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Guidelines of Damming and design of River hydraulic structures for flood control

3[nd] Edition, 2024

Farhad Daliri *Dam-River consultant & hydrologist*

"Karun 3 dam with appurtenant hydraulic structures, spillway, outlet works, and stilling basin'',

(Arch dam). 2004, Khuzestan Province, Iran, 205m in height"

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Knowledge is empty of limit,

so we have not enough time to learn everything if you want to deep it in the real world.
Farhad Daliri, 2015

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"I can foretell the way

of celestial bodies, but can say nothing of the movement of

a small drop of water"
Galileo Galilei centures ago

GUIDELINES OF

DAMMING AND DESIGN OF RIVER HYDRULIC STRUCTURES FOR FLOOD CONTROL

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Signature

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ISBN 978-600-6923-21-5 Water & Environmental Modeling (flood control-water supply-groundwater),

By Farhad Daliri in Persian, 2014, 2019.

1. Dams-Rivers engineering 2. Hydrology – Design 3. Hydraulic- Design

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Chapter 1

Dam Hydrologic Design

1.1 Why and Where We Need Dams?

Water requirement pattern (Water use is real and often is more than requirement but we can manage it by parsimony based on social awareness) and available water supply sources pattern, (for example conjunctive use of groundwater and surface water resources or kinds of resources non-class water harvesting) are main criteria to decision making whether need dam or not in a special location (Fig 1.1).

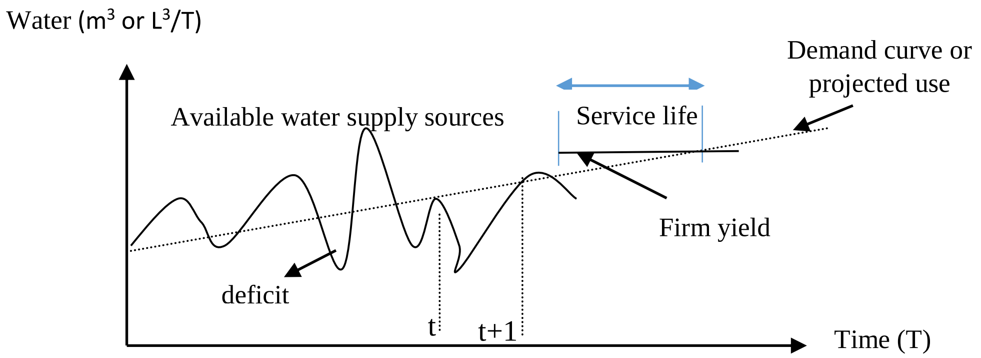


Figure 1.1 Water needs and water supply curves in a special location (After Daliri, 2019[1])

As described in above figure, when a water-supply project (drinking water, irrigation, hydroelectric, navigation …) drawing water directly from a stream may be unable to satisfy the demands of its consumers during low flows. Moreover, often these rivers either carry little or no water during portions of the year, or becomes a torrent after heavy storms and a hazard to all activities along its banks. In this conditions, a dam can retain such excess water in wet periods of high flow (Damaging floods) for use during low flows of drought. On farms, stock tanks or farm ponds (Kinds of negarim system, micro-catchment, and Hotcakes in Sistine Baluchistan – Iran, are small man-made or natural holes with dimensions 200 m^2 in 3 m depth) or kinds of small embankments (Height < 10 m) may storage the ephemeral flow of small creeks for useful purposes.

Unsustainability is vital condition of the human life [1]**.** Hydrologists know dams have many disadvantageous environmental impacts, also know have many advantageous environmental impacts simultaneously. But are you (Environmentalists) ready life in the world without dams?!! (Fig 1.2).
So, we need dams, but must reduce dam's negative impacts based on understanding of Eco-Hydro systems (Hydraulic, Hydrology, Environmental and ecology schemes) [1,10].

Figure 1.2 Environmentalists, if you are ready so you must find water similar the above example

1.2 Framework of Dam Creation Phases

Planning of a water-resources developing project initiates recognition of the need for a project or what is the problem? After that, it is followed by the conception of alternative technically feasible solutions that would satisfy the problem or the needs. These options must be first economic feasibility and then social and environmental acceptability. Finally, financial feasibility and public practicality are important in the choice of final option. If the option is dam construction, generally the process of the dam creation may be according with figure 1.3.

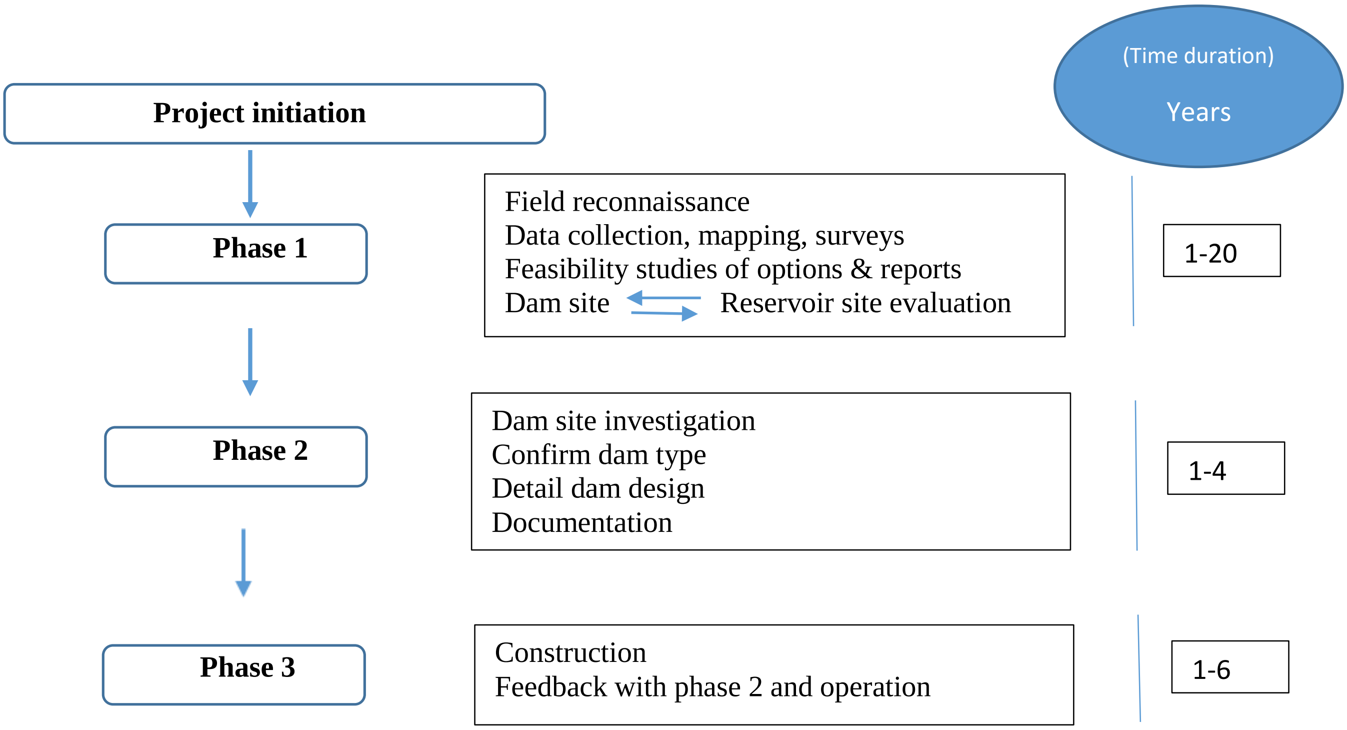


Figure 1.3 Framework of large dams creation phases, Daliri, 2020

Main considerations in Phase 1

- Assessing water needs (Quantity and Quality) and losses (evapotranspiration, infiltration in transition canals, …
- Calculation of watershed yield (snowmelt, peak flood, low flow, …
- Dam site and reservoir site selection (foundation, sediment yield, …
- Dam storage size at each proposed site (evaporation, seepage, release, firm yield...
- Preliminary design and cost estimation and justification

Main considerations in Phase 2

- Soil testing (foundation, spillway site, reservoir, dam materials…
- Seepage losses
- Stability of dam sides and foundation
- Sedimentation in dam
- Dam type and site selection criteria
- Dam body and outlet structures design (pipelines, spillway, …
- Dam economics
- Final review and approval

Main considerations in Phase 3

- Selecting dam builders and an engineer or supervisors
- Construction (Setting out, diversion of water, …
- Equipment
- Emergency Action Plan for flood forecasting, impounding, earthquake, …
- Checking with standards and design maps
- Final inspection and measurements (Hydrologic, Hydraulic and Geotechnical criteria)

1.3 Dams Hydrologic Design

We can summarize problems of Water – Resources Engineering especially in damming projects based on some questions to studies and facilities required. 1. How much water is needed for spatial and temporal patterns? 2. How much water (Min and max flows, annual yield, volume and debit of floods and snow melt, groundwater, non-class water such as fog, dew, and rain roof harvesting, wastewater soil aquifer treatment (SAT), conjunctive use of fresh water with semi and brackish water, transition of water, Qantas under ephemeral rivers, ...) probability can be expected or available? 3. Probability of drought? 4. Who and for what may use the water (location, max load, quality)? 5. What kind of water (chemical, bacteriological, sediment, radioactive, toxic …) is it? 6. What structural problems (dam body and foundation, geotechnical cases, spillways, sluiceways, gates, intakes, channel works, levees, pipelines, river engineering and training, locks, pumps, turbines, purification) exist? 7. Does project affect wild life, natural beauty, social, commercial and law aspects? 8. Is the project economic? 9. What criteria for integrated system management (ISM, Daliri, 2020 [1]) with climate change impacts must be consider? 10. How dynamic systems, be consider? For example, maybe by stochastic process! 11. How dam rule curves and operation policies in crisis situations can be improved? [1].

1.3.1 Dam height and river reservoirs volume

We can classify reservoirs based on aim of exploitation, accuracy and principle of design to 4 groups:

- Distribution-Reservoirs.

Distribution reservoirs apply for water distribution in pipe network lines for urban and agriculture demands (d). So velocity (P_r), debit, reservoir volume (C_r) must hourly supply in critical period [a,b] by pumps. So reservoir volume:

$$C_r = \int_a^b (d - P_r)\,dt \qquad \Delta S = 0 \quad \Big|_a^b \Rightarrow d > P_r,$$ ΔS: Change in storage (Eq.1-1)

- Flood control reservoirs.

There are some schemes such as flood control or sediment control reservoirs that this engineering works described in [1,10].

- Short-term and carry over or long-term Reservoirs (water supply reservoir dams)

Short term reservoirs (small farm dams) often have a 30 years' life time (service life) opposite carry over reservoirs have more than 50 years' life time. So, later type maybe need to supply water demands several sequent years. Principle of design of water supply reservoir dams described in continue.

Water supply reservoir dams

A reservoir may be a single – purpose (e.g. flood control, or water use such as hydropower only or …) or multi – purpose structure, in which zones of storage are showed for variate demands in figure 1.4. The first variables to be determined in a water – supply reservoir design are:

- Location of the dam and reservoir (section 1.4)
- Height of the dam (Sum of heights: dead volume + life volume + flood storage + freeboard)
- Elevation and capacity of the spillway and discharge works
- Mode of operation of the discharge works (**Rule curves based on operation study)**

So, two hydrological variables are paramount:

- Reservoir storage capacity (S_t)
- Firm yield (Yd_t)

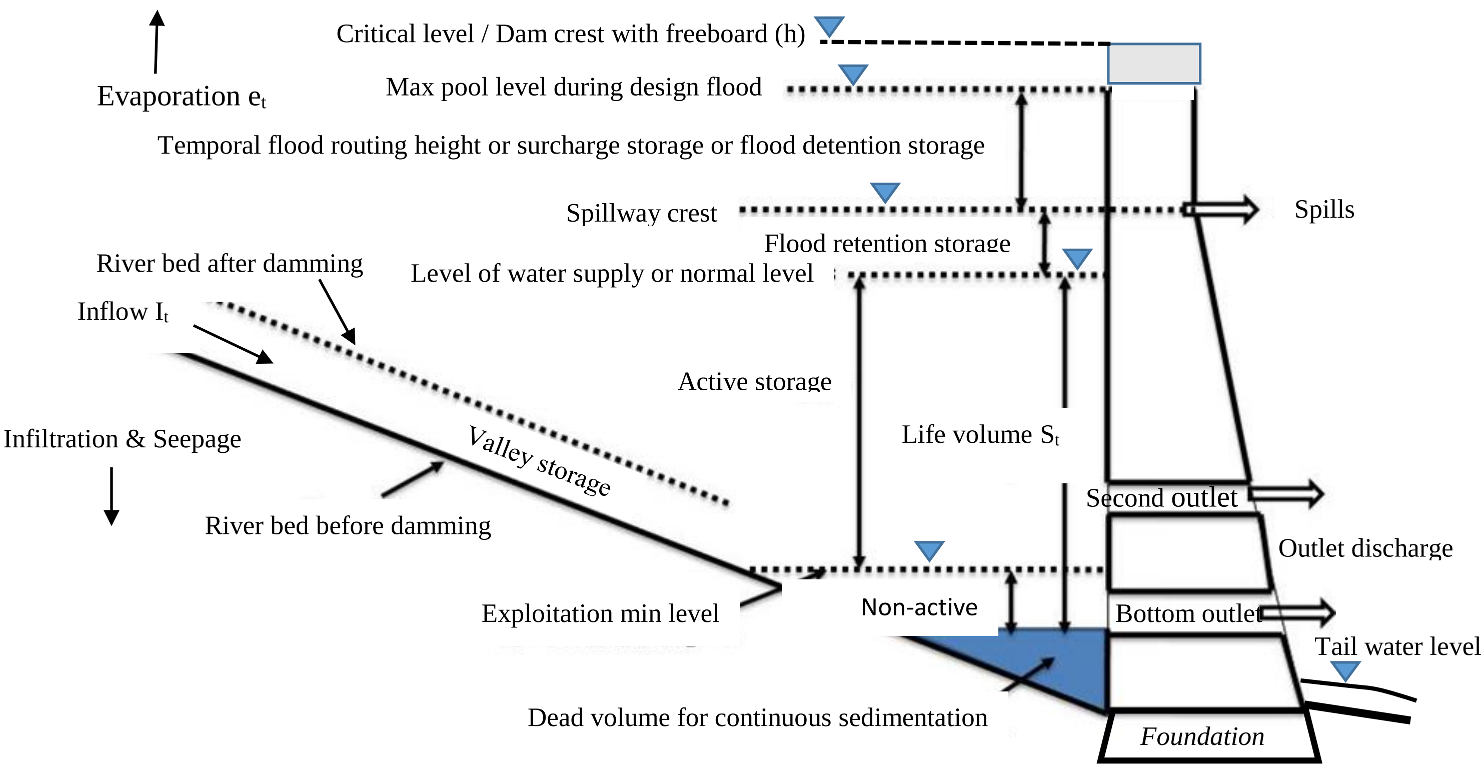


Figure 1.4 Zones of storage in a multi-purpose reservoir dam with two sluiceways, [1]

1.3.1.1 Life volume estimation

Life volume or useful storage estimation (S_t): active + non active storage (such as environmental, firefighting …reserve)

Firm yield (Yd_t) mean (daily, monthly or annual) rate of release of water through the dam reservoir (S_t) that can be guaranteed from an analysis of available historical data or simulated data in ungauged basins (Rainfall-Runoff modeling) by simple methods (Suit to phase I) or complex techniques (Suit to phase II) that may be classified to similar below (A, B, C groups) that can calculate deterministic or stochastic and dynamic or stationary base on risk and uncertainty analysis (**reservoir reliability**).

- A) Graphical methods (simple methods): $\sum(Flow - Demand)$
- B) Mass equation or operation study (rational methods): Simulation
- C) Hydro-Systems analysis (complex methods): Optimization / Simulation

Methods of second group (B) always must be run, because we must control results of complex methods in C group, although results of A group often are useful, and C group results cannot be exact always (Daliri, & et al. 2009). Basic concept of all above methods based on below mass **continuity equations**, although in methods of hydrosystems engineering maybe use other constraints such as water quality, reservoir stratification (in this state we can withdraw water of reservoir different levels by vertical intakes based on water temperature, O_2, BOD…to reduce negative impacts of environmental and water treatment costs), and economic considerations.

Naturally, the storage capacity and firm yield and dam site selection are interconnected, so the larger the storage, the greater is the firm yield, with the limit that the firm yield can never be greater the mean annual inflow rate to the reservoir.

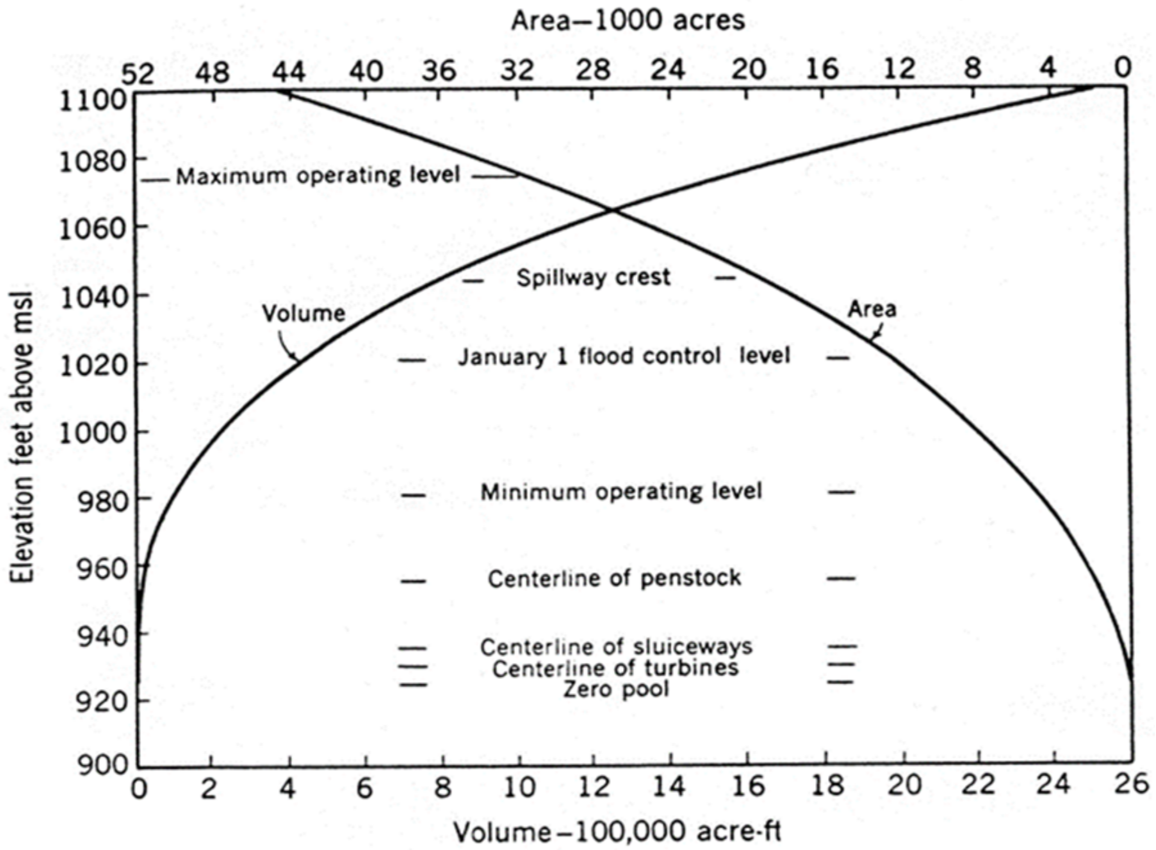


Figure 1.5 Example of elevation – storage and elevation – area curves [3]

These curves can calculated by survey and planimeter DEM in GIS or other softwares or estimated through of the prismoidal formula: Volume $=1/3\,(A_1+\sqrt{A_1A_2}+A_2)\Delta z$

Hydrologic design steps of reservoir dams for water use involves:

1. Projection into the future of the water demand to be met by the dam (growth of population, develop of agriculture, industry…);
2. Calculation of the surface Area – Storage capacity – Elevation curves (figure 1.5) for present and future conditions, (effects of sediment distribution in reservoir extent not dead volume);
3. Computation of the firm yield of the reservoir for present and future conditions;
4. Comparison of the water demand and firm yield of the reservoir to determine its ***service life***, (Figure 1.1) or period of years during which the reservoir will be adequate to meet the demands. Service life, and height of the dam must be economic as well as other technical considerations.

