STANDARDS/MANUALS/GUIDELINES FOR SMALL HYDRO DEVELOPMENT

Civil Works –
Guidelines for Structural Design of SHP Projects

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CHAPTER-I

CIVIL WORKS FOR SHP PROJECTS:
GUIDELINES FOR STRUCTURAL DESIGN

INTRODUCTION

1. General

The SHP projects can be of River-off type or Canal type. The structures to be designed will depend upon the type of project. Typical Layout of River-off project giving necessary structures is shown in fig. 1 and for Canal type project is given in fig. 2. The structures to be designed will be using either R.C.C. / PCC or steel. The relevant standards with the latest versions are to be used. Limit state design philosophy will be used in general, unless otherwise specified. For water retaining structures No crack basis of Design will be used. Earthquake resistant design for earthquake prone areas is necessary.

2. Structures for River-off type Small Hydro Projects

(a) Intake Structures.
(b) Feeder and Power Channels.
(c) Cross Drainage works.
(d) Settling Basin - Desilting Tank.
(e) Forebay Tank
(f) Penstock Supports.
(g) Power Housing Building.
(h) Machine Foundations.
(i) Tail Race Channel.

3. Structures for Canal type Small Hydro Projects

(a) Canal
(b) Power House.
(c) Bridge, if any

4. Basic Data for Structural Design

(a) Soil Properties: Angle of Repose (\(\phi\)), Coefficient of Friction (\(\mu\)), Bearing Capacity, Density, Details of Soil Strata at various depths. Ground water level
(b) Earthquake related Data: Importance factor, Zone of earthquake.
(c) M 20 Concrete and Fe 415 reinforcement will be used in general for R.C.C. work, unless otherwise specified, and 1:3:6 Mix will be used for P.C.C.

5. References: Appendix I

(a) Codes
(b) Books
6. The detailed design requirements/ details for each structure depending upon type of Small Hydro Project are given in subsequent chapters.

Fig 1:

Fig 2:
CHAPTER II

DESIGN DETAILS FOR STRUCTURES FOR RIVER – OFF TYPE SHP PROJECTS

1. **INTAKE STRUCTURES**: The Intake Structures comprise of following elements:

   (see fig. 3)

   (a) Trench Weir
   (b) Cut off Walls.
   (c) Intake Chamber
   (d) Abutments.

**Trench Weir**

This structure is built across the river bed and therefore runs a risk of heavy boulder striking during floods, even though full pitching is done on both sides up to the top of weir. It is a normal practice to provide thicker walls. The design is nominal with nominal skin reinforcement as per IS:3370 Cl. 7.1. A typical section is shown in Fig. 3.

**Cut off Walls**

In all three cut off walls are to be provided, first on upstream side of Trench weir and two on downstream side. All these walls are fully buried in the ground and therefore soil pressure from the either side would almost neutralize. Nominal sections and nominal reinforcement are therefore provided. Typical details of a cut off wall are given in Fig. 3.

**Intake Chamber**

A key plan of the intake chamber along with one typical section is shown in fig 4. The four vertical walls and top and bottom slabs are monolithic. The roof slab has a manhole opening. The walls and slabs are designed as per available dimensions and boundary conditions for water pressure / soil pressure for worst conditions on no crack basis. In this chamber wall A has a large opening almost extending to full width of the chamber where trench weir joins the chamber. Similarly all B has an opening for the outlet to feeder channel. This opening is extending to about little less than half the width of wall B. These large openings would change the behavior of these walls in particular and hence should be accounted for in the analysis.

The moments in individual walls are obtained using coefficients from Reynolds handbook (27). A typical calculation of design moments in wall C is given below:

Wall C: dimensions 2.9 m x 5.85 m
Height of standing water = 4.95m

\[ l_x = 2.9 \text{ m}, \quad l_z = 5.85 \text{ m} \]

\[ k = \frac{l_x}{l_z} = 0.5 \]

max. moment in the wall using Coeff. from Reynold’s handbook.
Fig 4: Details of Intake Chamber
Vertical direction –
-ve moment at base = 0.012 x 49.5 x 5.85^2 = 20.3 kN m/m
-ve moment at top = 0.004 x 49.5 x 5.85^2 = 6.8 kN m/m
+ve moment near centre = 0.004 x 49.5 x 5.85^2 = 6.8 kN m/m

Horizontal direction –
-ve moment at edges = 0.05 x 49.5 x 2.9^2 = 29.8 kN m/m
+ve moment at midspan = 0.024 x 49.5 x 2.9^2 = 10.3 kN m/m

Tension in walls A&C = 24 wa^2 = 58.8 kN/m

Abutments

The stability of Abutment has to be checked for both conditions i.e. minimum water from inside and full earth pressure from out side, as well as maximum water from inside and earth pressure from outside. Normally Masonry Abutments are provided but if needed R. C. C. abutments may be provided.

