'508 hrs. until 5% dbr	1'856 hrs. until 5% dbr	1'467 hrs. until 5% dbr	1'467 hrs. until 5% dbr
Diameter width in mm of ringnet Number of wire windings out of single wire 3 mm ø	300 or 350 dia 7 / 12 / 16 /19 (no of Windings)			
Post type	As per manufacturer specification			
Distance between posts	Subject to design calculations as well as approval			
Post height				
Rope diameter max				
Barrier height				
Corrosion protection	Wires see above; steel parts hot dip galvanized			
Components of rockfall barrier	Posts, mesh or nets, brake elements, support ropes (top, bottom, lateral and retaining ropes), anchor base plates, running wheel, shackles, etc.)			

SYSTEMS , COMPONENTS AND PROPERTIES	SPECIFICATIONS
TYPE OF APPLICATION	DEBRIS FLOW BARRIERS
Pressure class in kN/mhfl-1	60, 80, 100, 120, 140, 160, 180
1:1 field tests	The suitability of the specified systems and their components shall be proven by full scale field tests under real debris flow conditions. Prove of consideration of interaction of the process and system for the particular project-conditions must be given. Respective documentation by an independent institute has to be provided:
Net	Multiple ring net (4 times interlinked)
Wire coating	95% Zn / 5% Al (based on EN 10244-2) Min. 150 g/m2
Tensile strength of steel wire	1’770 N/mm2 (based on EN 10264-2)
Diameter width in mm of ring net Number of wire windings (single wire 3 mm ø)	300 7 / 12 / 16 (Number of Windings)
Post type	As per manufacturer specification
Distance between posts	Subject to design calculations
Post height	
Rope diameter max	
Barrier height	
Corrosion protection	Wires see above; steel parts galvanized
Components of debris flow barriers	Posts, ring nets, brake rings, support ropes (top, bottom, middle), boarder ropes, retaining ropes, spiral rope anchors, abrasion protection, shackles, etc.

High Tensile Steel Wire

High Tensile wire used for rock fall protection and slope stabilization must comply to ASTM A 370 – 97 with minimum tensile strength of 1700 N/mm².

Table 1-7: Wire diameter and tolerances.

Wire Diameter	Application	Mitigation measure
3 mmØ	Active	Slope stabilization
4 mmØ	Active	Slope stabilization
3 mmØ x 300 mmØ x 16 to 19 strands	Passive	Rockfall protection
3 mmØ x 300mmØ x 7 to 12 strands	Passive	Debris flow protection

Corrosion Protection

The High Tensile wire used for rock-fall protection and slope stabilization must comply with following specification.

COMPOUND: 95% Zn and 5% Al. or Superior Coating shall be used.
ASTM A 123

Accelerated Weathering Test

1 SO_2 Spray Test (DIN 50018)
2 NaCl Spray Test (DIN 50021)

Certifications and Field Tests

All systems specified above and in the following shall be tested and certified by independent accredited institutions and additionally approved by governmental bodies or principals. The materials, protection systems and services are based on state of the art technology and knowledge. Hence, the contractor shall comply with Certification required by their principals.

Design and Installation

Natural hazard mitigation is a very complex field. The geotechnical as well as geological/ hydrological situation can vary within meters. Therefore it is compulsory to address in particular to each problem A well prepared risk assessment followed by a detailed analysis and an engineered design will provide a safe, sustainable and economic solution. The protection systems shall be designed based on the site conditions in accordance with the manufacturer's requirements and the requirements specified in this specification.

The design calculations shall include:

a. Statement of all assumptions made and copies of all references for data used in the calculations.
b. Analyses demonstrating compliance with all applicable earth and water surcharges, seismic, or other loads.
c. Analyses or studies demonstrating durability and corrosion resistance of system for the proposed location and environment. The designer shall provide all corrosion protection devices necessary for the system to have a minimum service life of 100 years in the proposed location and environment.
d. The design calculations shall ensure that the slope has an adequate factor of safety with respect to both the internal stability of the reinforced soil and rock mass including anchor pull-out and anchor tensile strength, and overall external stability.
e. Design calculations that consists of computer program generated output shall provide thorough documentation of the sources of equations used and material properties.
f. External loads which affect the internal and external stability of the slope such as those applied through hydrostatic and seismic loads shall be accounted for in the design.

g. Since the design is based on the complex software and its output, the engineering behind application of the design of such system shall be well defined & explained. If possible to be supported by manual calculations.