Basic equations to calculate Firm Yield and Live Storage:

Based on hydro-systems (Hydrology & Hydraulic) mechanics [1], compressible, and non-steady continuity equation for a fluid volume can be written:

$$0 = \frac{d}{dt}\int_{cv} \rho . d\forall + \int_{cs} \rho v . dA \qquad \text{(Eq. 1-2)}$$

$v, \rho, dA, d\forall$, Respectively: velocity, density, output area, and control volume.

We can rewrite it for non-compressible, and non-steady form for reservoirs hydrology:

$$\frac{ds}{dt} = I(t) - Q(t) \qquad \text{(Eq. 1-3)}$$

Or:

$$S_{t+1} = S_t + I_t - Q_t \quad \text{or} \quad S_j = S_\circ + \sum_{i=1}^{j}(I_i - Q_i) \qquad \text{(Eq. 1-4)}$$

The determination of required capacity for a river reservoir is usually called an operation study and is essentially a simulation of the reservoir operation for a period of time based on low flow (drought analysis) or mean flow with 500 to 1000 sequent trace (for example) to analyze reservoir reliability in accord with an adopted set of rules.

We can consider graphically term 2 in Eq. 1-4 $\sum_{i=1}^{j}(I_i - Q_i)$ by sequent – peak algorithm (figure 1.6).

Values of the cumulative sum of inflow minus withdrawals (evaporation and seepage) are calculated.

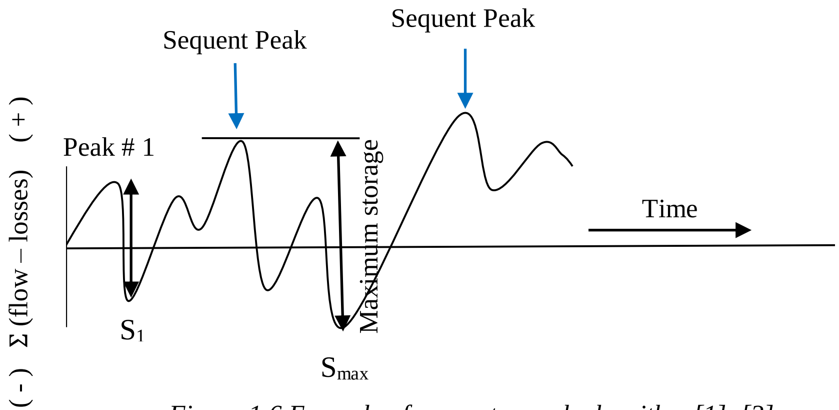


Figure 1.6 Example of sequent – peak algorithm [1], [3]

The first peak and the sequent peak are identified. The required storage for the interval is the difference between the initial peak and the lowest trough in the interval (S_{max}). The process is repeated for all cases in the period under study (historical and synthetic data) and the largest value of required storage can thus be found. We can analyze S_1, S_2 … S_n series based on probability functions and identify $S_{95\%}$ for example for potable uses or $S_{99\%}$ for hydropower generations or $S_{65\%}$ for agriculture demands.

We can modify the above equation by adding spills variable (O_t) for reducing losses of spillway and find minimum storage capacity that supply required firm yield based on demands (Yd_t).

$$S_t = S_{t-1} + I_t - Yd_t - A_t e_t - o_t \quad t = 1,2,....T \qquad \text{(Eq. 1-5)}$$

Equation 1-5 can simulate for n sequent (for example 260 runs) to obtain zero spills with minimum storage that optimum firm yield (debit to supply all demands). So, if seepage be zero and release be environmental demand or firefighting and spills [1]:

$$Yield = \inf low - evaporation + precipitation - release$$

We can calculate yield for n sequent year and sort this series from low to high value based on best fitness of probability distribution, then for example extract mean $yield_{65\%}$, mode $yield_{50\%}$, and firm $yield_{85\%}$ (85% debits are larger than firm $yield_{85\%}$) [1].

Methods that set in C group, need expert know programming in **LINGO** software [1] for example and develop above equation to linear or non-linear objective functions with its constraints [1]. Also there are many simulation models to run above equations such as **HEC-RESIM** and **HEC-HMS** [1,10].

1.3.1.2 Dead volume estimation

We can summarize the main process of reservoir sedimentation in figure 1.7.

Sources of sediment

In **water erosion**, at first must find sediment sources from up-stream area of dam and maybe side bank of reservoirs. Soils or rocks, erode during time by rain drop, and temperature variability naturally and by agriculture activities be accelerated generally. Also civil works such as road developing, bad exploitation of forest, can amplify process of erosion. Erodible soils such as kinds of marl (yield usually clay and silt) and sensitive formations such as schist, granite, (yield often silt, sand and gravel) can produce sediment. These productions (sediments) be transported by force of gravity in water flow and always are less than soil erosion in initial location. So, **sediment delivery ratio** equation is:

$$\text{SDR} = \frac{Sediment}{Erosion} < 1 \qquad \text{(Eq. 1-6)}$$

There are many type of erosion shapes and process, so expert must know them to control or estimate SDR or sediment yield. For example, process of splash erosion initiates of rain drops and develops to rill erosion shape especially in clay and loamy soils. These micro channels develop gradually by run-off shear strain or stress and channel erosion be constituted. Gully erosion is another important water erosion that often yield huge portion in reservoir sedimentation. In gully erosion, first solubility process under the ground causes tunnel, then the ground, collapse immediately. There are other groups of erosion that called, land mass erosion, and river erosion. In mass erosion, huge volume of soil gradually (solifluction) or immediately (landslide) falls. So it can cause water wave if occurs in full reservoir and can bring many sediments in short time.

Wind erosion: Dust and sand storm in arid and semi-arid climate, often can bring particles of clay, and silt or fine sand to fill reservoirs considerable. Although, this kind of erosion often be forgetting but it is vital in dam’s operation rule curves planning. Before estimating wind erosion, at first must find withdrawal area (desert and deforest area) and search if transition path and sedimentation of the area set on reservoir area or not?

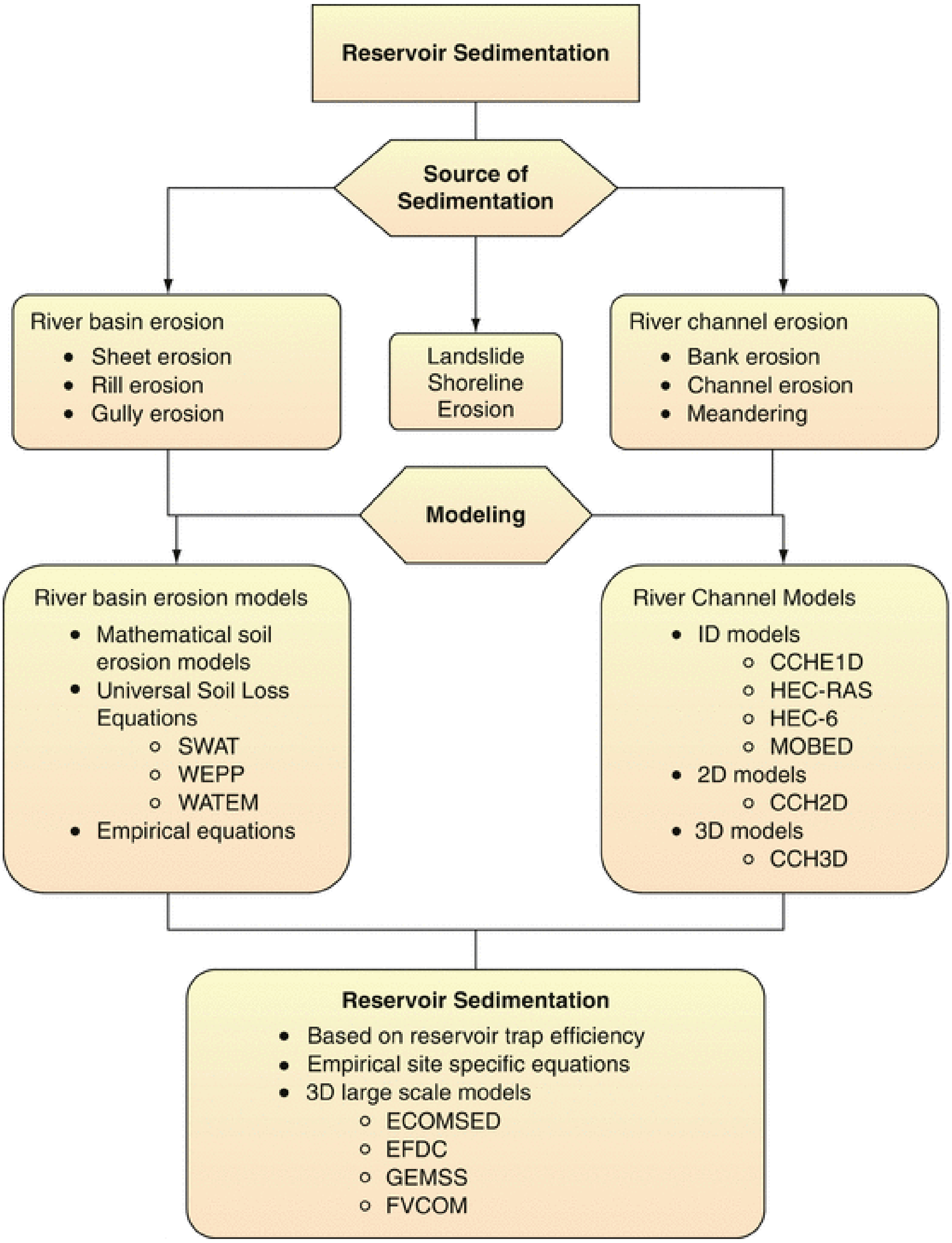


Figure 1.7 The main process of reservoir sedimentation calculations

River - basin sediment yield estimation

Water erosion

We can estimate natural and accelerated (man – made) erosion on upstream watersheds a reservoir by calibrating of water erosion/and or sediment empirical relations and then convert its results to sediment at entrance of the reservoir by SDR (SDR is relevant to slop especially, and must adjust for the basin) and routing the sediment movement in reaches based on river hydraulic concepts in gauged or ungauged river basins (Daliri, 2007). So, expert must always analyze measured **suspended load** (clay and silt) and **bed load** (more than the sizes) at hydrometery stations. If the basin has not the appropriate station, he or she must find at least one **representative station** to calibrate the erosion equation and SDR, then estimate sediment load for the reservoir watershed. Estimation of bed load is a hard task. So often a percentage (often 15 t0 20% but maybe 5 to more than 50% based on Madok table, 1975) of suspended load be supposed instead of bed load (coarse sand, gravel, rock).

Case study (Lorestan Rodbar Dam), Sediment load calculations by Daliri method, 2013 and [1]

Modify effect of debris floods on suspended load by Daliri methods, 2013, [6]

Bed load and considerable volume of suspended load, generally relevant to flood flows or high flows conditions so, often these data missed. To modify or better these results, we can select monthly maximum debit of historical data as a datum from the stations in data series. After that counting with comparing daily debit that are more than the datum and suppose these are number of debris floods. We can average of peak flows debit in data series and identify corresponding load from sediment rating – curves. Now if multiply this value in number of debris flood, we can estimate value of bed load average annually and it must add to suspended load with appropriate density. Sediment rating – curves can calculate by FAO or USBR methods.

Daliri sediment suspended load equation, in ungauged basins 2013 [6,27,28]

Daliri suggested 6 main factors sediment yield (SY) in **ungauged** watersheds are area A (Km^2), annual debit Q (cms), slope factor (S% = S' + S") including main river slope S' and relief effects S", catchment slope effect indirectly or SDR (A^S), potential of erodibility (E), and α, n are coefficients of plant cover that obtain from table 1.1.

$$SY = C(SY') \pm B \qquad \text{(Eq. 1-7)}$$

$$SY = C(A^s.S.E.[\alpha Q^n]) \pm B$$

SY' and SY: Respectively: sediment yield before and after calibration (T/Y)

C and B Calibration parameters

A: Area (Km^2) and s: SDR index from equation 1-8. $0.6 < s < 1$.

$$s = A^{(0.894A^{-0.048})} \qquad \text{(Eq. 1-8)}$$

E: Potential of erodibility is a descriptive factor to set between 1 to 25 in homogeneous units based on sum of 3 other factors value including 1. Formation erosion sensitivity, 2. Geomorphology faces, and 3. Land use that can assign to a value by below methods:

1. Feiznia formations resistant tables, 1995, value 1 for resistant stones to erosion and value 17 for marl and other sensitive stones. In these table climate effects considered. Natural resources journal, univ. of Tehran, Iran, No. 47.
2. Geomorphology faces. (0 to 4). Based on expert experience and interpretation of air photos or satellite images we can assign 0 for areas without erosion and value max for gully and land movement erosion for example.
3. Land use. (0 to 4). We can assign 0 value for natural conditions and 4 for bad agriculture management for example.

*Table 1.1 Values of α, n coefficients in equation 1-7. (After Daliri, 2014, [1]) **

α	N	Type of cover plant
4000	1.02	Forest
59000	0.82	Forest with good range
177000	0.65	Poor range with bush
446000	0.72	Desert area and poor of plants

(Before that Fleming developed these values to run-off estimation) *

Streams sediment transport fluctuations widely from zero during dry weather to extremely large quantities during major floods. So expert must analyze long term historical data. If there is not this data, so, there are some numerical solutions methods for sediment modeling that can be added to a continuous hydrologic simulation model. So we can extend a short sediment record and estimate more reliable the mean annual sediment inflow. Of course, to do this effectively, daily sediment samples should be collected for two or three years at least to provide the data with which to calibrate the simulation model.

Wind erosion

If wind erosion effects on our project, there are many empirical methods that based on wind tunnel experiments such as Negli table or in IRAN, Ekhtesasi table. Also, these equations need some other variables values relevant wind aerodynamic, density and particle size … to estimate weight and volume of the sediments.

Reservoirs sedimentation and dead volume estimation

All reservoirs destiny is to be filled with sediment. But, hydrologist must design excess storage (dead volume) to operate the dam during the reservoir economic service life. So, reservoir planning must include consideration of the probable rate of sedimentation in order to determine whether the useful life of the proposed reservoir will be sufficient to warrant its construction. Also to identify location of bottom outlet must design height of dead volume. Generally, distribution of sedimentation in reservoir are similar to figure 1-8. As showed in the figure fine sediment (silt) often deposits close the dam body and coarse sediment (gravel and larger sizes) deposits in reservoir entrance where called delta and moderate sizes such as sand sediments set between them. To estimate the amount of sediment a reservoir (S_r) by equation 1-9 we need analyze the below data for appropriate time steps Δt.

- Total sediment load inflow S_I
- Specific weights of particles
- Area –volume – height curves that must modify periodically
- Capacity – inflow ratio
- Trap – efficiency (T_e) that often estimate from Brune curves method.

Reservoir trap efficiency relate to below factors:

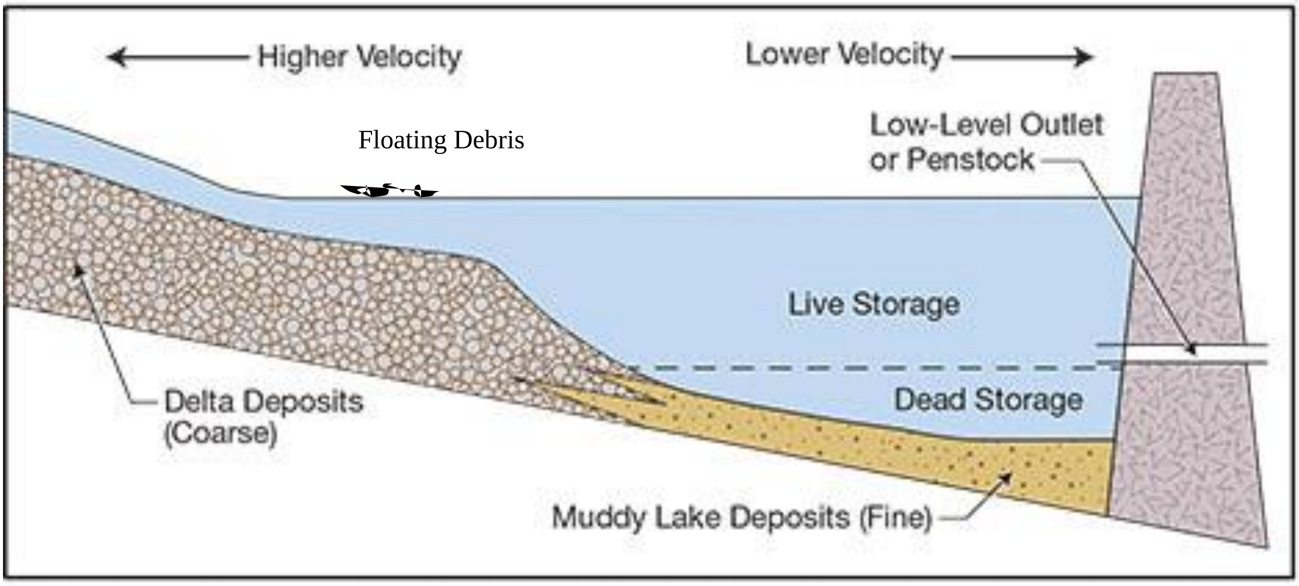
- Reservoir operation
- Characteristic of sediment
- Reservoir life, shape and dimension
- Type of discharge works

So the amount of sediment a reservoir (S_r) can estimate:

$$S_r = T_e * S_I \qquad \text{(Eq. 1-9)}$$

.The volume be occupied by the S_r can then be computed, using a reasonable value of **specific weight** for the deposited sediment. Specific weight is relevant to:

- Reservoir operation
- Particle type (sand, silt or clay)
- Age of deposit
- Compaction characteristics



Typically, sedimentation in the reservoir behind a dam takes the form of progressively finer materials being deposited as the flows approach the dam.