A typical Design calculation with Stability Analysis is given below.

Design of Abutment

Considering 1 m length of the abutment, whose section is shown in fig. 5(a).

(i) Dry earth on outer side and no water on the other side (fig. 5(b)).
(ii) Maximum water pressure from inside and full earth pressure on outside. This is approximately equal to 50% water pressure from inside with no earth on the outer side. (fig 5(c)).

Case I – The forces acting are as shown
density of masonry = 21 kN / m^3
density of soil = 16.85 kN/m^3
Total vertical weigh, W = W_1 + W_2 + W_3

= 83.8 + 237.4 + 190.4
= 511.6 kN

Total overturning moment = 205.8 kN m
Restoring moment about B
= (83.8 x 0.3 + 23.74 x 1.733 + 190.4 x 2.87
= 983 kN m
F.O.S. against overturning = \frac{983}{205.8} = 4.78

F.O.S. against sliding = \frac{0.45 \times 205.8}{(100.6-18.5)} = 2.80
eccentricity where resultant strike the base

= \frac{(983-205.8)}{511.6} = 0.48 m < \frac{b}{6} OK
Soil Pressure

\[ \frac{511.6}{4} = \left( 1 \pm \frac{6 \times 0.48}{4} \right) \]

\[ = 220 \text{ and } 36.0 \text{ kN} / \text{m}^2 \]

**Case II** – The force acting are as shown

Restoring moment \( = 848 \text{ kN m} \)
Overturning moment \( = 165.0 \text{ kN m} \)

F.O.S. against over turning \( = 5.14 \)
F.O.S. against sliding \( = \frac{0.45 \times 321.2}{77.5} = 1.86 \)

Distance where resultant strike the base

\[ e = 2 - \frac{848 - 165}{321.2} \]

\[ = -0.086 \text{ m} < \frac{b}{6} \]

There is no uplift under the base.

---

**Fig 5: Stability Analysis of Abutment**
2. FEEDER CHANNEL AND POWER CHANNEL

(a) **Feeder Channel**: Feeder channel, if provided below ground level will be designed as a box section having side walls, base slab and top slab, monolithic. The side earth pressures, top vertical burden and uplift from below, as well as water pressure from inside will be considered along with self weights and various load combinations will be considered to consider worst effects. Typical section for feeder channel and design form is given in fig. 6.

(b) **Power Channel**: Power channel is usually of trapezoidal shape with or without top cover slab. The earth pressure from outside and water pressure from inside will be considered with worst combinations. Fig 7 shows typical section through power channel (uncovered section) and fig 8 shows typical section through power channel (covered section). If a covered section is to be provided, the top cover slab will be sloping and in that case the section may be designed as equivalent rectangular box section.

(c) **Cross Drainage Works**: Super Passages and Aqueducts have to be designed during the length of the power channel. For design of Super passage, a rolling load of boulder has to be considered and top slab will be separate and resting on existing power channel with dowels to avoid slipping. For design of Aqueduct, the level of top of abutment should be higher than water level and top of aqueduct should be designed for pedestrian traffic. The aqueduct is analyzed for both transverse and longitudinal bending and designed on no crack basis. The longitudinal analysis may be done by simple beam theory, limiting the maximum tensile stress to permissible bending stress. The transverse analysis may be done by moment distribution method and accordingly thickness and reinforcement are provided. Edge beams are to be provided at supports. The abutments and wing walls are to be provided at supports.

Typical details of Super passage & Aqueduct are given in fig 9. Fig 10 shows equivalent section to be adopted for simple beam analysis.
(a) CROSS-SECTION

(b) DESIGN FORCES

FIG. 6 - FEEDER CHANNEL

FIG. 7 - TYPICAL SECTION THROUGH POWER CHANNEL

FIG. 8 - SECTION THROUGH COVERED PORTION OF POWER CHANNEL
Fig 9:
Fig 10: Section of Aqueduct
3. DESILTING TANK (SETTLING BASIN)

General Arrangement

Desilting tank acts as a settling basin to remove the silt from the water entering the power channel. It is a R.C.C. rectangular basin open at top and is rectangular in shape with transitions both at the entrance and exit points. Water from feeder channel enters the tank through a transition. The splinter walls are provided (see fig. 11) to regulate the flow of water through the transition. Floor. Desilting tank is divided in to suitable no. of parts. Each part is made in the form of a hopper (figs. 12 & 13). Arrangement is made to flush out the silt from the bottom of the hopper base through a pipe. This arrangement is operated from outside the tank.