Unforeseen issues during installation if not already taken into account during design, shall be part of the responsibility of the contractor. The manufacturer shall supply an installation manual for the respective system, describing in detail all steps of installation. Maintenance manuals shall be submitted after installation for systems requiring maintenance (i.e. rock fall and debris barriers).

Corrosion Protection Table as Per EN ISO 14713 – 1: 2009 (E)

Enviroment to be considered for Design and factor of safety to be assumed while considering corrosive enviroment. Such factors for wire shall be defined by Testing of Wire by reputed Institutes like TRI , USA or LGA , Germany or similar instituitions know for their credibility.

Table 1 — Description of typical atmospheric environments related to the estimation of corrosivity categories

Corrosivity category C Corrosion rate for zinc (based upon one year exposures), r_{corr} ($\mu m \cdot a^{-1}$) and corrosion level	Typical environments (examples)	
	Indoor	Outdoor
C1 $r_{corr} \leqslant 0,1$ Very low	Heated spaces with low relative humidity and insignificant pollution, e.g. offices, schools, museums	Dry or cold zone, atmospheric environment with very low pollution and time of wetness, e.g. certain deserts, central Arctic/Antarctica
C2 $0,1 < r_{corr} \leqslant 0,7$ Low	Unheated spaces with varying temperature and relative humidity. Low frequency of condensation and low pollution, e.g. storage, sport halls	Temperate zone, atmospheric environment with low pollution (SO_2 < 5 µg/m^3), e.g.: rural areas, small towns. Dry or cold zone, atmospheric environment with short time of wetness, e.g. deserts, sub-arctic areas
C3 $0,7 < r_{corr} \leqslant 2$ Medium	Spaces with moderate frequency of condensation and moderate pollution from production process, e.g. food-processing plants, laundries, breweries, dairies	Temperate zone, atmospheric environment with medium pollution (SO_2: 5 µg/m^3 to 30 µg/m^3) or some effect of chlorides, e.g. urban areas, coastal areas with low deposition of chlorides, subtropical and tropical zones with atmosphere with low pollution
C4 $2 < r_{corr} \leqslant 4$ High	Spaces with high frequency of condensation and high pollution from production process, e.g. industrial processing plants, swimming pools	Temperate zone, atmospheric environment with high pollution (SO_2: 30 µg/m^3 to 90 µg/m^3) or substantial effect of chlorides, e.g. polluted urban areas, industrial areas, coastal areas without spray of salt water, exposure to strong effect of de-icing salts, subtropical and tropical zones with atmosphere with medium pollution
C5 $4 < r_{corr} \leqslant 8$ Very high	Spaces with very high frequency of condensation and/or with high pollution from production process, e.g. mines, caverns for industrial purposes, unventilated sheds in subtropical and tropical zones	Temperate and subtropical zones, atmospheric environment with very high pollution (SO_2: 90 µg/m^3 to 250 µg/m^3) and/or important effect of chlorides, e.g. industrial areas, coastal areas, sheltered positions on coastline
CX $8 < r_{corr} \leqslant 25$ Extreme	Spaces with almost permanent condensation or extensive periods of exposure to extreme humidity effects and/or with high pollution from production process, e.g. unventilated sheds in humid tropical zones with penetration of outdoor pollution including airborne chlorides and corrosion-stimulating particulate matter	Subtropical and tropical zones (very high time of wetness), atmospheric environment with very high pollution (SO_2 higher than 250 µg/m^3), including accompanying and production pollution and/or strong effect of chlorides, e.g. extreme industrial areas, coastal and offshore areas with occasional contact with salt spray

During the joint inspection of the existing project road by representatives of FIPL and NHIDCL following stretches were observed to have undergone distress in form of sinking. The slopes on hill as well valley side have, however, been reported to be intact without any signs of instability.