*Adapted from Morris, G.L. and J. Fan, *Reservoir Sedimentation Manual*, McGraw-Hill, New York, 1998.

Figure 1.8 Schematic of the sediment accumulation in a typical reservoir

To study pattern of sediment distribution must know it is relevant to below main factors:

- Valley slope
- Length of reservoir (shape of reservoir)
- Distribution of Particles size
- Capacity – inflow ratio

There are some empirical and numerical methods to calculate distribution of sediment accumulation during service life based on annual or 5 year to 5 year. So time steps, Δt are dependent to condition of sedimentation and sediment inflow amount, type of sediment and the accuracy that we need.

In primary studies (Phase I) we can estimate percentage of occupied in dead volume by silt during y year (φ_s) from Morty equation approximately:

$$\varphi_s = K\,C^{n} \qquad \text{(Eq. 1-10)}$$

C = (sediment inflow during y year / total capacity of the reservoir) * 100

Table 1.2 Values of K, n coefficients in Morty equation 1-10.

Reservoir code	Reservoir shape	k	N
I	Lake	3.39	0.78
II	Hill and plain	9.33	0.56
III	Hill	25.12	0.35
IV	Gorge	32.36	0.3

Erosion and sediment control

Actually, reservoir sedimentation cannot be prevented, but we can retard it by erosion control works at up-stream dam by watershed management (Abkhizdari in Persian) or remedy it by **flashing** techniques at reservoir [1].

1.3.1.3 Design capacity of spillway

A spillway is necessary to discharge floods and prevent the dam from being damaged or overtopped. Although, in some conditions maybe operator be forced use and open other outlet works such as sluiceways. Kinds of spillways and outlet works be described in chapter 2. Here we want to find appropriate return period of design flood and the characteristics of the design flood such as volume, velocity, debit, and quality that can calculate from flood hydrograph, flood sedigraph and flood pollugraph [1].

Return period (T)

Optimum return period can calculate by economic methods (benefit/cost) or by risk and uncertainty analysis in design phases or by expert judgment based on tables and statistical relations in primary phases [1]. The required capacity of the spillway or maximum outflow rate through the spillway depends on:

- Return period or design flood (inflow hydrograph to the reservoir)
- Available reservoir storage (permanent and/or temporal storage routing)
- The discharge capacity of the outlet works

The selection of the spillway design flood (T) is related to the degree of protection that ought to be provided to the dam that depends on:

- The type of dam (earth dams are sensitive for example)
- Dam location (hydrologic conditions)
- Consequences of failure of the dam

The probable maximum flood (PMF) is commonly used for a high dam with an inhabited area while a smaller flood based on frequency analysis is suitable for low dams with no an inhabited area. A determination of the area that would be flooded if the dam were to fail or based on analysis of the flood wave resulting from the breach of a dam by **DAMBRK program** is useful in determining the acceptable risk. In phase I studies we can use the below method to select T simplicity if the **event probability** P be equaled or exceeded in any one year:

$$\mathrm{P}(X \geq x_T) = \frac{1}{T} \qquad \text{(Eq. 1-11)}$$

If fail permissible risk of a dam be 10 % and useful life the dam be n = 50 years, so T can estimate by equation 1-12 about 475 years.

$$T = \frac{1}{1-(1-RISK)^{\frac{1}{n}}} \quad \text{(Eq. 1-12)}$$

Flood storage estimation in reservoir dams

We must estimate flood hydrograph for design flood storage needed. As showed in figure 1-4 we have 2 parts in reservoir that called:

- Flood retention storage
- Flood detention storage

In a retention pound, flood water maybe is holding for a considerable length of time, and then use it for aesthetic, agricultural, consumptive, or other uses. Opposite the detention basin is the holding of runoff for a short period of time and releasing it at a regulated flow rate without damaging downstream dam. Several different methods exist for the detention of storm water in rural and urban area that be described in [1]. There are many flow simulation models such as HEC-HMS, and empirical methods such as SCS-CN, CN methods (Daliri modified the selection of CN, Daliri, 2011[7, 8]), to calculate flood hydrograph and storage routing the flood in rivers by Muskingum- Cunge method and in reservoirs by modified pulse method for instance. Theoretical computation of the change in shape of a **flood wave** on the basis of **wave mechanics** in reservoirs and rivers (**unsteady flow**) is difficult task, but a numerical solution of the differential equations is feasible using a computer [1].

In considering the next generation of hydrological models, it is important to distinguish between different reasons for modelling. What the next generation model looks like might well be different for different purposes. If the interest is only to show that we "understand our science and its complex interrelated phenomena" then a model structure might look very different to that needed to make flood forecasts at a particular site of interest. In fact, based on Representative Elementary Watershed concepts all the balance equations can be reduced to the simple form (Reggiani and Rientjes, 2005) [9]:

$$\frac{\partial \psi}{\partial t} = \sum_i Q_\psi + R + G \qquad \text{(Eq. 1-13)}$$

Where ψ is the mass, energy or momentum, t is time, Q_ψ is a flux of ψ for the ith internal or external boundary, R is a source or sink term, and G is an internal production term. Examples of such balance equations are given by Reggiani and Rientjes (2005) for multiple processes within a REW.

1.3.1.4 Dam freeboard (h)

There are some errors or unknowing about calculations and rare events that dam designers (hydrologist, hydraulic man and dam body designer) could not consider all them in calculations. These packages of uncertainties maybe classified similar below and must consider to height of the dam as a freeboard. In reservoir dams, especially embankment dams, wind setup and waves action is important and has a major value in freeboard in below equation, other uncertainties may be h_0:

$$h = h_{wind} + h_{hyd} + h_{struc} + h_0 \qquad \text{(Eq. 1-14)}$$

- Wind setup, reservoir oscillations, wave height, and wave run-up (h_{wind}), [1, 12].
- Risk and hydrologic uncertainty (h_{hyd}), [1].
- Foundation, Structural and geotechnical uncertainties (h_{struc}), [1,10].

The above equation (1-14), can modify for dyke structure for example. In dykes that construction in meandering rivers, must consider arc effect of the rivers on water level [1]. Other uncertainties such as subsidence effects and wave action is similar the dams

We can use safety factor (S_f) or safety margin (S_m) instead the freeboard based on available tables or expert judgments or based on risk and uncertainty analysis to calculate factor or margin to the dam height [1].

There are many hydrology text books about dam reservoirs hydrologic design, dam flood design, uncertainty analysis, water supply risk analysis and reservoir reliability, safety factor and safety margins for determination of dam freeboard and water planning. Readers can find complementary subjects in references such as Daliri, 2020 [1], Chow, 1988 [2] and Linsley, 1999 [3].

1.4 Site selection in general

The appropriate factors to reservoir and dam site and dam type (chapter 1, 2) selection are interconnected to each other. Also if the foundation be constituted of depth alluvial or soluble formations (such as lime), the dam body cannot be concrete type. Or if the site have an earthquake faults, the dam embankment type is preferred to concrete type body reasonably. General rules for site selection of reservoirs and dams are:

- The cost of the dam type is often a controlling factor in selection of a site.
- Also, cost of relocation of the population and probable damages in upstream of the dam must not more than of a reasonable amount. So the cost of real estate for the reservoir including road, railroad, cemetery, and dwelling relocation must not be excessive.
- If the inflow has huge sediment or bad quality water, we forced to select another site, so the dam type may be different. It is virtually impossible to locate reservoir site having completely ideal characteristics. Although always the foundation of the site must have technical criteria and tributary areas that are unusually productive of sediment should be avoided if possible.
- The reservoir site must have adequate capacity. So, a deep reservoir is preferable to a shallow one because of lower land costs per unit of capacity, less evaporation loss, and less likelihood of weed growth. Although there are some notations such as quality stratification and sediment trap problems in deep reservoirs.
- The quality of the stored water must be satisfactory for its intended use. Also must pay attention to toxic and salt geology formations.
- The reservoir banks and adjacent hillslopes should be stable. Unstable banks will contribute large amounts of soil material to the reservoir.
- The environmental impact of the proposed reservoir must be studied and made available to the public to ascertain the social acceptability of the project.
- The Lorestan rodbar dam, have reservoir clearance problems about the removal of trees and brush that was an expensive operation (difficult to justify on an economic basis) and hard to erect. The main disadvantages resulting from leaving the vegetation in the reservoir are the possibilities that:

1) Trees will eventually float and create a debris problem (Figure 1-7)
2) Decay of organic material may create undesirable odors or tastes in water – supply reservoirs. (This problem occurred in Ilam dam, Iran).
3) Trees projecting above the water surface may create an undesirable conditions and restrict the use of the reservoir for recreation. Frequently all timber that would project above the water surface at minimum pool level is removed. This overcomes most of the problems cited earlier at some saving over the cost of complete clearance.

In Lorestan rodbar dam we forced at first to estimate the number of the trees to estimate cost and time for activities based on plot method of satellite images (Landsat) [1].

- Reservoir leakage problems from most reservoir banks occur but the permeability is so low that leakage is of no importance. If the walls of the reservoir are of badly fractured rock, cavernous limestone or permeable volcanic material serious leakage may occur. This leakage can result in a loss of water and also in damage to property where the water returns to the surface ground. If the leakage occurs through a few well defined channels or within a small area of fractured rock, it may be possible to seal the area by pressure **grouting**. If the area of leakage is large, the cost of grouting may be excessive, so must relocation of the site. Small distribution reservoirs are often lined with plastic membranes to assure water tightness. Of course infiltration from bottom of the large reservoir will clog naturally by clays.

1.5 River – Basin Management Hydraulic Structures (Urban & Rural Hydro-Systems)

In chapter 2 be mentioned main methods for damming. there are many methods for improving the reservoir exploitation during service life or to increase the service life the dams. Often these methods are classified in watershed management practices and river training that be named erosion control, and flood control measures. Also, in watershed management and river engineering there are several effective flood control methods in urban and rural basins. This method will discuss in subsequent chapters.

Other subjects are remediation works based on world experience and available research about reservoir and dam body problems, ecology, hydraulic, hydrology, commercial, society and environmental schemes [1].

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Chapter 2

Principle of Damming

2.1 Introduction

Water retaining structures or dams are hydraulic structures for damming a streamflow to arise the water level (Impounding reservoir or artificial lake). As importance, dimensions, the complexity of the relevant problems of the dams, it should be solved during design and construction. Also it has many environmental, social and commercial effects, so dams fall into the other of the most significant engineering structures.

The most widespread dams, are embankment dams, then various kinds of gravity concrete dams. They are built with local material such as clay, silt (loam), fine and coarse sand, gravel, crushed stone, concrete and reinforced concrete. In particular, structural elements may be required asphalt, steel, wood, plastic materials or modern materials such as, geosynthetics…

The failure of a dam cause serious loss of life and property, so the design, construction, operation and maintenance of dams need skillful and qualified experts especially Hydrologist, Hydraulic man, dam body designer as well as other expert fields such as geotechnical, environmental, economic, etc. The failure of the Teton Dam in Idaho in June 1976 added to the concern for **Dam Safety** (**Flood and Earthquake criteria**) to define design criteria for dam and to develop risk – based analysis methods for application to dam safety evaluation. These methods can calculate kinds of uncertainties [10], [11] to be helpful in delineating that require **Rehabilitation**.

2.2 Dam types

Dynamic behavior of dams based on design type and materials can restrict exploitation of the reservoirs as a constraint in rule curve optimization (chapter 1 & [1]). As well as this subjects, the designer must gain data about discharge works. All they must be considered in the selection of the best type of dam for a given site (chapter 1) as well as engineering feasibility and cost of construction criteria.

Dams are in 4 main groups on the basis of the type and materials of construction, including 1. gravity dams, 2. arch dams, 3. buttress dams and 4. embankment dams (figure 2.1 and 2.2).

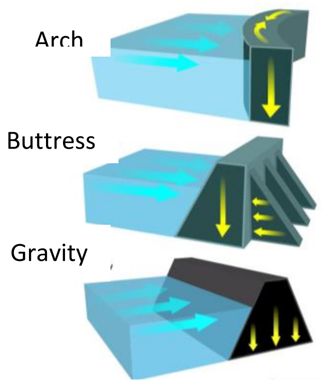

Figure 2.1 Types of concrete dams

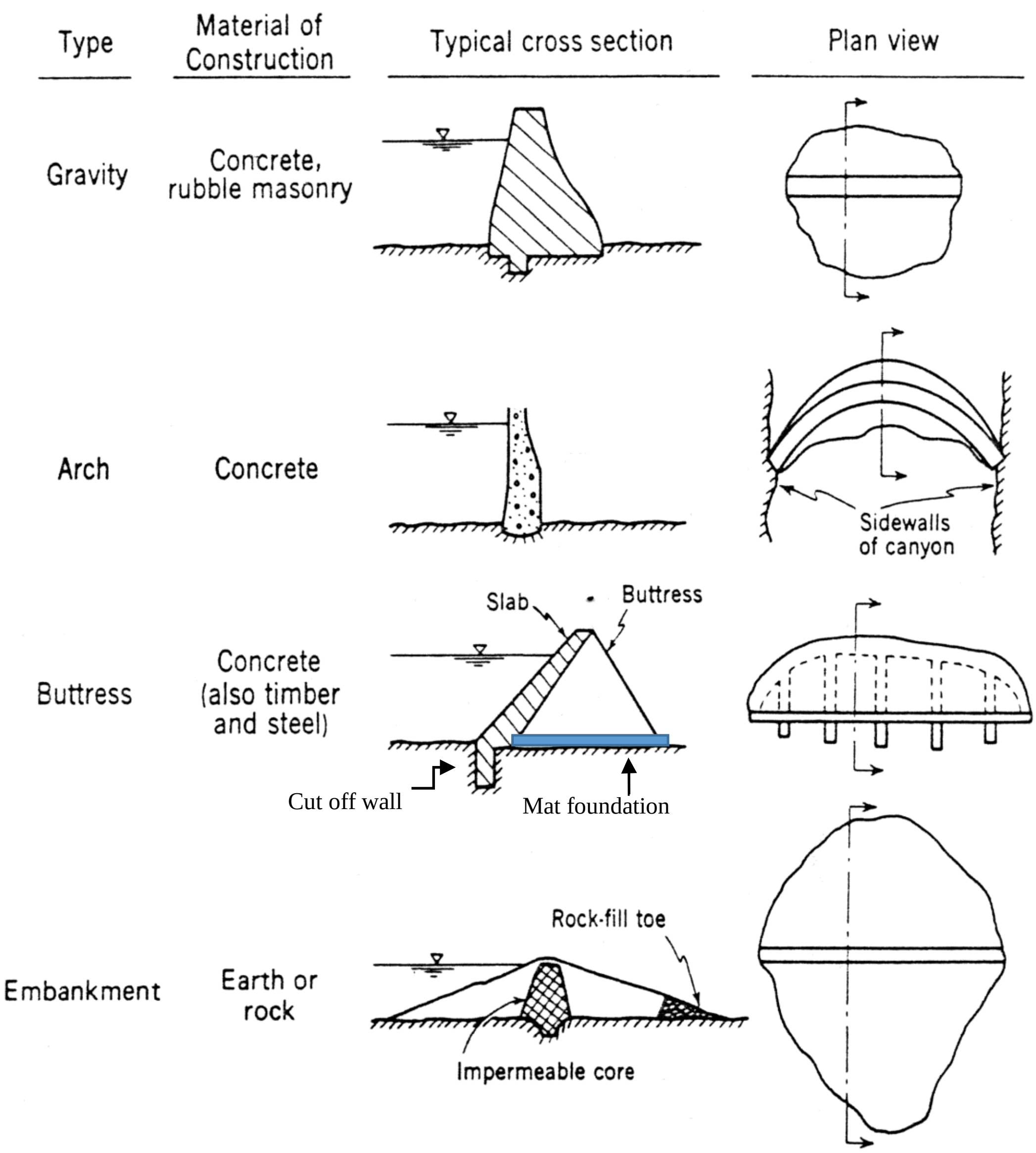

Figure 2.2 Basic types of dams [3]

Also types of dams can classified according to use, design or material. More than one type of dam may be included in a single structure. For example, curved dams may combine both gravity and arch action to achieve stability. Both engineering feasibility and cost problem are vital in dam type selection as well as problems in interconnection with reservoir site selection (Chapter1). Feasibility is governed by technology, topography, geotechnical cases, and climate conditions. For instant in cold climate, temperature effects are a problem for both the concrete and earth dams. Because concrete spalls when subjected to alternate freezing and thawing, arch and buttress dams with thin body are maybe avoided in areas subject to extreme cold. The relative cost of the various types of dams depends mainly on the availability of construction materials near the site and accessibility of transportation facilities.

Embankment dams are erected by placement and compaction of local earth fill and rock fill materials and they can be constructed in 3 main types and are more sensitive to overtopping of flood water and wave actions (Figure 2.3):

1. Homogenous. When the dam body is made of more or less impervious material;

2. Zoned. When the dam body is constructed zones of various materials, in which case impermeability is ensured by means of a relatively thin zone of cohesive material;
3. Third type of embankment dam, that the impermeability of which is achieved by means of facing or internal core (concrete, reinforced concrete, asphalt, geosynthetics, or steel).

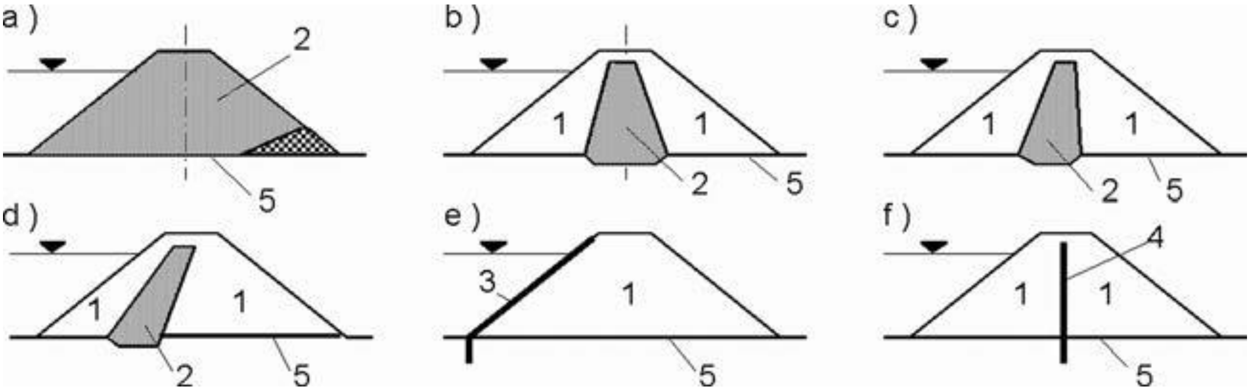

Figure 2.3 Basic types of embankment dams. (a) Homogeneous body; (b,c,d) zoned body; (e) with impermeable face lining, and (f) with an internal core wall made of artificial material, (1) permeable earthfill material; (2) impervious earth material; (3) impervious facing; (4) diaphragm wall core made of impervious artificial material; (5) impervious foundation. [12]

Concrete dams (Figure 2.4) opposite the embankment dams, is not sensitive to wave actions and we can over which overflowing of water as a rule, is not allowed for an embankment dam so, they can be constructed as no-overflow dams (a) or overflow dams (b).