Design Philosophy

Desilting tank, being a water retaining structure will be designed on no crack basis. The walls are designed for earth as well as water pressures, while base slab should also consider uplift.

4. FOREBAY TANK

General Arrangement

Forebay Tank is a water retaining structure normally with two portions, one shallow and other deeper portion. The water from Desilting Tank enters the shallow portion and goes to Penstocks from deeper portion. The tank may be constructed above or below ground level and sometimes partly above ground level (fig 14).

Design Philosophy

It is designed as water retaining structure. Each wall is designed for bending in both horizontal as well as vertical direction depending upon aspect ratio and support conditions. Stability of the walls is checked as retaining walls with relevant forces. An expansion joint is provided for connecting these walls to base slab for which nominal thickness and reinforcements are provided. The walls are designed for both conditions viz. Tank full with dry earth pressure from outside and Tank empty with full earth pressure from outside. While considering earth pressures both static as well as Dynamic parts are to be considered for underground tanks. Forces to be considered on various walls are given in figs. 15 & 16.
Fig 11: Typical Details of Desilting Tank
Fig 12: Desilting Tank – General Arrangement
Fig 13: Details of floor of Desilting Tank
Fig 14: General Arrangement of Forebay Tank
Fig 15: Forces on Wall A
5. PENSTOCKS AND THEIR SUPPORTS

General Arrangement

The Penstocks carry water to Power House Building. The Diameter and thickness of Penstocks are designed from the considerations of Head of water and Amount of Water (Discharge), as well as spacing of Saddle supports. Normally Mild Steel Penstocks are used. The Penstocks are supported above ground surface on concrete blocks called Anchors/ Saddle supports. The anchors are provided at all horizontal and vertical bends along the alignment of the penstock. Saddle supports are provided along the straight length at regular intervals which will govern the thickness and diameter of penstock.

Design Philosophy

The forces acting on the anchor are computed as per IS; 5330 code and combinations of forces are considered for both expanding and contracting conditions for both Penstock full and Penstock empty conditions. The design of the anchor is done for over turning, sliding as well as maximum and minimum base pressures. The design of Saddle supports is also done to satisfy stability requirements of Overturning, sliding and maximum and minimum base pressures.

Figs. 17, 18, 19 and 20 give typical details of penstocks, (Plan & longitudinal section), anchor blocks and saddle supports respectively.
Fig 17: Typical Plan Showing Penstock & Anchor Blocks
Fig 18: Typical Longitudinal Section of Penstock
Fig 19: Typical Details of Anchor Block
Fig 20: Typical Details of Saddle Supports
6. POWER HOUSE BUILDING

General Arrangement

The Power House Building is normally a Reinforced Concrete Framed Structure, with the columns having isolated or combined footings. The Power House Building houses the Turbines and Generators. Roofing is normally of Corrugated sheets supported on sloping beams of the frame/ Roof Trusses. Service bay and Control Room portions may have R.C. C. Slab. Masonry walls/ Concrete Block walls are provided as filler walls to provide the enclosure. Walls are normally non load bearing elements. An Over head Travelling Crane is provided in the machine area, which moves on R.C. Gantry Girders supported on brackets attached to the columns.

Design Philosophy

The R.C. Frame of the Power House Building is analyzed using Stiffness approach for various loads and load combinations. Design of individual members is done using Limit State Design method. Design of C.G.I. sheets and steel Purlins is also done using relevant I.S. Standards. The Gantry Girder is designed using the available data, and the loads are transferred on the columns through brackets. The Frame is then analyzed using the software for various loads and their combinations. The wind effect is also transferred from Roof and walls. The earthquake effects are also considered in earthquake prone areas. The members are then designed including Columns and foundations. If part of the Power House Building is below ground level, the walls are designed as R.C. Retaining walls.