S.No.	Chainage	
	From	To
1	31+900	32+100
2	37+100	37+400

Presently, there are no structural pavement layers in existing road and only subgrade is visible. There is no side drain. As such the subgrade is open to water ingress and consequent weakening thereof. Increase in the moisture content of the subgrade reduces its strength and bearing capacity. Also, there are fair chances of the underflow of water beneath the subgrade at the locations where distress has been noticed possibly on account of washing away of fines of subgrade material along with the seepage water.

The entire project road has been already been recommended for reconstruction. Provision should be kept for identifying all such distressed locations and landslide prone stretches. At vulnerable locations, proper geotechnical / material investigation in form of trial pits or boreholes may be necessary during construction to find out the sub soil properties and the extent of permeable & impermeable layers of sub-strata. Besides, the locations of cross drainage structures should be fixed. An elaborate drainage system consisting of but not limited to side surface drains, trench drains, catch water drains and proper arrangement for inletting of water from hill slopes in upstream side which passes safely through culverts should be provided.

As an interim measure, in sinking areas, the provision of 1.2m dia. pipe culverts spaced at 20-30m c/c to facilitate the drainage of seepage water from the hill slope with provision of side drains and collecting pit can be made.The settled road stretches should be made good by providing and roller compacting good subgrade soil mixed with gravel.

1.3.3 Erosion Protection

1.3.3.1 Seeding and Mulching

It consists of preparing slopes, placing topsoil, furnishing seeds, fertilisers and mulching materials, jute netting, coir netting or polymer netting and incorporating the same on slopes.

Mulching materials may consist of straw, hay, wood shavings, or sawdust in dry condition suitable for placing with a munch blower and free from weed seed and such organic materials as may detract their effectiveness as a mulch or be injurious to plant growth.

The top soil should be free from noxious weeds, with 2-12 % organic matter and duly tested by appropriate agricultural authority for presence of any residuals of herbicide or sterilents.

A suitable grade of bituminous emulsion free from any solvent or diluting agent toxic to plant life may be used as a tie down for mulch.

The netting may be of jute netting consisting of undyed jute yarn woven into a uniform open weave with approximate 25mm square openings. Black or green geo-netting weighing not less than 3.8kg/1000sqm. may be made of uniformly extruded rectangular mesh having mesh opening of 20mm x 20mm. Alternatively, a layer of biodegradable mulching materials sandwiched between 2 layers of polymer netting or non-woven coconut fibre coir netting may also be used.

The area to be seeded is brought to the required slope and cross-section by filling, reshaping eroded areas and refinishing slopes, Topsoil should be evenly spread over the area to the required depth. All live plants should be eliminated by suitable means using agricultural implements and all stones 150 mm and larger should be removed. The soil shall then be excavated on the contour to a depth of 100mm and all clods larger than 25mm to be crushed and packed. Where necessary, water should be applied. All topsoil should be compacted by slope compacter, cleated tractor or similar equipment as approved by Engineer so as to produce a uniform rough textured surface ready for seeding and mulching which will bond the topsoil to the underlying material. The entire area should be covered by a minimum of 4 passes of compacting equipment. The fertiliser to the required quantities shall be spread and thoroughly incorporated into the soil surface as part of seed-bed preparation.

All seeds should be planted uniformly. Immediately after sowing seeds, the area should be raked, dragged or otherwise treated so as to cover the seeds to a depth of 6mm. Seeding operation should not be performed when the ground is muddy or when the soil/whether conditions would otherwise prevent proper soil preparation and subsequent operations.
Soil moisture should exist throughout the zone from 25mm to at least 125mm below the surface at the time of planting. Watering of the seed areas should be done as per directions of Engineer.