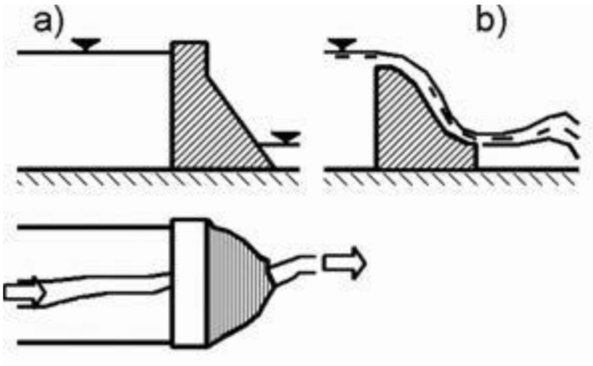

Figure 2.4 View of usual cross-section of gravity (massive) dams. (a) no-overflow; (b) overflow dam. Arch and buttress dams are also constructed as overflow dams, although much less frequently than the massive dams are [12].

Also, concrete dams can have outlet structures in the dam wall for discharging the water from the impounding reservoir. Opposite of the concrete dams, the embankment dams cannot have these discharge works in the dam wall. Of course we can set bottom outlets under embankment dams with technical considerations.

We can classify dams based on aim and use of them such as **diversion** dams, **regulatory** dams and **reservoir** dams. The diversion dams often be constructed when the median of river flow or reliability analysis be in accepted rang of the needs. Another diversion dam such as coffer dams be constructed upstream of the main dam to divert floods and stream flows. Regulatory dams set in downstream of the main dam to manage and divert excess water that be released for hydropower production [1]. In final Concrete dams are less in number than earth dams in the world (ICOLD).

2.3 Investigation for hydraulic structures in General

We must base on data gathering of field activities, survey and soil and geotechnical testing, hydrogeology, and other relevant discipline in studies phases (figure 1.3), consider states of foundation, seepage problems and other relevant criteria. Moreover, in design phases and in some cases during operation must calculate or consider some effects and water actions on the structures special dams:

- Mechanical actions (water and ice pressure, percolation, uplift, …
- Chemical and physical action (metal corrosion, concrete damaging, …
- Biological action (wooden part and concrete damaging, …
- Influence of sediment and detritus deposition, floating debris, abrasive action of silt, sediment
- Environmental impacts (groundwater, earthquake, habitat, water quality, temperature, …

According to water utilization (waterpower, water transportation, kinds of water supply, hydraulic land reclamation) there are many means of hydraulic structures that they set in 3 main groups including **1. water retaining** structures (dams, detention basin in urban areas and ….), **2. water conveying** structures or artificial water courses (channels, natural rivers, tunnels, pipelines that can be underground or surface or elevated structures) and **3. intake** structures. These 3 branches of structures can divide to sub groups of special hydraulic engineering structures based on water utilization branches including below that must do relevant investigations:

- Appurtenant hydraulic structures to the dams (gates, spillways, and outlet works)
- Hydropower structures (chambers, mechanical equipment and…
- Watershed management (check dam, small gravity dam, …
- River regulation (sill, epi, hard point, rip rap seat, cut-off, ….
- Flood mitigation (retarding basin, land management, dyke …
- Recreational structures (rowing path, swimming pools, kayaking, fishing, …
- Fishery structures (fish ladders, fishponds, …
- Irrigation-Drainage (stilling basin, channel, siphons, valves, outfall, pipework, pumps, …
- Relevant structures to floods (bridges, culverts, airports, roads, …
- Sewerage systems (manholes, shafts, pumps, weirs, biological stations, treatment plant, …
- Water transportation (harbors and wharves, locks, lifting structures, …
- Water supply systems (intakes, pumps, valves, pipework, …

We can create channels by 3 methods similar to figure 2.5.

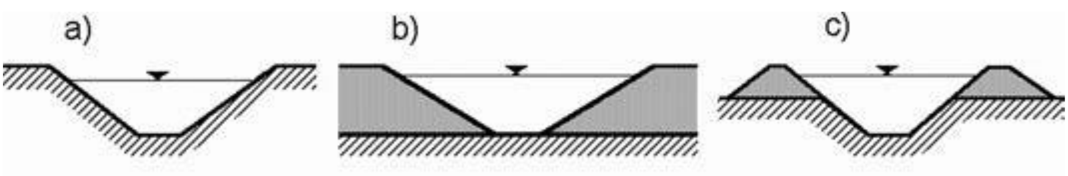

Figure 2.5 Channels are one kind of water conveying structures – (a) Excavation into the ground, (b) formed by embankments, (c) both of a and b. Course beds which are made of reinforced concrete, wood, metal or ground.

2.3.1 EPC and Operation / Service

EPC (Engineering, Preparation, and Construction) of a dam can guarantee some failure of the dams and further **dam safety [26]**. Because responsibility of during dimensioning, design, revision of design documentation, construction and supervision control works return to one responsible company.

Also during construction and especially in the operation phases, the structures must be subjected to permanent organized **monitoring and repair (maintenance)** based on operation rule curves (chapter 1). So, within the body of the dam wall, as well as in the surrounding ground, it is necessary to incorporate appropriate equipment for measuring main below occurrences from the viewpoint of safety and for the secure operations.

- Stresses
- Seepage
- Pore pressures
- Deformations
- Seismic

We can divide all above mention of the dam creation and hydraulic structures to four stages:

1. **Investigation**, expert teams must evaluate all data on natural conditions of the watershed, such as: topography, geology, hydrology and hydrogeology, seismicity, climate, rivers, local materials, geotechnical problems, etc.
2. **Design**, in this stage based on requirements and data obtained from investigations, designers establish the dimensions of the structures, then work out necessary plans to determine the methods of construction, as well as the necessary plant and equipment. Also in this stage it is possible to calculate the economic indicators for construction financing budget.
3. **Construction**, in this stage responsible contractor must organize all activities such as dismantling of the plant and equipment and, in the end, handing over the structure for reservoir impounding and operation.
4. **Operation / Service,** the last stage includes periodic services, useful exploitation and management of its operation, supervision of design requirements and the condition of the structure and equipment. Moreover, the exploitation team must analysis data result of monitoring and field observations of equipment, based on new methods of static and dynamic stability. In earth dam especially must check the rate of flows and capacity of spillway based on new data. Also regular repair and inspection as well as overall overhaul of the structure are vital.

In the end we must know although the high cost of dam construction is true, operational costs are relatively low. Therefore, investments could be **economically** justified. Moreover, in the design of such structures, any templated approach should be excluded.

2.3.2 Investigation for EPC

As mentioned in above sections of 2.3 before and during design and some cases in during construction it is vital to carry out the following investigative works:

- Geodetic maps scaled 1: 100 to 1: 2000 in location of the structures
- Preparation regional maps: DEM (Digital Elevation Model) of the reservoir, dam location and the river basin based on at least scaled 1: 100,000 (SRTM, Satellites, geodetic, air survey etc.)
- Preparation regional maps: DEM (Digital Elevation Model) of the reservoir, and the dam location and other structures based on at least scaled 1: 25,000 (SRTM, Satellites, etc.)
- Geological works. It can use all existing data, or by means of intermediary methods such as geophysical investigations (seismic and electrical tests, etc.) and direct methods (wells digging, trenches, deep boring, etc., for laboratory testing). The aim of it is:

To light on the geological state of the structures location, physical and mechanical properties of its minerals, hydrogeological conditions, geological boundaries, faults, geology relationship with drainage, vegetation, land use, access road, borrowing pits of local materials, reservoir sedimentation, etc.

- Geotechnical works. This item is vital in concrete dams and as well as in large embankment dams. Characteristics such as the elastic and deformational of the rock materials at the dam site, the foundations of the structures and materials which will be used for construction of the dam's body are considered based on field and laboratory investigations.
- Soil mechanic works, which determine the conditions of soil materials (non-rock) in the foundation, as well as in the borrow pits for local earth materials by means field and laboratory

equipment. This item is necessary for cases of earth fill and earth-rock dams to perform aeromechanical investigations in soil foundations, especially for seepage and etc.

- Hydrologic works. As mentioned in chapter one, this item must study before, during and after dam construction in operation times.
- Other investigation works, such as sources of supply of construction materials, living accommodation of workers, data that are vital in the realization of the construction works, river and environmental effects, economic effects, radioactive and bacteriological problems, risks to reservoir about erosion and sedimentation, floods and dam breach [26], first reservoir impounding, outlet works hydraulic models, and etc.

2.3.3 Foundation of dams

Foundations for kinds of hydraulic structures and dam types are variety according to region in final site selection dam. So, the site maybe rock, semi-rock or soil base foundation. Then based on state of foundation we must design and consider requirements for the foundation. Also, investigation works regarding dam foundation can be indirect or direct for sampling and testing. Although improvement of foundations is a remediation during dam construction or after it, it is wisely, to select the dam site with complete investigations in connected with appropriate dam type and the site.

General

In site selection often we can see topsoil, subsoil and bedrock. In bedrock, rocks maybe igneous or magmatic rocks (Granite, gabbro …), and sedimentary (hard and soft limestone, siltstone …), and metamorphic rocks (schist, marble, slate …). Engineering properties of these rocks such as density (kN/m^3), porosity (%), water absorption (%), and unconfined compressive strength (MN/m^2) are important. Semi- rock foundations are usually represented by medium to highly weathered rock and sometimes completely and often are unsuitable as foundations for dams. Soil is the completely weathered materials in the upper layers of the earth and we can classify them to clay (<0.002 mm), silt (0.002 to 0.063 mm), sand (0.063 to 2 mm) and gravel (2 to 63 mm) that for hydraulic purposes can be divided into two characteristic types:

- Non-cohesive materials: sand and gravel.
- Cohesive materials: clay, loam, loess.

Behavior of these soils are characterized by compressibility under loading (settlement), angel of internal friction, and water permeability.

Requirements for the foundation and Investigation works

In design and construction of dams, we must consider three important factors including (figure 2.6):

- Stability
- Deformability
- Water impermeability

To study above items, it is necessary to investigate by indirect and direct methods to gather data. Indirect methods of determining geological conditions, rock and soil mechanical data include a number of geophysical methods such as geoseismic sounding and geoelectrical sounding. Direct methods are sounding drilling and digging test pits, adits, shafts and trenches.

Improvement of foundations

Often there is some deficiencies about natural foundations that must be improved in two ways including 1 – grouting and 2- backfilling of fractures, or tectonic zones and soils. Depth and situation of grouting and backfilling can be calculated by geotechnical experts based on field and laboratory investigations (figure 2.7). As showed in the figure a concrete layer for example is used as bedding for **consolidation grouting**, which receives the required pressure and prevents the appearance of a water–cement mixture (laitance) on the surface. The thickness d of the concrete layer is determined by the condition: the weight of concrete should be larger than the pressure at grouting [12]:

$$d = P.\ 10^3 / k_s\ \gamma_b \qquad \text{(Eq. 2.1)}$$

Where, P is maximum pressure at grouting (MPa); γ_b is weight by volume of the concrete (kN/m^3); k_s is safety coefficient, which allows increase in P, and is usually between 1.5 to 2.

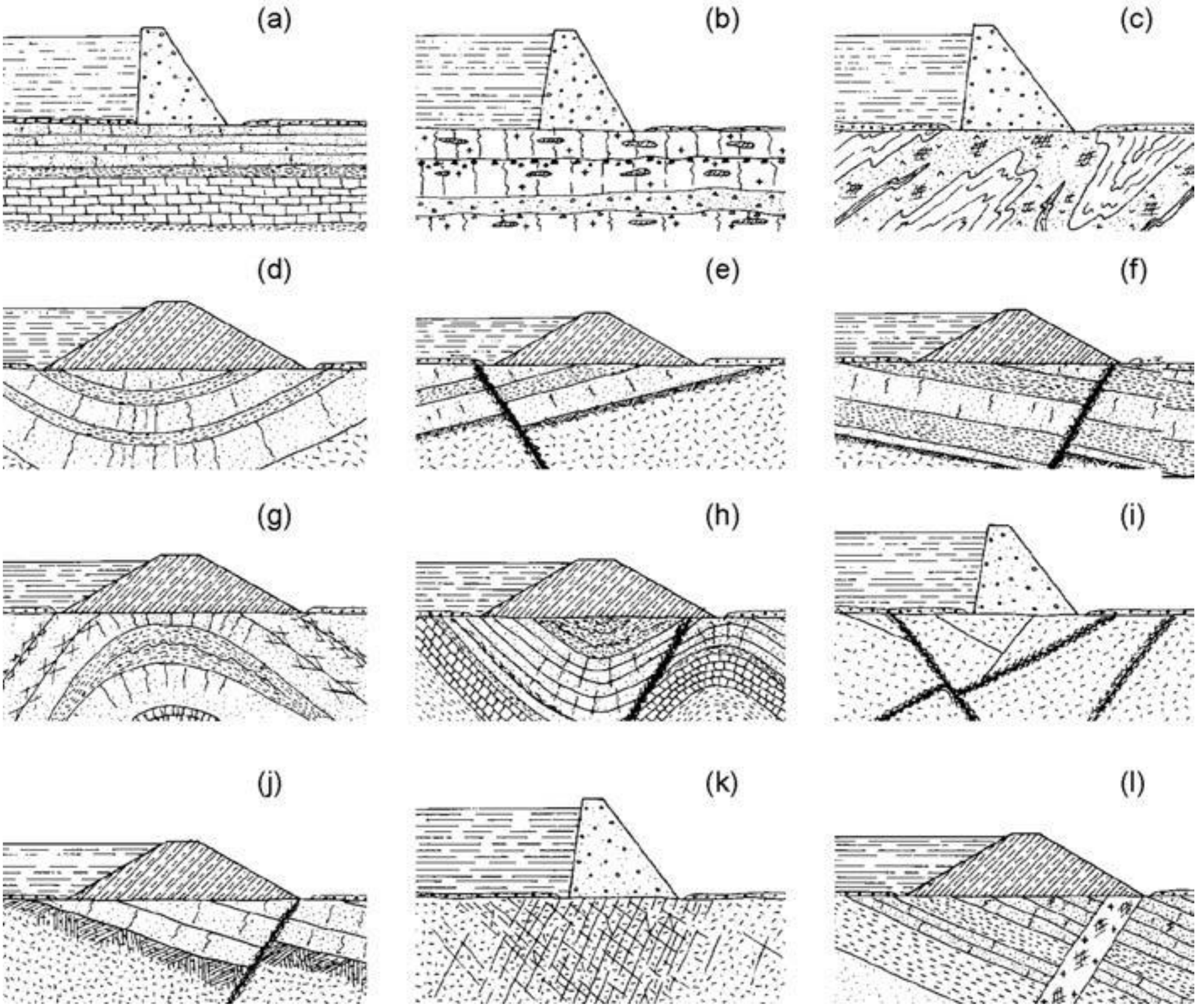

Figure 2.6 kinds of disposition of extending layers in bedrock (after Wahlstrom, 1974)

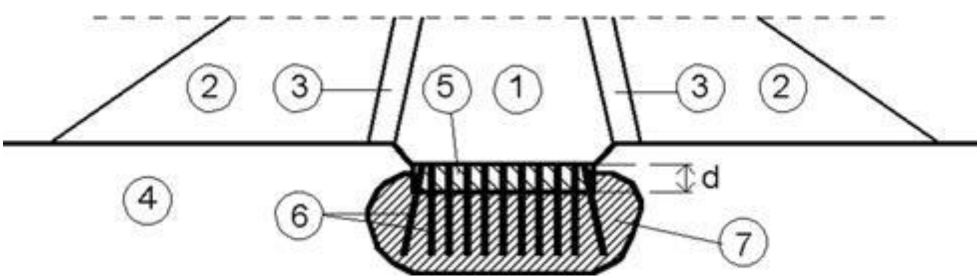

Figure 2.7 Consolidation grouting beneath the core of an embankment dam. (1) Clay core; (2) course grained material; (3) filter zones; (4) rock in the foundation; (5) concrete layer; (6) boreholes; and (7) consolidated foundation [12].

2.4 Gravity dams

Design of gravity dams must base on principles of mechanics of materials and be statically and dynamically stable and the design must be such that **stresses** in the concrete do not exceed **allowable limits**. So to have a stable structure, the reactive forces developed by the rock and soil upon which the dam rests must be capable of **balancing** the active forces (R) such as:

Dam weight, hydrostatic pressure, uplift and seepage, ice pressure, and earthquake forces. These forces (figure 2.8) are transmitted to the foundation and abutments of the dam, which react against the dam with an equal and opposite force, the ***foundation reaction***.

Moreover, the effect of temperature on thickness of dam body (t) or influence of temperature changes, shrinkage and expansion of concrete on stresses in dams, wave load, hydrostatic pressure caused by sediment deposits in the reservoir, influence of cavitation and aeration on hydraulic structures, of dynamic forces caused by water flowing over the dam and seepage though the body of concrete dams and foundations may require consideration in special cases (figure 2.9).