Figs. 21, 22 and 23 give typical details of Power House Building.
Fig 21: Typical Details of Power House Building Plan of Ground Floor Level
Fig 22: Section Showing Details of RC Frame
Fig 23: Typical Details of Power House Building
7. MACHINE FOUNDATIONS

General

Depending upon the type of Turbine, Rated capacity, Operating Speed, Runaway Speed and using the available data of loads from Turbine, Generator and the soil properties, the type of Machine foundation is decided. For a Block type foundation under a rotating type machine, the prime consideration of the foundation design is that Resonance has to be avoided. This requires that the natural frequency of the foundation should be 20% away from the generating frequency of the machine. Still further, if one of the natural frequency of the foundation is lower than the operating frequency, then during start up or shut down of the machine, the same would be crossed. A proper check has to be made for the build up of Amplitudes during the small fraction of time when natural frequency is crossed.

Design Philosophy

The dimensions of the block foundation are chosen and the computations for the natural frequency and the amplitude are done. The soil pressures are then checked. To reduce the cost sometimes hollow portions inside the block are filled by boulders.

Figs. 24 and 25 give typical details of machine foundation and tail race channel.

8. TAIL RACE CHANNEL

General

The tail race Channel may consist of some covered portions and some open portions to finally discharge the water on the downstream of the river.

Design Philosophy

The tail race channel in the covered portion is designed as box section subjected to various forces. The loads to be considered include u.d.l. from top, wt. of walls and top and bottom slabs, u.d.l. and triangular earth pressure from outside, water pressure from inside including wt. of water. The open sections are designed for maximum earth pressure from outside and the same thickness and reinforcement is usually provided on both faces of walls and base.
Fig 24: Plan of Machine Foundation
Fig 25: Typical Sections of Machine Foundation
CHAPTER III

DESIGN DETAILS FOR STRUCTURES FOR CANAL TYPE SHP PROJECTS

1. CANAL WALLS

General

Canal walls may be of following types:
1. Stand alone T-shaped.
2. Counter fort type.
3. Trough section.
4. Flared walls.

Load Conditions

2. Canal Full – Outside earth dry (always present)/ submerged.

Earth pressures should be active with surcharge (if any). Passive earth pressures are to be used where warranted. Account should be taken for submerged earth where necessary. Dynamic earth pressures should be considered where necessary as per IS: 1893 Cl. 8.2. Water pressure should include free board to accommodate worst condition during floods. Uplift pressure will be considered if water level exists below base of wall. Secondary effects Viz. Temperature, shrinkage, creep will be taken care by minimum reinforcement clause of relevant IS code.

The expressions to be used for earth pressures in various conditions are given below.

Loads:

Earth Pressure (Dry Earth)

\[
P = C_P \, w \, h
\]

Where \(C_P = \frac{1 - \sin \phi}{1 + \sin \phi}\)

\(\phi = \) Angle of repose

\(w = \) density of earth

with surcharge angle of \(\delta\)

\[
C_P = \cos \delta \cdot \frac{\cos \delta - \sqrt{\cos^2 \delta - \cos^2 \phi}}{\cos \delta + \sqrt{\cos^2 \delta + \cos^2 \phi}}
\]

Pressure due to uniform surcharge – where applicable
Passive earth pressure to be used where warranted

Dynamic Earth Pressure – IS 1893 Cl. 8.1
Earth Pressure (Submerged Earth)

Ref. JK & OPJ Vol I.

\[ P = w_1h + (w-w_1) - \frac{1-sin\phi'}{1+sin\phi'}h \]

\( w_1 = \text{wt. of water/ m}^3 \)
\( \phi' = \text{value of } \phi \text{ (Submerged)} \)

\[ C_p' = -\frac{1-sin\phi'}{1+sin\phi'}, \quad \phi' = \frac{2}{3} \phi \]

See figure below-

Dynamic effect – IS 1893 Cl 8.2

Water Pressure

\[ P = w_1h \]

\( w_1 = \text{density of water} \)

Note Include free board in \( h \) to accommodate worst condition during floods.

Self Weight

Concrete (reinforced) = 25 kN/m\(^3\)
Concrete (PCC) = 24 kN/m\(^3\)
Water = 10 kN/m\(^3\)
Compacted earth – as per soil report or 18 kN/m³ (assumed)
Earthquake effect on self wt. – IS 1893

**Design Philosophy**

Stand alone T- shaped – Check for stability
Sliding - \( \mu = 0.5 \), minimum FOS = 1.2, provide shear key if necessary.
Overturning- Minimum FOS = 1.5, Resultant within middle third- no uplift,
Maximum base pressure \( \leq \) Allowable ( soil report ), for seismic load combination allow increase by 33.3 %, Structural design of vertical slab, heel and toe for moment and shear.