Within 24 hours of seeding, mulching material mixed with organic manure should be placed so as to form a continuous, unbroken cover of approximate uniform thickness of 25mm using an acceptable mechanical blower. Mulching material should be held in place and made resistant to being blown away by suitable means approved by engineer. If required, the mulch material should be anchored in place with bituminous emulsion applied at the rate of 2300 litres per hectare. Any mulch disturbed or displaced following application should be removed, resseded and re-mulched as specified. Jute netting/geonetting/coir netting should be unrolled and placed parallel to the flow of water immediately following the bringing, to the finished grade, the area specified on the drawings or placing of seed and fertiliser. Where more than one strip is required to cover the given areas, they should overlap a minimum of 100mm. Netting should be held in placve by approved wire staples, pins, spikes or wooden stakes driven vertically into the soil.

1.3.3.2 Hydro-seeding

It is a form of Bio-engineering which pursues technological, ecological, economic as well as design goals and seeks to achieve these primarily by making use of living materials, i.e.

seeds, plants, part of plants and grassing by using hydro seeding technique. Hydro seeding is an effective tool for treatment of a variety of unstable and / or eroding sites. Hydro seeding is now widely practiced throughout the world for the treatment of erosion and unstable slopes.

Functions and Effects of Hydro seeding

Technical functions

- Protection of soil surface from erosion by wind, precipitation, frost and rain water
- Protection from rock fall
- Elimination or binding of destructive mechanical forces
- Reduction of flow velocity along banks
- Surface and/or deep soil cohesion and stabilization
- Drainage
- Protection from wind
- Aiding the deposition of snow drift sand and sediments
- Increasing soil roughness and thus preventing avalanche release

Apart from these, ecological functions are gaining an importance, particularly as these can be fulfilled to a very limited extent only by classical engineering constructions.

Ecological functions

- Improvement of water regime by improved soil interception and storage capability as well as water
- Consumption by plants
- Soil drainage
- Protection from wind
- Protection from ambient air pollution
- Mechanical soil amelioration by the roots of grass.
- Balancing of temperature conditions in near–ground layers of air and in the soil
- shading
- Improvement of nutrient content in the soil and thus of soil fertility on previously raw Soil
- Balancing of snow deposits
- Noise protection
- Yield increase on neighboring cropland
- Providing forage grasses for farm animals & wildlife.
- Increase revenue for surrounding persons living in adjoining villages

Landscaping functions

- healing of wounds inflicted on the landscape by disasters and humans (exploitation of mineral resources, construction work, deposition of overburden, tunnel excavation material, industrial and domestic waste) •integration of structures into the landscape

- concealment of offending structures
- enrichment of the landscape by creating new features and structures, shapes and colors of vegetation

Economic effects

Hydro seeding works are not always necessarily cheaper in construction when compared to classical engineering structures, however, they provide additional benefits. However, when taking into account their lifetime including their service and maintenance, they will normally turn out to be more economical. Their special advantages are:

- lower construction costs compared to "hard" constructions
- lower maintenance and rehabilitation costs
- Creation of useful green areas and woody plant populations on previously derelict land.
- Useful for income generation to surrounding villages by providing a free and permanent source of fresh forage grasses, thereby increasing more farm animals & generation of revenue for the surrounding villages.
- Beautification of the area.

The result of Hydro seeding protection works are living systems which develop further and maintain their balance by natural succession (i.e. by dynamic self–control, without artificial input of energy). If the right living but also non–living building materials and the appropriate types of construction are chosen, exceptionally high sustainability requiring little maintenance effort can be achieved.

The up gradation of these roads by modern civil engineering techniques and its further strengthening by environment friendly and sustainable Hydro seeding measures will certainly uplift the quality of life of the rural people residing near this road.

Although Hydro seeding costs more in the short term than the „do nothing" approach, in the long term there would be additional benefits from reduced maintenance costs.