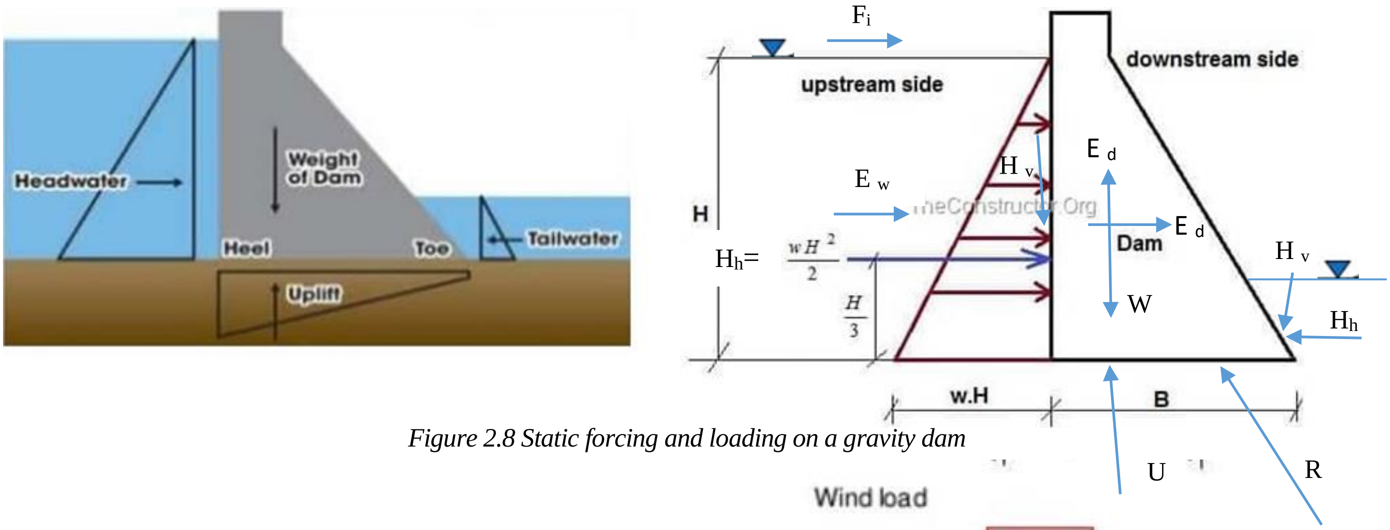

Figure 2.8 Static forcing and loading on a gravity dam

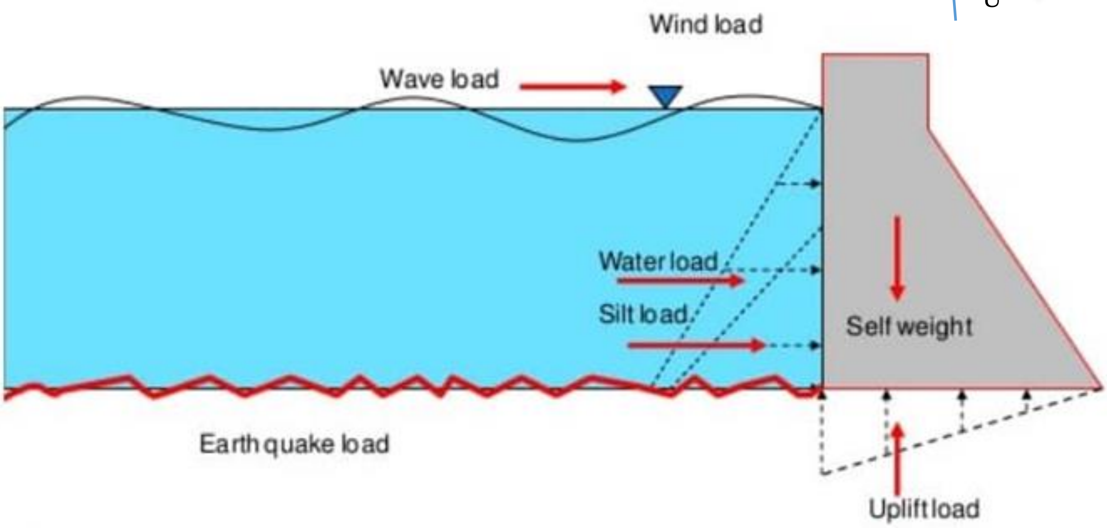

Figure 2.9 Dynamic forcing and special loading on a gravity dam

A graphical form of the **flow net** is illustrated by the example of **seepage** below a concrete gravity dam through a porous stratum that lies on a water impermeable rock (Figure 2.10, c). So, Seepage beneath a concrete dam and through the body can be problem. Measures for reducing the harmful effect of seepage by means of vertical partitions (figure 2.10, b (1)) and horizontal partitions b (2) are be showed.

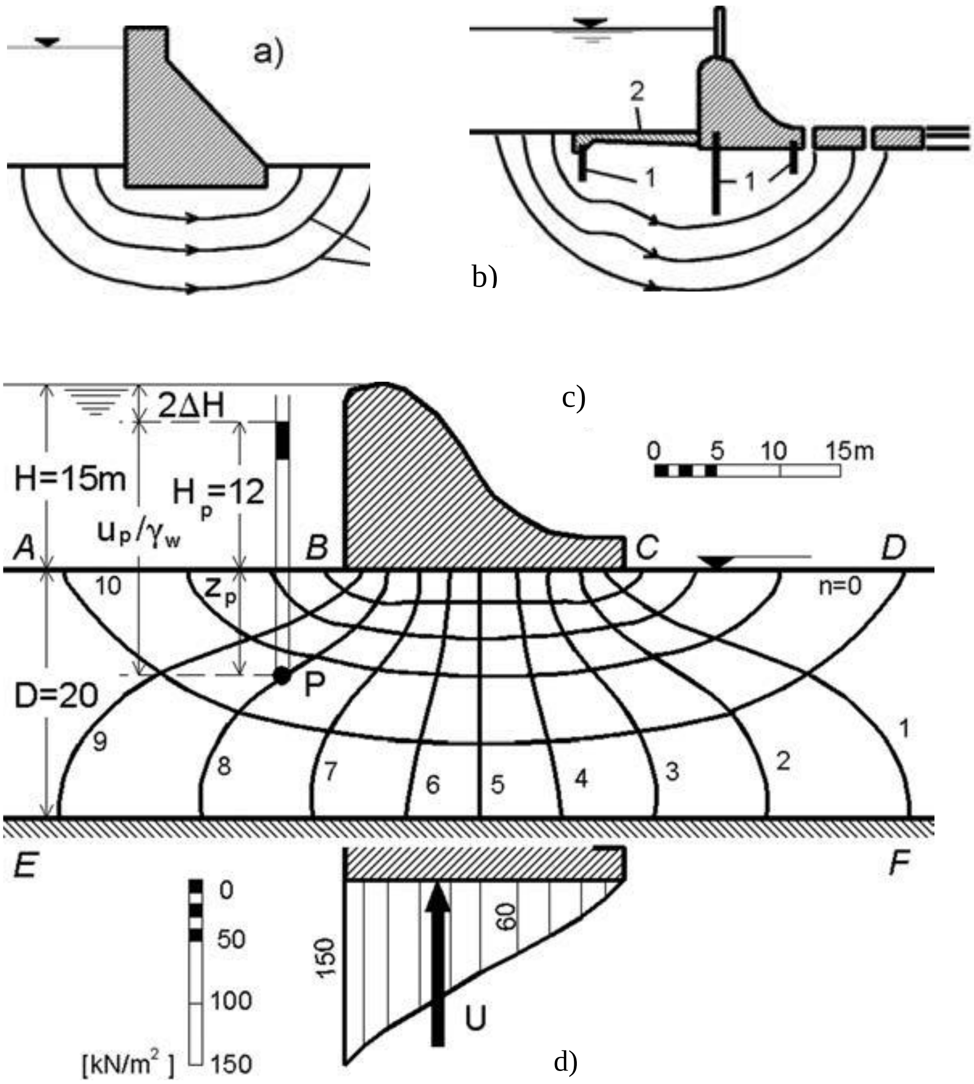

Figure 2.10 Flow net beneath a gravity dam [12]

The **weight of a dam** (W) is the product of its volume and the specific weight of the material. The line of action of this force passes through the center of mass of the cross section.
Vertical (H_h) and horizontal (H_v) Components of **hydrostatic forces** can calculate by fluid mechanic relations. H_h of the hydrostatic force is the force on a vertical projection of the face of the dam, and per unit width of dam is:

$$H_h = \gamma_w . h^2 / 2 \qquad \text{(Eq. 2.2)}$$

Where γ_w or γ is the specific weight of water (H_h= wH^2/2 in figure 2.8) and h is the depth of water at that section. The line of action of this force is h/3 above the base. The vertical component H_v of the hydrostatic force is equal to the weight of water vertically above the face of the dam at that section and passes through the center of gravity of the volume.
Uplift pressure (U): water behind and beneath the dam inevitably finds its way between the dam and its foundation and in some cases through the dam body and creates uplift pressure. The magnitude of the U depends on the material and character of the foundation, construction techniques, and percentage of the pores of the body materials. Uplift pressure in real, varies nonlinear but often assume it varies linearly from full hydrostatic pressure at heel to full tail water pressure at toe. For this assumption the uplift force U per unit width of dam is:

$$U = t \cdot \gamma_w (h_1 + h_2)/2 \qquad \text{(Eq. 2.3)}$$

Where: h1, h_2 are respectively the water depths at the heel and toe of the dam.
Actual measurement on dams indicate that the equation 2.3 estimates more than uplift force in real. Because the internal pressure distribution is influenced by drains systems in dam body, their size and spacing within the dam, and by grout curtains that may extend down into the foundation material. Also we can calculate the distribution of U by flow net (figure 2.10 c & d), numerical methods using finite elements or electric analogs and field activities such as piezo metric measurements.
Ice pressure. For dams located in ice-forming areas (an ice sheet subject to temperature increase will expand and exert a thrust against the upstream face of the dam and wind drag on the sheet of ice may also be a factor), a load of 145 kN/lin m of contact between ice and dam (or several times in severe climates) is often used for design purposes. This pressure primarily depends on the temperature rise of the ice, its thickness, and the lateral constraint to which it is subjected.
Earthquake forces. Here we have vertical and horizontal acceleration. If earthquake shakes the earth upon which a dam rests, the resulting **inertia force** equals the product of the mass of the dam and the acceleration caused by the earthquake. Horizontal acceleration can be assumed 0.5 to 1 g to act on the dam. Moreover, vertical motion may also occur during an earthquake, with a resulting vertical inertia force that acts momentarily to change the effective weight of the dam. In addition, earthquake cause oscillatory increases and decreases in the hydrostatic pressure on the face of the dam. Von Karman suggested that this force be computed from equation 2.4:

$$E_w = 0.555\, k\, \gamma_w\, h^2 \qquad \text{(Eq. 2.4)}$$

Where k is the ratio of the acceleration caused by the earthquake to that of gravity. The force E_w acts at a distance 4h/3л above the bottom of the reservoir.

Gravity dams may be constructed on rock or soil foundations, so behavior of the structure and stability of the gravity dam must test and best dimension according final field condition and balancing between components (R) of forces be designed. There are simplified approach to analysis of gravity dams based on elastic behavior of concrete (this method is adequate for the design of small dams < 10 m height) such as method of layers, full profile method and more sophisticated methods including finite-element methods, determination of stresses by the gravitational method, and theory of elasticity.
Wave and wind load effect are more important in earth dams, so these loads are described in embankment dam section. The readers need more details can find in references such as 12 & 3.

Construction of gravity dams
We can construct a gravity dam by pouring the concrete in blocks by **block – building technique** (old method) or making use of **roller – compacted concrete techniques** (section 2-5). The streamflow must be **diverted** by cofferdam before construction work. For this aim, there are two main methods. One method named two – stage construction the flow is diverted to one side of the river by a cofferdam, while work proceeds on the other side. After work is complete, flow is diverted through outlets or may even be permitted to overtop of the half of the dam. Another method for diversion, if geologic and topographic conditions are favorable, a tunnel or diversion channel can be used to convey the entire flow around the dam site (Section 2.9.3).
It is better the foundation be excavated to solid rock before any concrete is poured. Also cavities and faults in the underlying strata are **sealed** with concrete or grout and a grout curtain can place near the heel of the dam to reduce seepage and uplift effects. A grout of cement and water sometimes mixed with a small amount of fine sand is forced under pressure into holes drilled into the rock. Grouting at pressures 40 psi maybe done before concrete is placed for the dam, or more than 200 psi can be used but later is done from permanent galleries in the dam after dam is complete so that the weight of the dam can resist the grouting pressures.
The size of the individual **block forms** depends on the dimensions of the dam, with a maximum width of about 15 m on large dams and maximum height of a single pour is about 1.5 m. Sections are poured alternately so that each block is permitted to stand several days before another one is poured next to it or on top of it. Also it is vital they are **sprinkled** with water and protected from drying. After the forms are removed, the lateral surfaces of each section are painted with an asphaltic emulsion to prevent adherence to adjoining sections and to form construction joints to reduce **cracking** of the concrete.

Keyways (between sections to carry the shear) and metal **water stops** for dams (such as annealed copper strip are placed in the vertical construction joints near the upstream face) during construction activities are schemes to the dam act as a monolith and to prevent leakage respectively.
Inspection galleries to permit access to the interior of the dam are necessary for grouting works, for operation and maintenance of gates and valves, and drains water that seeps into the dam.
Heat problems in concrete must consider during construction. When concrete sets (often two sacks of cement per cubic m of concrete), a great deal of heat is liberated and the temperature of the mass is raised. As the concrete cools, it shrinks and then cracks may develop [3]. There are some solutions such as:

- Special low-heat cement may be used
- Very lean mixes are also used for the interior of the dam
- Temperature of the concrete mix should be between 10 to 25 C^0
- Cooling by circulating cold water by pipes embedded in the concrete that it only can use on large gravity dams because it is expensive.

2.5 RCC gravity dams

Design and forces are similar to gravity dams at section 2.4. Roller – Compacted Concrete (RCC) technology since the mid-1970s making use and in 1982 the first RCC dam was built (Willow Creek Dam in Oregon with 51 m high and 518 m long). The main advantages of the RCC dams are economics and rehabilitation (chapter 2) of dams. Its cost is normally estimated to be about 60% of the cost of a gravity dam constructed by conventional methods (section 2.4). RCC dams draws on concepts from the technology of gravity dams and earth-fill embankment dams. The upstream of RCC dams are usually made vertical while the downstream faces are commonly set on a slope of somewhere between 0.6 and 0.8 horizontal to 1 vertical that are stable, even during construction under the action of loads by heavy rollers on the concrete fill.
Also, there is no slump problems in RCC dams, rather dry, mix of cement, fine and coarse aggregate (sand and gravel), and fly ash (flue dust from burning of coal). It can be conveyed or hauled in trucks and/or conveyor belt and spread with a bulldozer. It can compact with a heavy static or vibratory roller. There is less shrinkage and cracking in RCC dam during curing, because it uses low quantities of cement in the mix that together with fly ash this serves to reduce the heat generation during setting. The max is spread in 20 to 45 – cm layers.
The technology of RCC can give the concrete a tensile strength of about 750 kN/m^2. There are some ways to achieve cohesion between lifts:
Small dams
It can be possible to limit the time between lifts according to Maturity Factor, 150 to 250 [$C^o \times$ hours]
Large dams
The time between lifts may be large, so the surface of the first lift is water sprayed and cleaned and then a 5 – cm layer of mortar is placed prior to the placement of the next lift.
There are seepage problems in RCC dams. Water tightness of the upstream face can be achieved by 3 main methods:

- By using conventional formwork and normal pouring procedures (section 2.4)
- By using interlocking precast concrete panels held in place by steel anchors embedded in roller compacted concrete
- By forming a succession of concrete curbs (figure 2.11)

There are types of RCC dams based on ways of construction and water tightness that related to properties of the applied RCC mixtures in the dam body such as cement content (high, medium, low), coarse and fine aggregate, pulverized fly ash and water such as hardfill dams. Varieties of these mixtures can show kinds of properties on compressive strength, tensile strength and modulus of elasticity.

The readers can find detailed subjects in reference of number 12.

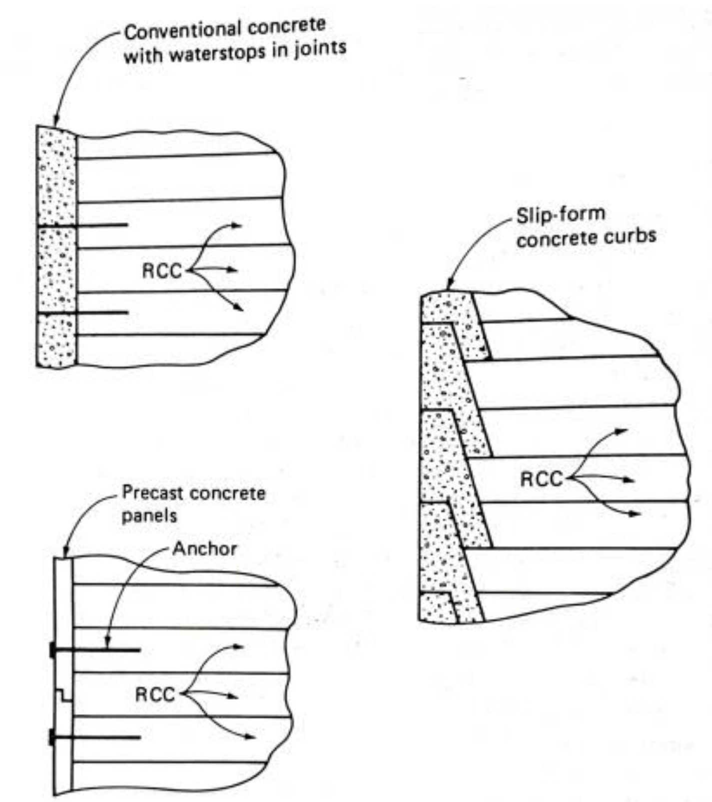

Figure 2.11 Seepage control at upstream face of RCC dams [3]

2.6 Arch dams

Design and forces location on arch dams is somewhat different from gravity dams. An arch dam is curved in plan (figure 2.12) and carries most of the water load horizontally to the **abutments** by arch action. So it is essential that the **sidewalls** of the canyon be capable of resisting the arch forces. From 1884 the first arch dam built with rubble masonry, but practically all arch dams constructed in recent years of concrete. Relatively few arch dams have failed, in comparison with the more numerous failures of other types of dams.

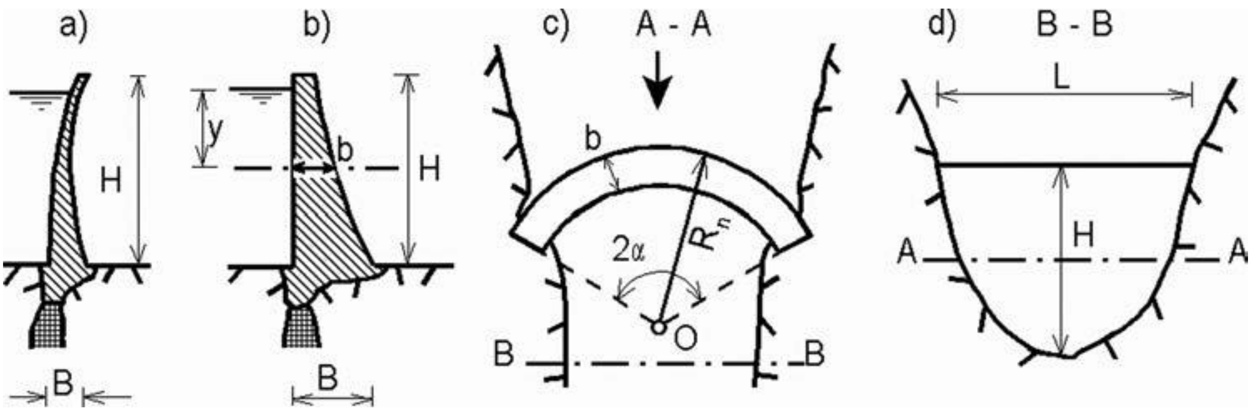

Figure 2.12 Schematic presentation of a true arch dam (a) and an arch-gravity dam (b) [12]

Structural elements (horizontal arches and vertical cantilevers) and types of arch dams – constant center or constant radius arch dam (U-shaped canyon), variable center or constant angle arch dam (V-shaped canyon) [3]

In principle an arch dam is visualized as consisting of a series of horizontal arches transmitting thrust to the abutments or a series of vertical cantilevers fixed at the foundation (figure 2.12). The horizontal component of the water load is resisted jointly by the arch and cantilever action. Structural analysis of arch dams is complex and the computations are lengthy [3]. The distribution of the load between the arches and the cantilevers is usually determined by the **trial – load method**, which begins with an assumption as to the load distribution. The **deflection** of the arch at any point should equal the deflection of the cantilever at the same point. **Stresses** in the dam and foundation can then be computed on the basis of this load distribution. Sophisticated analysis of arch dams that consider the effect of seismic loads are available.