Counter fort type –check for stability, structural design for vertical slab, heel, toe, front counter fort, back counter fort.

Trough Section – Base Slab continuous

Monolithic construction of slab and side walls, Filters to release uplift effect ( 50 % only ), Base slab design for moment + l force.

Trough Section- Base Slab Discontinuous

Walls treated as stand alone, Central portion of base treated separately, Provide wall projection out side for economy.

**Flared Walls**

Wall slope from vertical to 1 H: 1V- Design as stand alone, walls will be constructed in lifts of app. 1-2m, before constructing the next lift, compacted back fill will be placed behind the portion constructed.

Wall slope from 1 H: 1 V to 1.5 H: 1 V- Walls will be constructed with compacted back fill used as form work.

In each of above two cases (from vertical to 1.5 H: 1 V) the wall will be designed for all forces including earth pressure and self weight.

Wall slope from 1.5 H: 1 V or flatter than this- Wall assumed to rest on earth, provide minimum thickness (500 mm) and minimum nominal reinforcement.

Typical design details from M 20 concrete and Fe 415 reinforcement are given below:

Materials used

- Reinforced concrete M20
- Plain concrete (PCC) M10
- Steel Reinforcement Fe415

Materials quality – IS 456/IS 3370/IS1786
Design on no crock basis (IS-3370) if the tension face is generally in contact of water. Permissible stresses $\sigma_{ct} = 170 \text{ kg/cm}^2$

Total depth $D = \frac{Mx6}{b\times\sigma_{ct}}$

$\sigma_{cb} = 700 \text{ N/cm}^2$
$\sigma_{st} = 15 \text{ kN/cm}^2$
$m = 13.33$
$N = 0.384$
$j = 0.872$
$Q = 1.172$
Compute $A_{st}$ with $\sigma_{st}$ as 15 kN/cm$^2$

Design on crack basis (IS: 3370) if tension face is not in contact of water.

Permissible stresses

$\sigma_{cb} = 7 \text{ N/mm}^2$
$\sigma_{st} = 190 \text{ N/mm}^2$
$m = 13.33$
$N = -0.329$
$j = 0.890$
$Q = 1.025$

Effective depth $d = \frac{M}{Qb}$

Compute $A_{st}$ with $\sigma_{st}$ as 190 N / mm$^2$

Minimum value of total thickness (D) adopted 500 mm for requirement of durability unless decided otherwise.

Check for shear – IS: 456 Cl. 47.2 (WSD)

$Tc_{max} = 1.8 \text{ N/mm}^2$

Development length / splices/ detailing – as relevant IS Code

For Beams

- Minimum tension reinforcement IS:456 Cl. 26.5.1.1
- Minimum shear reinforcement IS:456 Cl.26.5.1.6
- Side face reinforcement IS:456 Cl.25.5.1.3

For Walls/ Slabs

- Minimum reinforcement IS:456 Cl. 26.5.2.1
For members designed on no crack basis (IS-3370), the minimum reinforcement in vertical and horizontal direction on each face governed by IS:3370 Cl. 7.1

**Min. Reinforcement for TOR Steel**

- IS:456 – 0.12% on each face and each direction
- IS:3370-For D>450 mm, 0.16% in each direction divided equally on two faces.
- IS:3370 (draft) – 0.35% of 250 mm on one face
  - 0.35% of 100 mm on one face

Where thickness of wall varies, the min. reinforcement based on percentage of wall thickness shall be varied by curtailment or changing the spacing/dia as appropriate.

### 2. POWER HOUSE BUILDING

**General**

Types of Structures are as given below:

1. **Main Area**: Isolated R. C. Columns, with filler walls, Raft Foundation (in general), R. C. Gantry Girder, Steel Roof Truss- I.S. rolled sections/steel tubular sections with welded connections, CGI sheets.

   Alternatively- R.C. rigid Gable Frames with vertical columns, horizontal intermediate beams and top inclined (gable)beams, steel Purlins and CGI sheets

2. **Service Area**: R.C. slab, beams, columns, appropriate foundation.

**Loads**

1. Dead load including truss, purlins, roof sheeting.
2. Live Load
3. Wind load- Wind direction parallel and perpendicular to ridge with normal permeability.
4. Earthquake load
5. Snow load (where applicable)
6. Earth / Hydrostatic Pressure.