1.4 IMPROVEMENT PROPOSALS AND DESIGNS OF STRUCTURES

1.4.1 Design Metodology For Structures

1.4.1.1 Foundation

Detailed methodlologyadoptedfor selection of foundation for various structures are discussed in following section:

1.4.1.2 For Short Span Bridges & Culverts

In soils having limited load carrying capacity at a shallow depth for very small structures like culverts and Minor Bridges with low discharge, it is recommended to adopt single or multi-cell closed box type of structure with adequate bed protection against scour wherever necessary

1.4.1.3 Medium and Long Span Bridges

In general, where hard strata is met with at shallow depth, open foundation shall be provided. In case the safe bearing capacity of sub-soil is reasonably good and scour depth is low, open foundation with isolated footing is provided. In situations where both the above solutions are not feasible, pile or well foundation may be provided

1.4.1.4 Open Foundation

In general, the design of open foundation conforms to provisions of Cl: 707 of IRC: 78–2014 and Bearing capacity and settlement criterion shall be checked with IS: 6403 and IS: 8009 for ultimate bearing capacity and allowable settlement. The various specific assumptions for the design of open foundation shall be as follows:

- The maximum base pressure at corner of the open foundation shall be checked with Bearing capacity of the underneath soil. The following limiting values shall be considered for computation of safe load:
- Results of sub-soil investigation shall be used for adopting the value of angle of internal friction “Φ” and cohesion “C” of the soil.
- The overburden soil shall be considered as dry or partially submerged or fully submerged for the purpose of calculation of safe bearing capacity.
- Depth of influence of soil strata can be assumed 1.5 times the width of foundation.

1.4.1.5 Structure Proposals:

On the basis of methodology presented in previous section, the structure proposals for the Project Highway are provided herein below:

Minor Bridge

S. No.	Design Chainage (Km)	Type of Existing Structure	Existing Span Arrangement (m)	Skew	Existing Total Width (m)	Proposed Span Arrangement (m)	Proposed Structure Type	Proposed Total Width (m)	Remarks
1	37+027	PSC Box Girder	1 x 53		8.50	Existing Retained with Repairing & Maintenance			
2	38+165		1 x 43		Under Construction				

Culvert

S. No.	Design Chainage (km)	Existing Span Arrangement (m)	Type of Existing Structure	Existing Total Width (m)	Proposed Span Arrangement (m)	Proposed Structure Type	Proposed Total Width (m)	TCS Type	Remarks
1	29+820	-	-	-	1x2x1.5	RCC Box	11.1	4	New-Construction
2	29+910	-	-	-	1x2x1.5	RCC Box	11.1	4	New-Construction
3	30+037	1 x 1.0	Pipe Arch	7.80	1x2x1.5	RCC Box	11.1	4	Reconstruction
4	30+209	-	-	-	1x2x1.5	RCC Box	11.1	4	New-Construction
5	30+510	1 x 1.0	Pipe Arch	7.70	1x2x1.5	RCC Box	11.1	4	Reconstruction
6	30+917	1 x 1.0	Pipe Arch	8.20	1x2x1.5	RCC Box	11.1	4	Reconstruction
7	31+109	-	-	-	1x2x1.5	RCC Box	11.1	4	New-Construction
8	31+241	1 x 2.5	Slab	6.70	1x2x1.5	RCC Box	11.1	4	Reconstruction
9	31+368	-	-	-	1x2x1.5	RCC Box	11.1	4	New-Construction
10	31+502	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
11	31+583	1 x 1.5	Slab	7.00	1x2x1.5	RCC Box	10.4	3	Reconstruction
12	31+692	1 x 2	Slab	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
13	31+864	1 x 1.0	Pipe Arch	9.90	1x2x1.5	RCC Box	10.4	3	Reconstruction
14	31+930	1 x 1.0	Pipe Arch	9.20	1x2x1.5	RCC Box	10.4	3	Reconstruction
15	32+200	1 x 1.0	Pipe Arch	8.80	1x2x1.5	RCC Box	10.4	3	Reconstruction
16	32+338	1 x 1.0	Pipe Arch	9.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
17	32+470	1 x 1.0	Pipe Arch	8.90	1x2x1.5	RCC Box	10	2	Reconstruction
18	32+559	1 x 1.0	Pipe Arch	9.00	1x2x1.5	RCC Box	10.4	3	Reconstruction
19	32+930	-	-	-	1x2x1.5	RCC Box	10	2	New-Construction
20	33+170	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
21	33+319	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction

S. No.	Design Chainage (km)	Existing Span Arrangement (m)	Type of Existing Structure	Existing Total Width (m)	Proposed Span Arrangement (m)	Proposed Structure Type	Proposed Total Width (m)	TCS Type	Remarks
22	33+430	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10.4	3	Reconstruction
23	33+490	1 x 1.5	Slab	6.10	1x1.5x1	RCC Box	10.4	3	RHS Widening
24	33+706	1 x 2	Slab	7.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
25	33+767	1 x 1.0	Pipe Arch	8.40	1x2x1.5	RCC Box	10.4	3	Reconstruction
26	34+016	1 x 2	Slab	7.00	1x2x1.5	RCC Box	10.4	3	Reconstruction
27	34+137	1 x 1.5	Slab	7.40	1x2x1.5	RCC Box	10.4	3	Reconstruction
28	34+540	1 x 1.0	Pipe Arch	9.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
29	34+753	1 x 1.5	Pipe Arch	8.95	1x2x1.5	RCC Box	10.4	3	Reconstruction
30	34+940	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
31	35+163	1.500	Slab	7.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
32	35+378	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
33	35+511	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10.4	3	Reconstruction
34	35+680	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
35	35+900	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
36	36+003	1 x 1.0	Pipe Arch	8.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
37	36+205	1 x 1.0	Pipe Arch	8.70	1x2x1.5	RCC Box	10	2	Reconstruction
38	36+269	1 x 2.5	Slab	7.60	1x2x1.5	RCC Box	10	2	Reconstruction
39	36+465	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
40	36+790	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
41	37+307	1 x 1.5	Slab	7.20	1x2x1.5	RCC Box	10.4	3	Reconstruction
42	37+444	1 x 1.0	Slab	7.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
43	37+715	1 x 1.0	Pipe Arch	7.80	1x2x1.5	RCC Box	10.4	3	Reconstruction
44	37+929	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
45	38+333	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
46	.	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10.4	3	Reconstruction

S. No.	Design Chainage (km)	Existing Span Arrangement (m)	Type of Existing Structure	Existing Total Width (m)	Proposed Span Arrangement (m)	Proposed Structure Type	Proposed Total Width (m)	TCS Type	Remarks
47	38+506	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10.4	3	Reconstruction
48	38+643	1 x 1.0	Pipe Arch	6.70	1x2x1.5	RCC Box	10.4	3	Reconstruction
49	38+777	1 x 1.0	Pipe Arch	8.10	1x2x1.5	RCC Box	10.4	3	Reconstruction
50	39+003	-	-	-	1x2x1.5	RCC Box	10	2	New-Construction
51	39+505	1 x 1.0	Pipe Arch	6.30	1x2x1.5	RCC Box	10	2	Reconstruction
52	39+761	1 x 1.0	Pipe Arch	9.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
53	39+871	2x1.200	Pipe Arch	9.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
54	39+883	1 x 2	Slab	7.60	1x2x2	RCC Box	10.4	3	RHS Widening
55	40+097	-	-	-	1x2x1.5	RCC Box	10	2	New-Construction
56	40+400	-	-	-	1x2x1.5	RCC Box	10	2	New-Construction
57	40+582	1 x 1.0	Pipe Arch	7.20	1x2x1.5	RCC Box	10	2	Reconstruction
58	40+737	1 x 1.0	Pipe Arch	6.65	1x2x1.5	RCC Box	10	2	Reconstruction
59	41+003	1 x 1.0	Pipe Arch	7.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
60	41+224	1 x 1.0	Pipe Arch	6.70	1x2x1.5	RCC Box	10	2	Reconstruction
61	41+314	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10	2	Reconstruction
62	41+532	1 x 1.0	Slab	7.25	1x2x1.5	RCC Box	10.4	3	Reconstruction
63	41+608	1 x 1.0	Pipe Arch	5.90	1x2x1.5	RCC Box	10.4	3	Reconstruction
64	41+979	1 x 1.0	Pipe Arch	9.17	1x2x1.5	RCC Box	10	2	Reconstruction
65	42+066	1 x 2.5	Slab	7.15	1x2x1.5	RCC Box	10	2	Reconstruction
66	42+111	1 x 1.0	Pipe Arch	7.00	1x2x1.5	RCC Box	10	2	Reconstruction