The arch dams have narrow base width, so uplift pressures are less important than for gravity dams. But, stress caused by ice pressure and temperature changes may become quite important in arch dam design. The simplest methods to arch analysis is to assume that the horizontal water load is carried by arch action alone (figure 2.13).

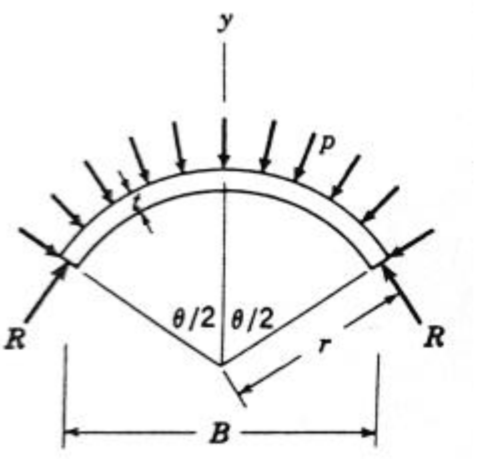

Figure 2.13 Free – body diagram of an arch rib [3]

We know intensity of hydrostatic pressure is $P = \gamma h$, so for unit height on a rib, the total downstream component of hydrostatic force is:

$$H_h = \gamma h 2r \sin \theta / 2 \qquad \text{(Eq. 2.5)}$$

Since $\Sigma F_y = 0$, so this force is balanced by the upstream of the abutment reaction ($R_y = H_h$):

$$R_y = 2R \sin \theta / 2 \qquad \text{(Eq. 2.6)}$$

So:

$$R = \gamma h r \qquad \text{(Eq. 2.7)}$$

We know if the thickness t of the arch rib is small as compared with r, there is little difference between the average and maximum compressive stress (σ) in the rib so, $\sigma \approx R/t$, and the required thickness of the rib is:

$$t = \gamma h r / \sigma_w \qquad \text{(Eq. 2.8)}$$

Where σ_w is the allowable working stress for concrete in compression.

Construction of arch dams

Concrete is placed in a manner similar to that gravity dams, usually in 3 m lifts, to 6 m at the upper levels that the section is quite thin. A layer of **mortar** is usually placed between lifts to ensure better bond. Since the cross section of an arch dam is relatively thin, care must be taken in the pouring, mixing, and curing of the concrete in order to secure adequate resistance to weathering and **seepage**.

The foundation of arch dams must be stripped to solid rock and the abutments should be stripped and excavated at approximately right angles to the line of thrust to prevent **sliding** of the dam.

Seams and pockets in the foundation and abutment are **grouted** in the appropriate manners.
Small arch dams have radial and horizontal construction **joints**, while large arch dams have circumferential joints as well. All joints must have keyways, and water stops must be provided to prevent leakage.
Also, to minimize **temperature effects** and **stresses**, the closing section of the dam is poured only after the heat of setting in the other sections is largely dissipated.
For **static analysis** of arch dams except the above method there are methods including method of independent arches, central cantilever, the finite element method and the experimental method. Also we can design and construction roller-compacted concrete arch dams. The reader can study more detail data in reference 12.

2.7 Buttress dams

The mains components of the buttress dams are membrane (**slab** about 45^0) and **buttress** (figure 2.1 and 2.2). So the water load be transmitted to a series of buttresses at right angles to the axis of the dam. There are several types of this dams including flat-slab (Ambursen, 1903) type, multiple-arch type (arch-deck buttress dams), and massive-buttress dams such as mushroomed head buttress dam (figure 2.14). These differ in that the water supporting membrane in one case is a series of **flat reinforced-concrete slabs**, while in the multiple-arch a series of **arches** that permit wider spacing of buttress. Required concrete in buttress dams is less than gravity dams but are not necessarily less expensive because of the increased formwork and reinforcing steel involved. So we must select optimum economic and technical spacing of buttresses.
Since a buttress dam is less massive than gravity dam, the **foundation pressure** are less and a buttress dam may be used on foundations that are too weak to support a gravity dam. If the foundation material is permeable, a **cutoff wall** extending to rock may be desirable to seepage control (figure 2.15).
Slab slop and buttresses assist the dam against sliding and overturning. We can increase height of a buttress dam by **extending** the buttresses and slabs. So these types of dams are often used where a future increase in reservoir capacity is contemplated and weak foundation there is.

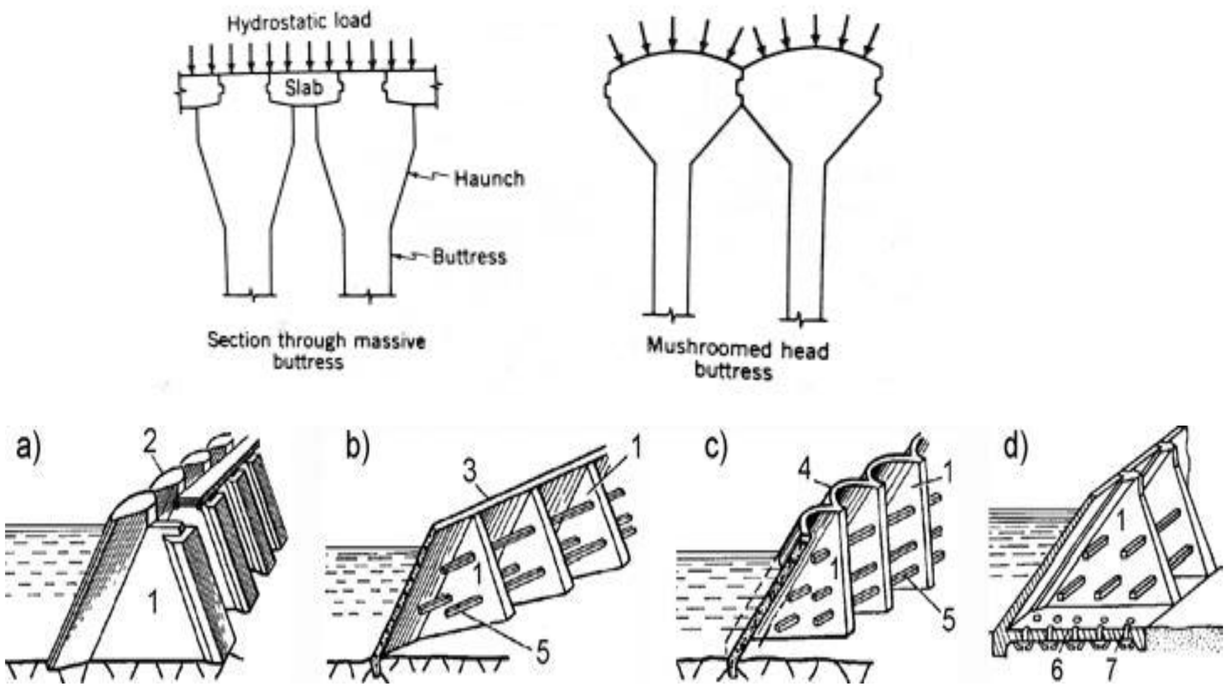

Figure 2.14 Structural elements of a flat-slab and massive buttress dam [3], [12]

Types of buttress dams. (a) Massive-head type; (b) flat-slab type; (c) multiple-arch type; (d) with a foundation slab (6). (1) Buttresses; (2) massive head; (3) flat slab; (4) arch; (5) Longitudinal stiffening beams; (7) drainage openings

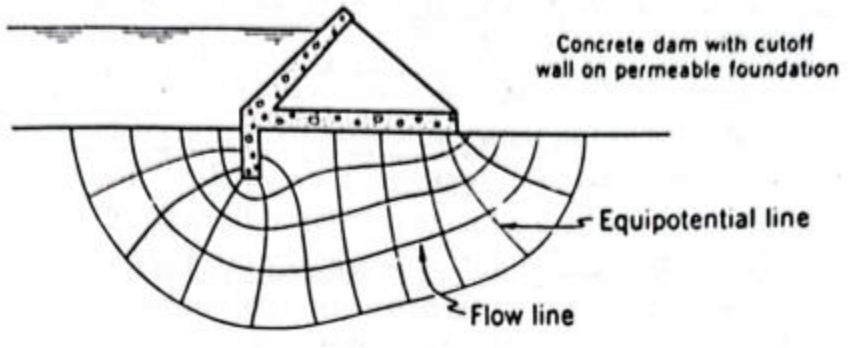

Figure 2.15 Seepage flow net under typical buttress dam when bed rock is depth [3]

Forces on buttress dams are subjected to the same forces as gravity and arch dams. Since slope of the up-stream face of the dam, Ice pressures are not usually important as the ice sheet tends to slide up the dam. Uplift pressures are relieved by the gaps between the buttresses. The total uplift forces are usually quite small and can generally be neglected except when a ***mat foundation*** is used.

Flat-Slab design

As shown in figure 2.14 the slab, or water supporting member, is designed by assuming that it consists of a series of parallel beams that act independently of one another. The slabs are not rigidly connected to the buttresses; they are designed as simply supported beams by standard methods of reinforced-concrete design. Beams thickness and amount of reinforcement required increase with depth below the water surface, since each beam is designed to withstand the component of water load normal to it [3].

Buttress design

For prevent of complex analysis, buttress design (usually are reinforced) is based on simplifying assumptions. A buttress is usually to consist of a system of independent columns. The load on each column is a combination structure load and water load. Also the columns are assumed to be curved so as to avoid eccentric loading [3].

Foundation pressures

After trial design of slab and buttresses, foundation pressures are calculated and buttress footing are design. The base of the buttresses is variate from spread footing or mat foundation may be required.

In final step the stability of the entire structure against **sliding and overturning** is controlled. If the results and safety factors are not sufficiently high, may be the slop of the slab be flattened or other parameters changed.

Arch-deck buttress dams are more **rigid** than the flat-slab type, so requires a better foundation. Arches for a multiple-arch dam are designed in the same manner as a single arch-dam, but cantilever action is commonly ignored.

Joining slab to buttress

Buttress spacing varies with height of dam about 5 m for dams under 15 m high to 15 m spacing for dams more than 50 m high. Closely spaced buttresses can be less massive, and the slabs can be thinner, but more formwork is required (figure 2.16).

The best buttresses spacing is that which gives minimum overall cost. The slab is not rigidly attached to the buttresses. The joint between the slab and buttress is filled with asphaltic putty or some flexible joint compound. So this be caused each slab to act independently, and the minor **settlement** of the foundation will not seriously harm the structure. Buttresses are usually hunched where they join the slab. Flat-slab type are particularly adapted to wide valleys where a long dam is required and foundation materials are of inferior strength. The foundation material can be ranging from fine sand to solid rock, but the maximum practical height is less on poor foundation.

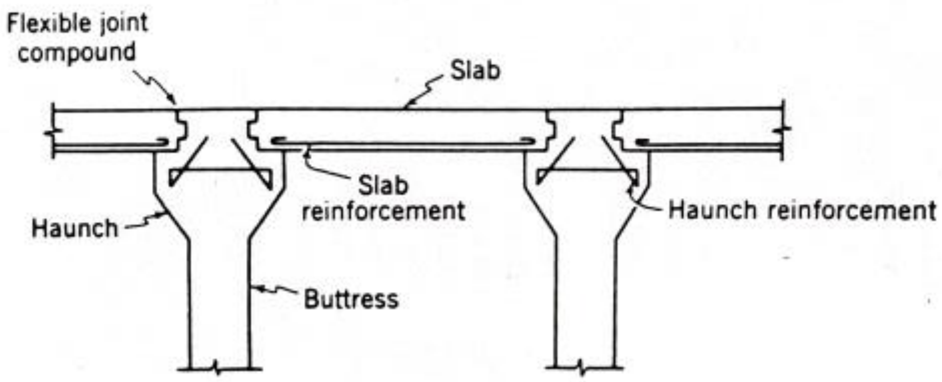

Figure 2.16 Method of joining slab to buttress [3]

Construction of buttress dams

The first steps in the construction of buttress dams is removal of overburden down to a suitable foundation and excavation of a trench for the cutoff wall. Also, since much less concrete requires for buttress dams, so the time for construction is usually less and the problems of water diversion somewhat simplified. Moreover, great care must be taken in the handling of concrete, construction of forms, and placing of reinforcing steel in order to develop fully the strength and water tightness of the thin section used in buttress dams. Deck and buttresses are placed in lifts of 3.5 m or more [3], the buttress construction being kept well in advance of the deck. Keyways are required in all construction joints.

2.8 Embankment dams in general

Also buttress dams under specific conditions may be better than of the embankment dams (figure 2.3), but main criteria are **cost** of construction and availability of **materials** to select the appropriate dam type. Totally we can say embankment dams are now competitive in cost with concrete in all sizes, although RCC dams may be preferred in some conditions. There are 2 main types of Embankment dams [3] including **earth-fill** type (the larger part of whose body "over 50%" is constructed from fine-grained earth materials-clay, loam, sand, or sandy – gravel material) and **rock-fill** type (is constructed from coarse-grained materials) [12]. Rock fill dams maybe designed and constructed with reinforced concrete facing, asphaltic concrete and other types of facing and internal non-earth core. All these types dam based on state of the foundation and availability of materials will consider and select. Basically, embankment dams utilize natural material with a minimum of processing and may be built with primitive equipment (such as **small farm earth dams** with max 10 m high [13]) under conditions where other construction materials would be impracticable.

New technology in earth-moving equipment have resulted in decreased cost for earth moving as compared with an increase in cost of concrete as a result of increased wage and material costs. Moreover, unlike high-arch and gravity dams, which require a sound rock foundation, or at least special schemes in soil foundations, embankment dams are readily adapted to earth foundation.

It should not be assumed that the construction and design of embankment dams is a simple task and their design can be rule of thumb criteria. Numerous **failures** of poorly designed earth-fill and rock-fill dams confirm that embankment dams require as much engineering skill in their conception and construction as any other type of dams. For example, continuous field observations of deformations, pore-water pressures and leakage are often made during the construction period to evaluate the initial design. Modification of design based on these observations are not uncommon for large embankment dams.

Forces and loadings

All forces that mentioned in section 2.4 may be need consider in this section, but influence of wave pressures and temperature effects may be vital and in some situations static and dynamic effects of ice pressures may be considered.

Wave pressure (P_w)
Wind itself and waves effects can be amplified when last and be power, even for concrete structures such as bridges as well as building and concrete dams. Waves be caused derangement can be produced by sliding of the mountain inside the reservoir, underwater displacement by earthquakes, by navigation vessels, high tides, etc. But waves are most often caused by wind blowing over the reservoir water surface (fetch length, L). So we must at first study wind velocity (V) and wind main direction from wind rose of the reservoir (figure 2.17).

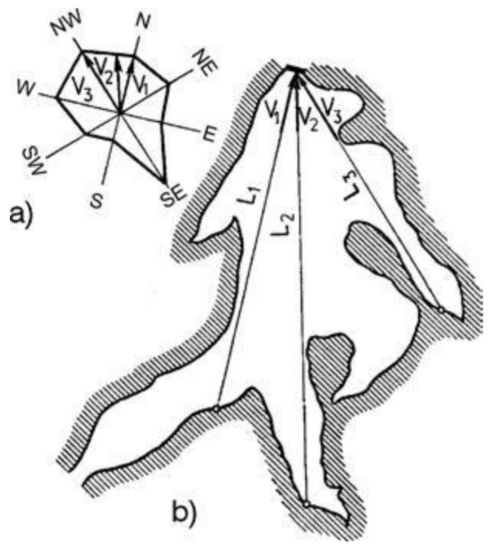

Figure 2.17 Result of wind rose study and three of fetch length [12]

Steepness of the wave is ration of the wave height and its wave length (h_w/λ), while the time interval in which the ridge of the wave displaces in a horizontal direction as much as is the length of the wave, is called the period (τ) of the wave (figure 2.18).

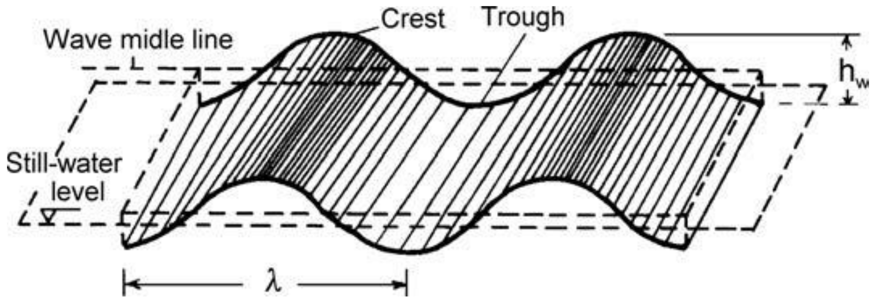

Figure 2.18 Typical wave parameters [12]

Type of the waves in river floods often are **progressive** but in the reservoirs are **standing** waves type. In the standing waves, there is no change of parameters (h_w, λ, τ) in a certain time period and they appear at a higher depth of water and as a frontal approach of regular eaves toward a vertical wall. The best known formula is modified Stephenson formula by Molitor (h_w, m):

$$h_w = 0.032\,(VL)^{0.5} + 0.76 - 0.27(L)^{1/4} \quad (L<32km) \qquad (Eq.\ 2.9)$$

$$h_w = 0.032\,(VL)^{0.5} \quad (L>32km) \qquad (Eq.\ 2.10)$$

Return period of the wind velocity (V, km/h) at a height of 10 m above the water table can identify by meteorology analysis and the fetch length is measured along a straight line from the bank to the structure.
This formula doesn't consider wave period, wave length, depth of lake, shape of the bank line, and duration of the wind and slope of the dam. Also, up until now, sufficiently accurate hydrodynamic and energy methods have not been discovered for a determination of wave parameters.
Determination of the action of standing waves upon the structure consists of a calculation of the rising of the water surface above the **static water level** ($h_{ws}=h_w 2/3$, section 2.8.1), a calculation of the pressure of the waves on the structure, as well as a determination of the overturning moment caused by those forces. For rough calculations only, the force from wave impact can be taken as an **additional** horizontal static force acting along the line of the static level of the water, in accordance with the following empirical formula:

$$P_w=2\gamma\,(h_{ws})^2 \qquad \text{(Eq. 2.11)}$$

Basis of the formula gain from hydromechanics relations. One of the best methods is the methods of Zagradskaya [12], that the reader can study more detailed information from reference 12.