**Load Combinations**

- DL + LL
- DL ±WL
- DL ±EQ
- DL + LL/ Snow ±WL
- DL + LL/ Snow ± EQ

Add Secondary effects as appropriate
Design Philosophy

Steel Truss is analyzed as normal Practice (pin-joints) using software, members and joints (welded joints) as per relevant IS codes.

Isolated R. C. Columns taken as bottom fixed and top free (normally), with effective length 1.25 L, Loads and moments coming from Crane added.

Rigid Frame analysis is done using Stiffness approach.

Design of members is done using Limit State Design concept

Expressions for limit state design as per IS:456:2000 are given below

Material used

- Reinforced concrete M20
- Plain/Lean Concrete M10
- Steel Reinforcement Fe415

Modulus of elasticity – effect of creep to be considered where necessary

Design of individual members of reinforced concrete structure will be carried out using limit state design concept (IS:456-2000). Design Aids SP-16 will be used wherever necessary.

- Maximum depth of neutral axis is taken as 0.48d where d is the effective depth of the section
- Limiting moment with respect to concrete is taken as
  \[ M_{\text{lim}} = 0.36 \sigma_{ck} b x z = 0.36 \sigma_{ck} b x_m (d-0.42 x_m) \]
  \[ = 0.138 \sigma_{ck} b d^2 \]
- Limiting moment with respect to steel is taken as
  \[ M_{\text{lim}} = 0.87 \sigma_y A_t (d-0.42 x_m) \]

Gantry Girder is analyzed as a continuous beam supported on Column brackets using the available data (Maximum Capacity of crane, Total Crane Girder weight, Weight of Trolley and Crab, Minimum clearance of Crab from center line of Rail, and Impact factor)

Machine Foundation is designed as Block and/or Raft, considering uplift (if present) using the available data.

3. Bridge

GENERAL: Relevant IRC Codes

IRC-5- 1998
IRC-6- 2000
IRC-21-2000  
IRC-83-1987  
Standard Plans for Highway Bridges- RCC Slab Superstructure,  
MOST Publication 1991

Geometrical Features

- Single Lane – 5 m wide carriageway  
- Two Lane – 7.5m wide carriageway  
- Footpath/ Kerb as specified.  

Live Loads:  
- One lane of IRC class 70 R or two lanes of IRC class A on Carriageway, whichever governs (for two lane bridge)  
- One lane of class A considered to occupy 2.3 m, the remaining carriageway width shall be loaded with 5 kN/m² (for single lane bridge).

Footpath load of 4 kN/sq m for super structure having foot path of kerb 0.6 m or more.

Wearing coat _ As applicable

Condition of exposure – IRC-21

Design Details

The strength of RCC structural members assessed by commonly employed Elastic theory (Cl. 304.21 – IRC 21).

Material commonly used: Concrete M 25 grade and Fe 415 reinforcement.

Structural Forms: If the span of the bridge is less than 10 m, The solid slab type bridge is most appropriate option., analysis to be done by effective width method (IRC – 21).

For spans more than 10 m, T- beam bridge will be economical. It consists of three structural elements – Longitudinal Girders, Cross beams and slab. The slab is having two way action generally and may be designed using Pigeud’s Curves. The load distribution among longitudinal girders may be obtained by Courban’s method subject to fulfillment of assumptions. Standard software may also be used for analysis.

Bearings: Bearings are provided in bridges to transmit the load from super structure to sub structure in such a manner that bearing stress induced in the sub structure are within permissible limits and also to allow for certain movements of the super structure. Fixed bearing permits rotation only, while expansion bearing permits rotation as well as translation.

Bearing for slab bridges- For high level bridges with slab spans, no special bearings are usually provided. A thick layer of kraft paper is inserted between the slab and the sub structure at supports. This arrangement is sufficient to allow for small longitudinal movements. To take care of rotations, each bearing area should be beveled or rounded at the edge.

Typical details of Slab type bridge are given in fig. 27.
Fig 27: Typical Details of Solid Slab Bridge
REFERENCES

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24. IRC-21-2000 – Code of Practice for Road Bridges- Section III- Cement Concrete (Plain & Reinforced), Indian Road Congress, New Delhi.
25. IRC-83-1987-Part II – Code of Practice for Road Bridges, Section IX, Bearings, part II Elastomer bearings, Indian Road Congress, New Delhi.
B. BOOKS

27. Reinforced Concrete Designers Hand Book by C. E. Reynolds & J. C. Steedman
31. Plain and Reinforced Concrete , Vol. II by Jai Krishna and O. P. Jain