S. No.	Design Chainage (km)	Existing Span Arrangement (m)	Type of Existing Structure	Existing Total Width (m)	Proposed Span Arrangement (m)	Proposed Structure Type	Proposed Total Width (m)	TCS Type	Remarks
67	42+500	1 x 1.0	Pipe Arch	7.75	1x2x1.5	RCC Box	10	2	Reconstruction
68	42+662	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10	2	Reconstruction
69	42+775	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10	2	Reconstruction
70	42+808	1 x 1.0	Pipe Arch	7.55	1x2x1.5	RCC Box	10	2	Reconstruction
71	43+109	1 x 1.0	Pipe Arch	7.10	1x2x1.5	RCC Box	10.4	3	Reconstruction
72	43+397	1 x 1.5	Slab	8.80	1x2x1.5	RCC Box	10.4	3	Reconstruction
73	43+691	1 x 1.0	Pipe Arch	7.87	1x2x1.5	RCC Box	10	2	Reconstruction
74	43+721	1 x 1.0	Pipe Arch	9.15	1x2x1.5	RCC Box	10	2	Reconstruction
75	43+869	1 x 1.0	Pipe Arch	8.67	1x2x1.5	RCC Box	10	2	Reconstruction
76	44+008	1 x 1.0	Pipe Arch	7.00	1x2x1.5	RCC Box	10	2	Reconstruction
77	44+251	1 x 1.0	Pipe Arch	7.85	1x2x1.5	RCC Box	10.4	3	Reconstruction
78	44+362	1 x 1.5	Slab	7.67	1x2x1.5	RCC Box	10.4	3	Reconstruction
79	44+411	1 x 1.0	Pipe Arch	7.00	1x2x1.5	RCC Box	10.4	3	Reconstruction
80	44+527	1 x 1.5	Slab	7.90	1x2x1.5	RCC Box	10	2	Reconstruction
81	44+644	1 x 1.0	Pipe Arch	8.00	1x2x1.5	RCC Box	10	2	Reconstruction
82	44+690	1 x 1.0	Pipe Arch	7.35	1x2x1.5	RCC Box	10	2	Reconstruction
83	44+842	1 x 1.0	Pipe Arch	7.92	1x2x1.5	RCC Box	10.4	3	Reconstruction
84	44+985	1 x 1.0	Pipe Arch	7.39	1x2x1.5	RCC Box	10.4	3	Reconstruction