Temperatures effects
This problem can be effect during construction, and operation of the dam. **Temperature variations** might arise from the adjacent environment (water, air), artificial heating or cooling, heat during curing and setting of the concrete, heat from the earth's foundation, etc.
Heat and temperature variations have a negative influence on hydraulic structures, so these factors should be carefully analyzed at the initial stage of design [12].
As a shown in figure 2.19 low temperature causes a **freezing** of earth materials, the result is an occurrence of crack and/or fractures in those materials, as well as a lowering of their strength at defrosting. Also in downstream face of an embankment dam the seepage line raises and drainage function be failed. So, to prevent this problem, drainage should be designed so as to enable the seepage line be below the depth of freezing of the earth materials [12].

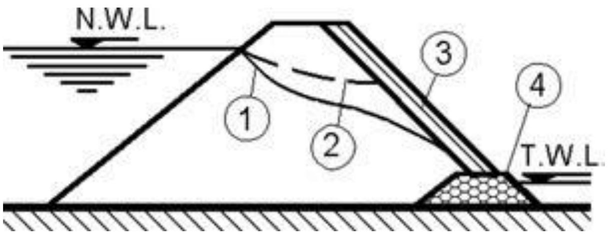

Figure 2.19 Rising of the seepage line at freezing of the downstream slope. (1) Seepage line prior to freezing, and (2) after freezing; (3) layer of frozen earth; (4) drainage. [12]

Temperature variations can effect on embankment dams that have facing made of artificial materials, especially influence on the part of the facing above the water level or on the entire facing, before filling up the impounding reservoir. Temperature in summer period could reach higher than 60^0 C, that has a harmful effect on all kinds of facing, but it is possible to use various protections in the form of thermal insulation or coating of white paint, which reflects sunbeams such as restoration of the asphalt facing of the El Grib dam – Algeria.

Tailing dams
There are 2 main methods of embankment dam construction including **normal method** that using material from borrow pit, and spreading and compacting them by appropriate equipment. Another method using checked mining of the nearby hill massif. So the tailing dams for depositing waste material that originates from mining processes are used.

The latter method can be considered and applied in highly specific conditions and rarely found in practice. For more details, the reader is referred to textbook Danilevsky, 1992 [12].

Seepage problems and Static and dynamic **stability** analysis are sensitive in embankment dams. So static stability analysis by method of creep of materials in the body and effects of earthquake on dam and on deformation and liquefaction problems must be consider under some situations [12].

2.8.1 Earth-fill dams

Small farm earth dams (High < 10 m) often design and construction **homogeneous** throughout (simple embankment) although a blanket of relatively impervious materials may be placed on the upstream face. **Levees** are often simple embankments, but large dams are usually designed and constructed based on **zoned embankment** dams (figure 2.20).

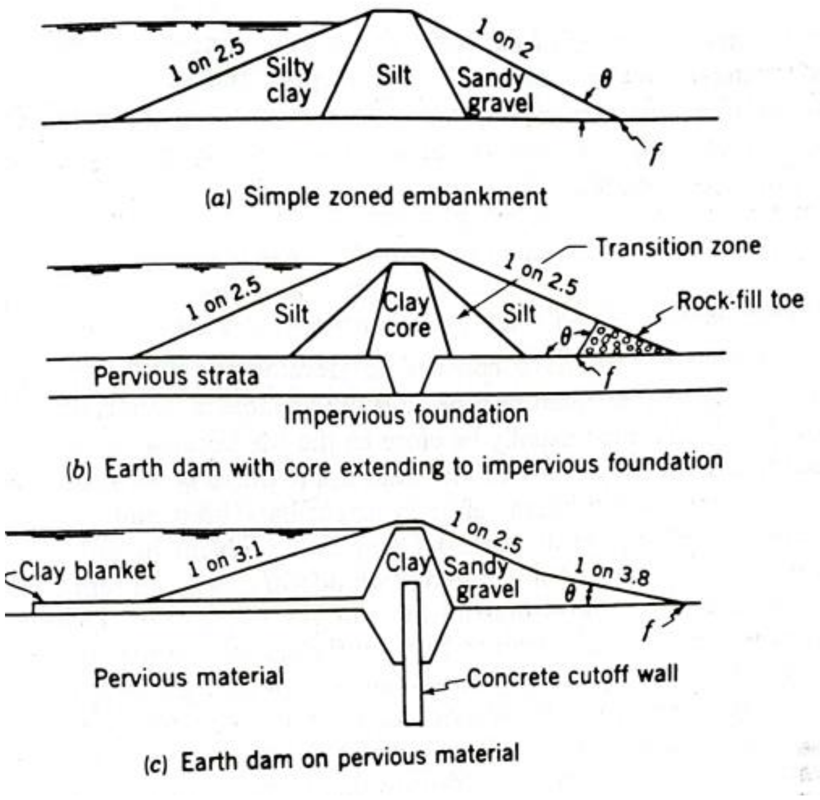

Figure 2.20 Cross section of typical earth dams [3]

Zoned embankments usually have:

1. Central zone (relatively impermeable core) of soil material
2. Transition zone along both of the core to prevent piping through cracks
3. Outer zones of more pervious material for stability.

Clay, even though highly impermeable, may not make the best core if it shrinks and swells too much. So, it is satisfactory, cores are clay mixed with sand and fine gravel, diaphragm-type dams have a thin central section of concrete, steel, or timber that serves as a water barrier. Also, thin concrete sections are easily cracked by earth loads, and it is difficult to form a perfectly watertight barrier of timber or steel. In addition, the diaphragm must be tied into bedrock or a very impermeable material if excessive under seepage is to be avoided [3].

Design of earth-fill dams

We must find best or optimum dam type based on minimum cost and other considerations such as technical, construction conditions, social, environmental, and available materials.

The structural design of an earth-fill dam is a problem in soil mechanics including assurance of stability of the fill and foundation and sufficient control of the water flow and seepage pressures in the dam foundation. Of course if the stability of the embankment is not impaired, there is little harm in seepage through a flood-control dam, but a conservation dam should be as watertight as possible.

There are some considerations in design:

- Basically it is difficult to analyze the probable behavior of natural fill materials in zoned embankment with the assurance attained in the design of concrete structures;
- Erosion control in up and down stream faces are vital;
- Empirical rules are often employed for the preliminary design of large earth-fill dams and for the final design of small dams [13];
- Final design of large earth-fill dams must involve finite element techniques that permit study of the kinds of **deformations**, **stability** analysis;
- Although the seismic resistance of an earth-fill dam is high because of its ductile behavior and large energy absorption, but in regions with earthquakes it is rational to provide a zoned earth-fill dam with side-filter transition zones between the outer shell and the dam core to prevent piping through cracks and to promote the subsequent healing of cracks suffered during an earthquake;
- Failures may also be caused by strong earthquake vibrations due to **liquefaction** of loose sand strata in the dam foundation.

Dimensions of earth-fill dams

Based on hydrologic, hydraulic and soil mechanics studies, we can gain final design of the earth dams. In here we continue and describe below elements:

- Height of dam
- Top width
- Seepage
- Pore pressure
- Slope stability
- Slope protection
- Geomembranes and geotextiles

- *Height of dam*

This element be described in chapter 1. Studies of earth-fill dam failures indicate that 40% overtopping of the dam because of:

- In sufficient freeboard (h), (chapter 1) and section 2.8.
- Inadequate spillway capacity, [1].

A freeboard allowance (h) for wind setup, waves, frost action, earthquake action and other uncertainties (h_o) must add to the reservoir hydrologic design (chapter 1). In equation 1-14, h_{wind} is relevant to the wind setup (h_d), reservoir oscillations (h_s), wave height (h_w), and wave run-up (h_t) according equation 2.12 and figure 2.21.

$$h = h_{wind} + h_{hyd} + h_{struc} + h_o \qquad \text{(Eq. 1.14)}$$

$$h_{wind} = h_d + h_s + h_w(2/3) + h_t + h_r \qquad \text{(Eq. 2.12)}$$

h_r is reserve heightening. The crest of the dam must be heightened with regards to the maximum static level of water in the impounding reservoir in order to prevent the possibility of water overflowing the dam. Also, the term $2/3h_w$ represents the rise of the wave above the static water level (h_{ws}).

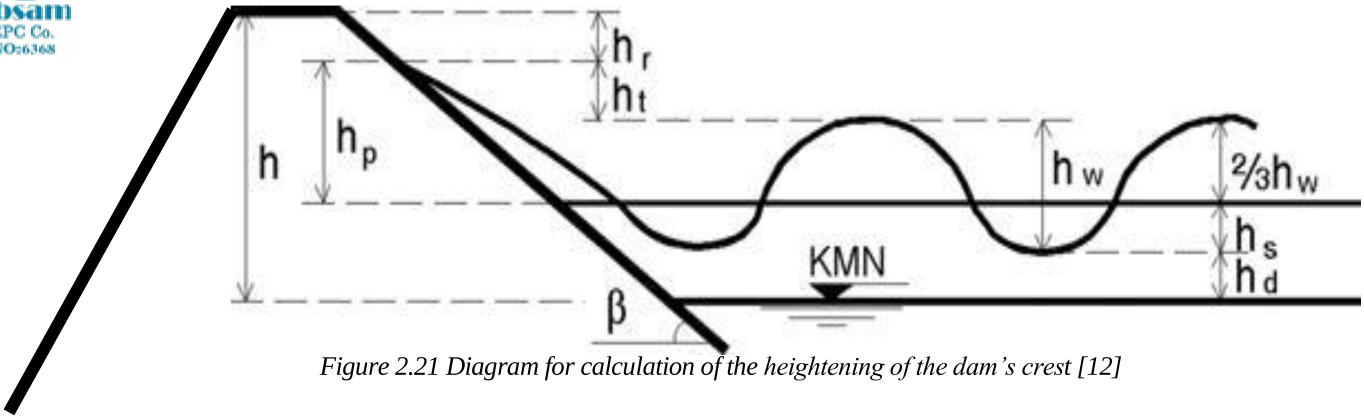

Figure 2.21 Diagram for calculation of the heightening of the dam's crest [12]

The **wind set-up** is the rise in meters above still-water level, can be calculated using the well-known Zuider Zee formula:

$$h_d = \frac{V^2}{63200D} L \cos\theta \qquad \text{(Eq. 2.13)}$$

V is wind speed (km/h), L, is fetch or effective length of water surface over which the wind blows (km), θ is angle between the direction of wind and the water surface to compute of effective fetch; and D is average depth of the lake along the fetch (m).
Wind set up is a result of tangential stresses between the wind and the water and of differences in atmospheric pressure over the reservoir. This results in hydrostatic unbalance, and a return flow at some depth must occur. The water surface slope that results is that necessary to sustain the return flow under conditions of bottom roughness and cross sectional area of flow that exist. Wind setup is generally larger in shallow reservoirs with rough bottoms. Also wind setup effects may be transferred around bends in a reservoir, and the value of fetch used may be somewhat longer than the straight line fetch [3].

Seiches or increase of the water level due to occurrence of oscillations of the reservoir surface, as a consequence of changes of the barometric pressure, alternating action of the wind, no uniform inflow and outflow, appear h_s that there are no methods for the calculation of it, so we can calculate h_w (section 2.8) or it is inserted into the reserve [12].

The height of the wave h_w can be replaced with the so-called significant height of the wave h_{sig}, so, we can use ***significant wave height*** that is the average height (m) of the highest one-third of the waves by:

$$h_{sig} = 0.005\, v^{1.06}\, L^{0.47} \qquad \text{(Eq. 2.14)}$$

v: is the wind velocity (km/h) about 7.6 m above the water surface., L is the fetch (km).
We must know, height of the waves are greater than h_{sig} in about 13% of the time, so if a more conservative design is indicated, a higher wave height may be chosen by coefficients of the table 2.1.

*Table 2.1 percentage of waves exceeding various wave height greater than h_{sig} [3] **

z/h_{sig}	1.67	1.40	1.27	1.12	1.07	1.02	1.00
Percentage of waves>z	0.4	2	4	8	10	12	13

*After Saville, McClendon, and Cochran

Since the design must be made before the reservoir is complete, wind data over land must generally be used, so the hydrologist must correct observed wind data for reservoir conditions.

Also waves are critical only when the reservoirs is near maximum levels. Thus in selecting the critical wind speed for reservoirs subject to seasonal fluctuations, only winds that can occur during the season of maximum pool levels should be considered. The direction of the wind and the adopted fetch must also be the same.

Wave run up. When a wave strikes a land slop, it will run up the slope to a height above its open water height. The amount of run up depends on the slope, roughness, and permeability of the surface, and also length and height of the wave or wave steepness, wave period and water depth. There are empirical formulae, which also contain effect of the permeability and slope parameters indirectly, so they yield only approximate results owing to the impossibility of the parameters being precisely determined. One of them is the formula of Junkovsky, which we often use:

$$h_p = 3.2Rh_w \tan\beta \qquad \text{(Eq. 2.15)}$$

Also we know from figure 2.21:

$$h_p = \frac{2}{3}h_w + h_t \qquad \text{(Eq. 2.16)}$$

Where R is a coefficient dependent upon the kind of protection of the slope – for concrete and asphalt concrete it amounts to 0.9–1; for placed stone, 0.75–0.80; while for rock fill, it is 0.55–0.60.

Additional information and techniques about the **freeboard allowances** and wind setup and wave height effects by other parameters such as duration of wind, and methods of computing the effective fetch for a narrow reservoir, or typical riprap on earth embankments are given in other references such as [14,3].

Reserve heightening (h_r) is assumed and usually ranges from 0.5 to 1.0 m.

$\mathbf{h_o}$ within itself, it should contain everything that is not included in the previous terms of expression. For example, **frost** in the upper portion of a dam causes heaving and cracking of the soil with dangerous seepage. An additional freeboard allowance up to a maximum of about 1.5 m should be provided for dams in areas subject to **low temperatures**. Also the designer may be considering this problem in drainage system or by other techniques during construction that on those situations doesn't need add this height.

$\mathbf{h_{struc}}$. **Consolidation** in earth material under load is occur but in coarse gravels the void opening is large enough to permit rapid escape of confined water and air, so full consolidation may occur before an embankment is finished. In fine-grained soils consolidation is less rapid, and it may be necessary to provide additional height of fill so that, after **settlement**, the embankment will be the desired height. The allowance consolidation can be determined by laboratory tests and observation of the settlement during construction. The usual consolidation allowance is 2 to 5% of the total height of the dam. Dewatering of the foundation material is sometimes used to accelerate consolidation. Also parapet walls with 0.5 to 1 m high are sometimes provided on the upstream side of the crest of an earth-fill dam. Such walls are considered only as an additional safety factor, but they may be constructed strongly enough to be as an element of freeboard. This latter practice is economical inly on dams exceeding about 10 m in height [3].

- *Top width (crest width)*

In designing embankment dams, it is necessary to select a stable and, at the same time, economical cross section. Dimensions of the cross-section depend on the type of the dam, its height, the kind of material in its foundation, as well as on the conditions of their construction and service. The crest and the slopes form the contour of the dam. The width of the crest can be determined by means of empirical formulae, as a function of the dam's height. Although, often these relations is not appropriate, it can be a guidance. The equation 2.17 is used in Japan:

$$b = 3.6\sqrt[3]{H} - 3 \qquad \text{(Eq. 2.17)}$$

Where b and H respectively are width of the crest, and height of the dam, in meters.

At first the top width of an earth dam should be sufficient to keep the phreatic line, or upper surface of seepage, within the dam when the reservoir is full. Also it should be sufficient to withstand earthquake shock and wave action. The smallest width of the crest for small dams can be 3–4 m (for maintenance), while for large dams it is 5–6 m. If there is a road or a railway line passing along the crest, then the width of the crest is determined by the category of the road or railway line. In such cases it usually reaches 10–12 m. Moreover, the highest embankment dam in the world, Nurek (Tajikistan), 300m high, has a crest 20m wide.

- *Seepage*

Seepage in embankment especially earth dams when is low and have not negative effect, is usual. But if the rate of pressure drop resulting from seepage exceeds the resistance of a soil particle to motion, the particle will tend to move. This results in ***piping***, the removal of finer particles usually from the region just downstream of the toe of the embankment. Some points of view:

- Piping failures in dams of clay is more than dams of silt and sand;
- Dispersive clays are highly erodible. **process of dispersion** or dispersive clay piping has caused abrupt failures of dam and levees in Oklahoma, Israel, Venezuela, Mexico, Mississippi;
- We can reduce seepage of dam body by selecting of sides correct slope that can gain based on numerical seepage analysis or study of the flow net (figure 2.22 and 2.23). In this condition, seepage flow passes of drainage system that construction by rock-fill or gravel in toe, or we can use of a very broad base, placing of an impervious blanket on the upstream face or in an appropriate place, use of a clay core, a diaphragm of timber, steel, or concrete.