S. No.	Design Chainage (km)	Existing Span Arrangement (m)	Type of Existing Structure	Existing Total Width (m)	Proposed Span Arrangement (m)	Proposed Structure Type	Proposed Total Width (m)	TCS Type	Remarks
85	45+116	1 x 0.9	Pipe Arch	7.40	1x2x1.5	RCC Box	10.4	3	Reconstruction
86	45+158	1 x 1.0	Pipe Arch	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
87	45+188	1 x 1.0	Pipe Arch	7.40	1x2x1.5	RCC Box	10.4	3	Reconstruction
88	45+285	1 x 1.0	Pipe Arch	6.83	1x2x1.5	RCC Box	10.4	3	Reconstruction
89	45+394	1 x 1.0	Pipe Arch	6.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
90	45+480	1 x 1.0	Pipe Arch	7.80	1x2x1.5	RCC Box	10.4	3	Reconstruction
91	45+780	1 x 1.0	Pipe Arch	7.50	1x2x1.5	RCC Box	10	2	Reconstruction
92	46+048	1 x 1.0	Pipe Arch	7.50	1x2x1.5	RCC Box	10	2	Reconstruction
93	46+210	1 x 1.0	Slab	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
94	46+263	1 x 1.0	Slab	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
95	46+293	1 x 1.0	Pipe Arch	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
96	46+495	1 x 2	Slab	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
97	46+575	1 x 1.5	Slab	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
98	46+742	1 x 1	Slab	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
99	47+000	1 x 1.5	Slab	7.10	1x2x1.5	RCC Box	10.4	3	Reconstruction
100	47+248	1 x 1.5	Slab	6.80	1x2x1.5	RCC Box	11.1	4	Reconstruction
101	47+680	-	-	-	1x2x1.5	RCC Box	11.1	4	New-Construction
102	47+913	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
103	48+197	1 x 1	Pipe Arch	5.40	1x2x1.5	RCC Box	10.4	3	Reconstruction
104	48+314	1 x 1.5	Slab	8.05	1x2x1.5	RCC Box	10.4	3	Reconstruction
105	48+562	1 x 1.0	Pipe Arch	9.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
106	48+706	1 x 1.0	Pipe Arch	7.90	1x2x1.5	RCC Box	10.4	3	Reconstruction

S. No.	Design Chainage (km)	Existing Span Arrangement (m)	Type of Existing Structure	Existing Total Width (m)	Proposed Span Arrangement (m)	Proposed Structure Type	Proposed Total Width (m)	TCS Type	Remarks
107	48+762	1 x 1.0	Slab	8.40	1x2x1.5	RCC Box	10.4	3	Reconstruction
108	48+987	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
109	49+230	-	-	-	1x2x1.5	RCC Box	10.4	3	New-Construction
110	49+607	1 x 1.0	Pipe Arch	8.10	1x2x1.5	RCC Box	10.4	3	Reconstruction
111	49+762	1 x 1.0	Pipe Arch	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
112	49+935	1 x 1.0	Pipe Arch	7.95	1x2x1.5	RCC Box	10.4	3	Reconstruction
113	50+150	1 x 1.0	Pipe Arch	6.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
114	50+200	1 x 1.0	Pipe Arch	8.25	1x2x1.5	RCC Box	10.4	3	Reconstruction
115	50+290	1 x 0.9	Pipe Arch	7.45	1x2x1.5	RCC Box	10	2	Reconstruction
116	50+383	1 x 1.0	Pipe Arch	7.50	1x2x1.5	RCC Box	10.4	3	Reconstruction
117	50+693	1 x 1.0	Pipe Arch	6.90	1x2x1.5	RCC Box	10.4	3	Reconstruction
118	50+843	1 x 1.0	Pipe Arch	7.27	1x2x1.5	RCC Box	10	2	Reconstruction
119	50+931	1 x 1.0	Slab	7.25	1x2x1.5	RCC Box	10.4	3	Reconstruction
120	50+990	1 x 1.0	Pipe Arch	6.30	1x2x1.5	RCC Box	10.4	3	Reconstruction
121	51+093	1.000	Pipe Arch	7.25	1x2x1.5	RCC Box	10.4	3	Reconstruction
122	51+200	1 x 1.0	Pipe Arch	6.30	1x2x1.5	RCC Box	10	2	Reconstruction
123	51+388	1 x 1.0	Pipe Arch	6.60	1x2x1.5	RCC Box	10	2	Reconstruction
124	51+499	1 x 1.0	Slab	6.40	1x2x1.5	RCC Box	10	2	Reconstruction

Minor Bridge is retained with minor repairs.

The proposed culverts are grouped and standard design of same is done for following cases (refer in Annexure A):

- Box Culvert of size 1x1.5x1.5 with maximum 1m cushion
- Box Culvert of size 1x2.0x2.0 with maximum 600mm cushion