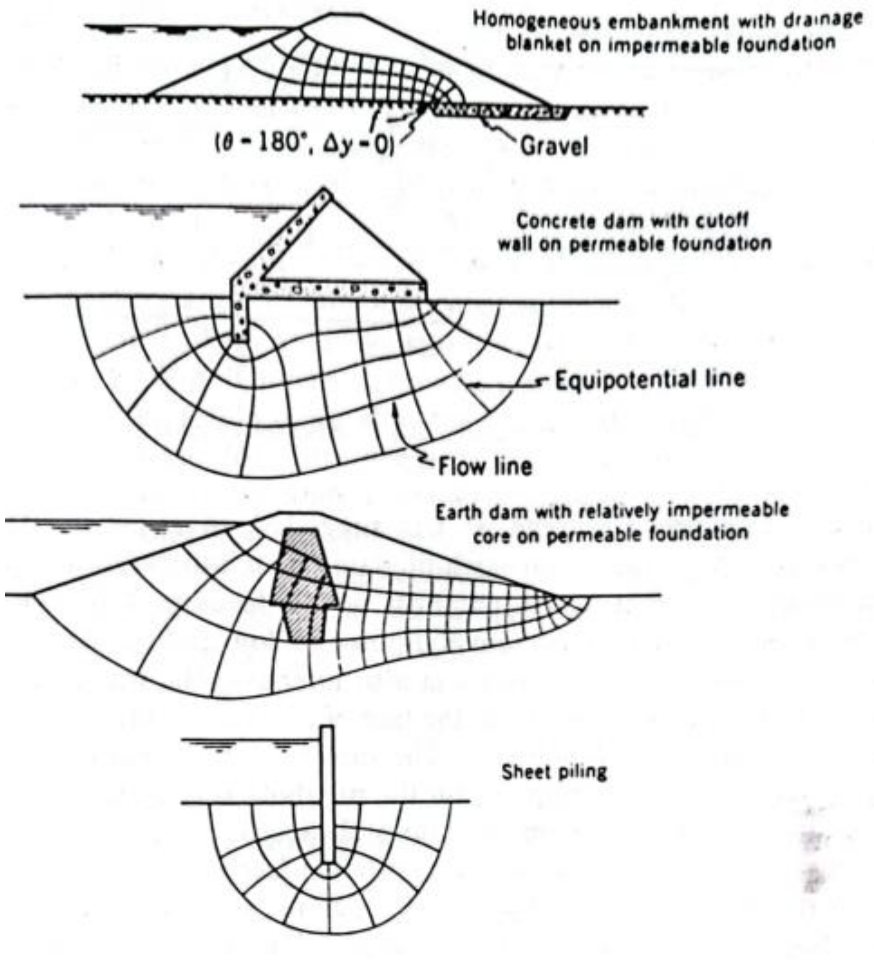

Figure 2.22 Seepage through under typical dams [3]

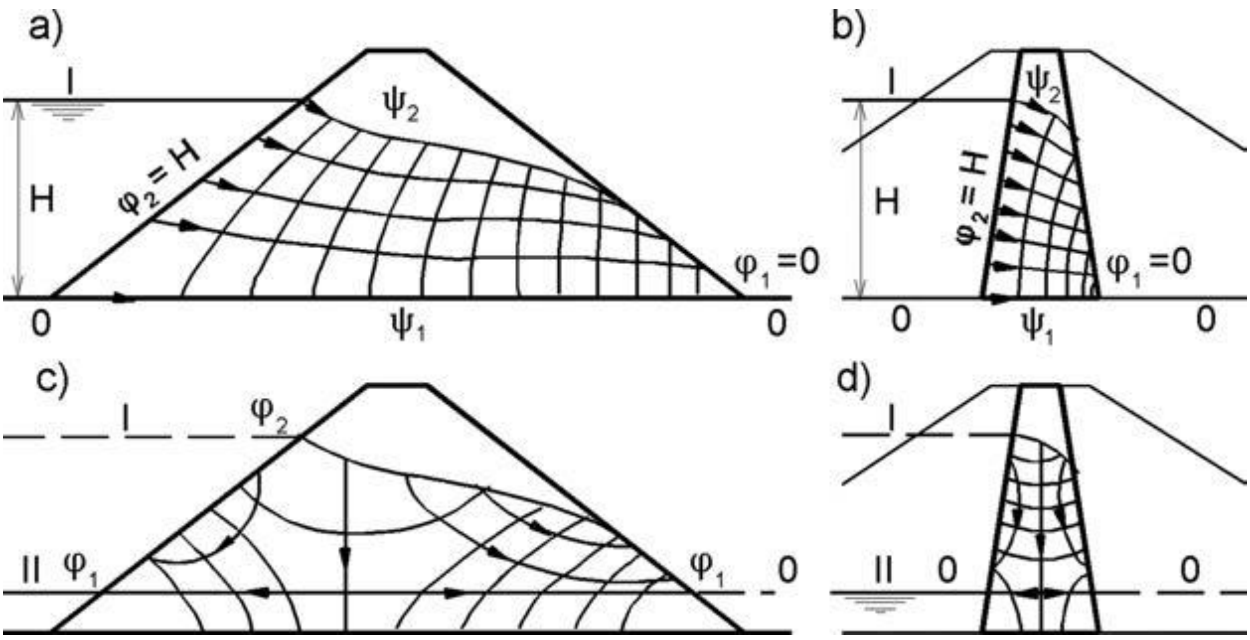

Figure 2.23 Location of the seepage line and construction of a flow net for an earth dam [12]

- Seepage of permeable foundation can control by extended blanket from upstream side dam into the foundation and connecting to the core of the dam, by one or more pile, concrete, or clay cores.
- A grout curtain formed by forcing cement grout through fractured rock to control leakage.
- Pore pressure increase when air and water and moisture of the soil, in effect of consolidation process escape and the intergranular pressure between soil particles gradually increases. When the reservoir is filled, water enters the pores and new pattern of pore pressure develops. The pore pressure distribution under steady state seepage conditions can be found from the **flow net**. For example, the pore pressure head at any point the net can be calculate to the deference in elevations or it is corresponding to the height that water would rise in a **piezometer** tube with opening at the point (figure 2.23).

- *Slope stability.*

We can design dimensions of the embankment based on empirical relations and then check those dimensions for seepage problems and slope stability of the dam. Sliding of a large mass of soil along a curved surface is a usual failure of an earth dam. Several methods for checking the stability of a fill can be found in the literature of **soil mechanics**. The simple form of the method of slices be described with assumes a condition of plane strain with failure along a cylindrical surface in figure 2.24.

- *Slope protection.*

Main aim of the slope protection is sustaining the materials in their place by riprap or concrete slabs. So effects of waves can control by riprap or concrete for example. Rip rap can **dump** or **hand-placed** that latter requires a lesser thickness and may be more economical if suitable rock is limited in quantity. The rock should be sound and not subject to rapid weathering and should be placed over a filter layer of graded gravel at least in 0.3 m thick. A filter layer of gravel to prevent the washing of fines from the dam body into the ripraps is required.

Weep holes are also required to permit escape of water when the reservoir is draw down. Also upstream –slope protection should have extended from above the upper limit of wave action to a berm or horizontal shell in the fill below the lowest anticipated pool level.

Rainfall erosion of the downstream in earth dams is a problem but in rock shell faces is not important. In earth small dams can be planted to grass as soon as possible after completion. Since the erosive action of the water can be increased with the slope length increases, berms should be constructed at about 15 m intervals of elevation to intercept rainwater and discharge it safely. This interval can calculate from erosion limit of the materials and size of the rip rap can gain based on hydraulic science.

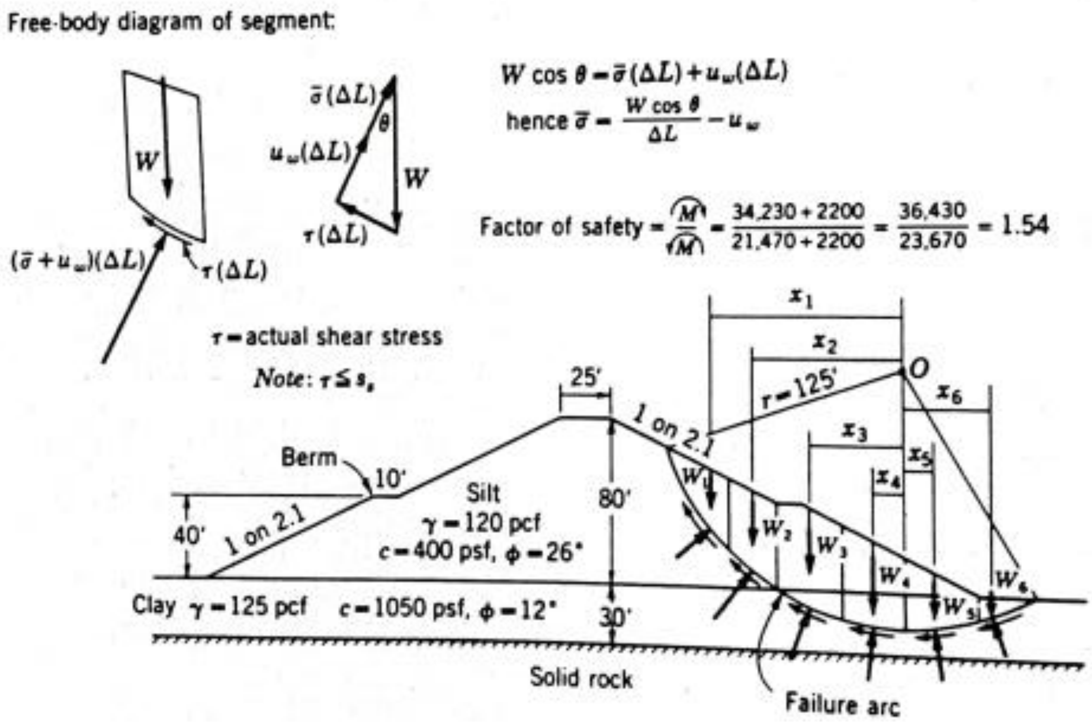

Figure 2.24 Calculations for stability by a methods of slices [3]

- *Geomembranes and geotextiles*

There are synthetic materials can be used widely in construction of the earth fill dams. A geotextile is a porous fabric of synthetic fibers while a geomembrane is impervious. These synthetics are commonly made of polymers which do not degrade when embedded in soil.
Large sheets of the latter are sometimes placed on the upstream face of an earth-fill dam to reduce seepage. Such sheets are usually protected with a layer of soil.
Geotextiles are used for a number of purposes:

- Separation
- Drainage
- Filtration
- reinforcement

Methods of construction
At first must project line of the dam site be identified. Then should be cleared of soil and loose rock and if the dam is to be built on solid rock, seams and fractures must be grouted. Common construction method is rolled-fill method. The procedure is to place selected materials in layers 15 to 45 cm and compact them with a heavy roller or trucks or sheep foot rollers and heavy pneumatic-tired rollers or in combination. Of course gravels are not suited for compaction by rolling, but vibrating equipment can be used.
In any case the material should be placed at a moisture content near that for **optimum density**. The other methods such as hydraulic-fill methods is not recommended [3].

2.8.2 Rock-fill dams

Rock-fill dams have characteristics midway between gravity dams and earth dams. A rock fill dam is one in which rocks serve as the main structural element. There are two types of rock-fill dams:

- The impervious face
- The impervious earth core (figure 2.25).

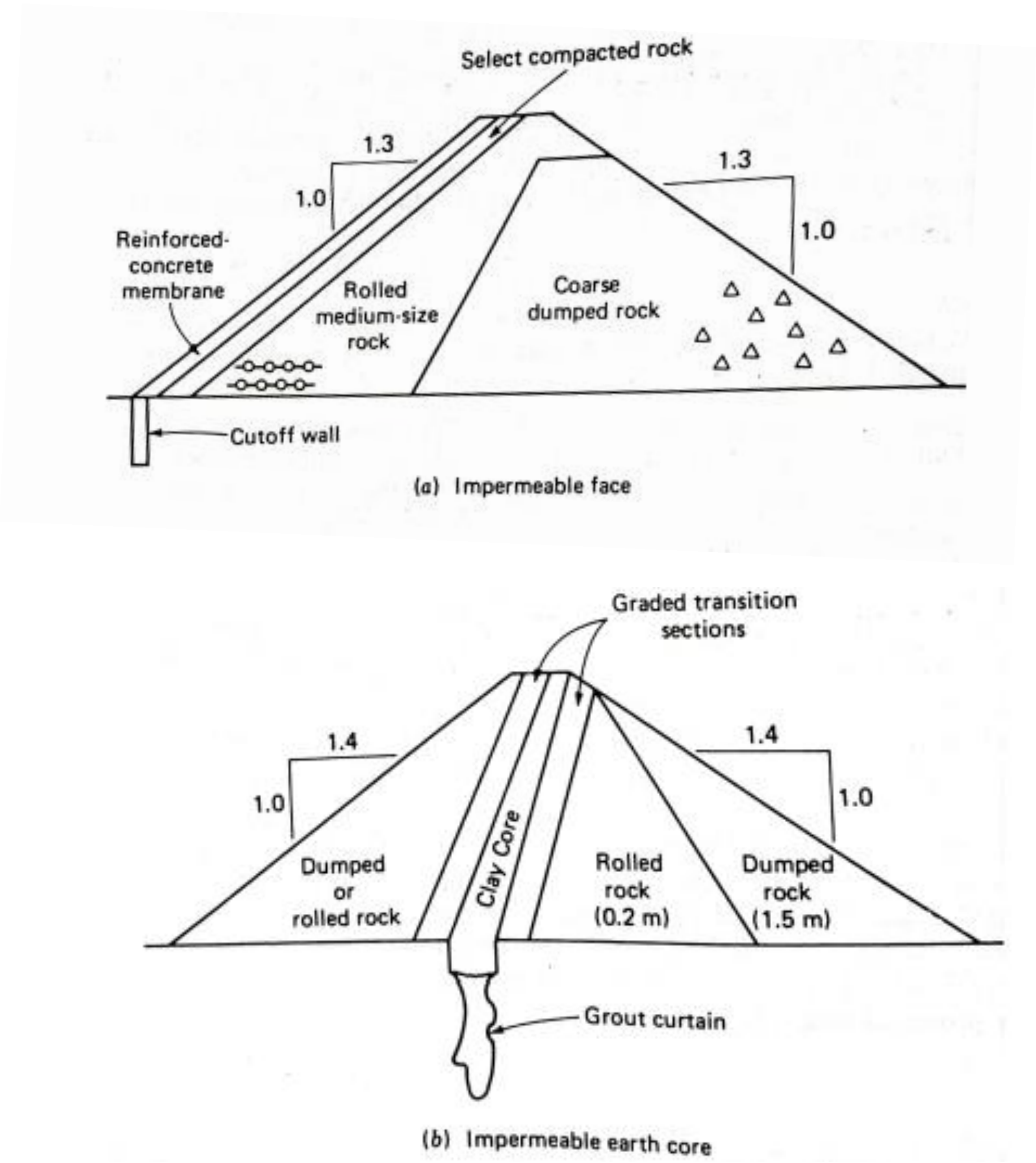

Figure 2.25 Cross sections of typical rock-fill dams [3]

2.9 Appurtenant hydraulic structures

We can mention them into below lists, so the reader can find detailed data in relevant text books.

Gates and valves
Spillways, bottom outlet and outlet works
Cofferdams
Pipeline in small dams

Other relevant structures such as power house, penstock, shafts, lift gate, stop logs and needles, ice control in spillways, collars, sluiceways, intakes, trash racks, butterfly valves, fish ways, stilling basin and baffle piers and chute blocks, surge tank, turbines, forebay, generator, pumps, flashboard, hydrant, surge chamber, electrical equipment …

2.10 Monitoring / service and schemes

After the dam construction was completed, depends on the dam type (concrete, embankment, small or large and technical data) are necessary, according to rules, do periodic monitoring and service to prevent of kinds of failure and sustainable operation and exploitation be sustained. We can divide these tasks to two category including:

1- Periodic monitoring and services or usual activities of operation, maintenance and repair
2- Especial schemes.

- Normal maintenance

Embankment dams

In the course of construction and particularly during the service period, it is necessary to perform continuous monitoring, i.e. observation, and surveillance of the embankment dam in order to have permanent insight into the condition and behaviour of the structure, enabling timely anticipation of any possible threat to its safety. Monitoring is carried out by means of measurements and keeping track of seepage phenomena, displacements and stresses both in the dam's body and in the foundation. In the case of dams impounding exceptionally large storage reservoirs, as well as those which are located in seismically active areas, it is necessary to keep track of the seismic activity before, during and after forming the reservoir. Surveillance means a continuing examination of the condition of a dam and its appurtenant structures and the review of operation, maintenance and monitoring procedures and results in order to determine whether a hazardous trend is developing or appears likely to develop (Penman et al., 1999; Singh & Varshney, 1995) [12].

Concrete dams

The objective of surveillance, monitoring, and instrumentation of concrete dams is the same as that for embankment dams. Of primary importance in surveillance and monitoring are data, on the basis of which it is possible to judge the safety of the dam; of secondary importance, is the information that might be used in designing other dams. In the case of concrete dams, we keep under observation the temperature in the dam's body, strains and deformations, opening of joints, stresses (for arch dams also in the foundation) as well as the uplift pressure of pore water, filtrated into the concrete and the foundation. Here also there are two main methods for performing measurements: (1) by means of precise instruments that measure displacements of permanent bench marks, set up on the surface of the dam, galleries, vertical shafts, tunnels in the abutments and in the measuring wells in the foundation; (2) by means of instruments that are incorporated into the dam's body and the appurtenant structures by means of which the above-cited measurements are carried out; in case of need, instruments are incorporated both into the base of the dam and into the concrete-rock interface (Goguel & Mpala, 1992) [12].

- Especial maintenance

Here there are especial **dam problems** such as:

- Dam break
- Sedimentation and flushing
- Failure of water supply
- Social, and commercial problems
- Ecology and environmental problems
- First impounding
- Flood control and river engineering
- Water erosion and watershed management
- Wind erosion
- Reservoir evaporation and leakage
- Contamination and enemy problems and thief,
- Inadequate spillway capacity and re-engineering
- Foundation failure, which includes piping
- Settlement of the foundation
- Re-engineering for improper protection against wave action, design, bad construction, lack of proper maintenance, …
- Dam collapsed because large cavities in its limestone foundation or a thin seam of clay in one abutment, instability of the banks of the reservoir, …
- A large, abrupt rock slide into reservoir can cause water to pass over the concrete dam, although the dam may be undamaged, but the flood wave (about 70 m) inundate downstream villages …
- Earthquake can occur liquefaction and the upstream face of the dam slumps to the reservoir
- …..

The best way to **assure safety** of a dam is through proper design, construction and the use of sound materials [3]. In addition, surveillance and monitoring of dams is vital. All dams ought to be inspected every few years, especially during the first filling of the reservoir short periodically.

Large vertical and horizontal movements of the crest of a dam and deformation of the embankment slopes are indicators of possible unsafe conditions.

Unusual seepage at the toe or edges of a dam or through cracks in the concrete are also indicative of possible problems, particularly if the seepage water is not clear and contains fine particles in suspension.

Piezometers have been installed in many earth-fill dams to keep track of the line of saturation. So we can calculate pore pressure and evaluate performance of the drain.

Concrete dams are sometimes provided with **strain gages** and **stress meters**, some of which are embedded in the concrete. These provide information relating to the performance of the dam that can be compared to the predicted design stresses [3].

Dam Rehabilitation and Schemes

If inspection of a dam indicates possible problems, so the ***remedial action*** should be taken at once. For instant if excess seepage see at toe so could lead to failure by piping. There are some ways:

- Could be drawn down water the reservoir and an impermeable blanket of clay or bentonite installed (figure 2.26)
- A geomembrane could be used
- Installation of a cut-off wall into the foundation near the upstream toe
- Grouting at foundation without emptying the reservoir
- Enhanced drainage at the toe of the dam

Often two or all three of these methods are used in conjunction with one another to reduce seepage through the foundation.

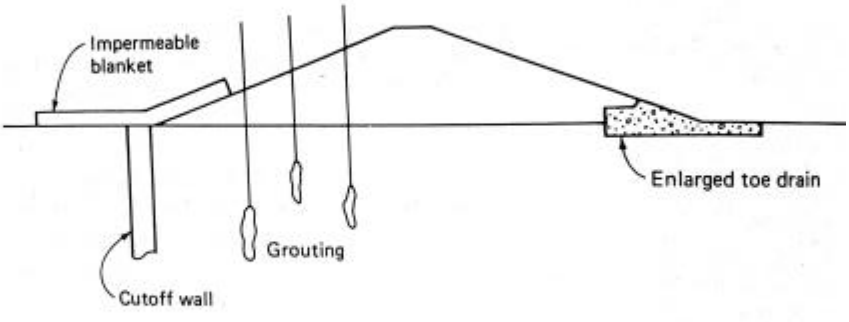

Figure 2.26 Remedial action for excess seepage through the foundation of an embankment dam: Impermeable blanket, slurry trench cutoff wall, grouting, enhancement of drainage with an enlarged toe [3]

Deformation of the crest of an embankment dam or bulging of the embankment indicates possible settlement of the foundation through fault movement or stability problems. Such problems can be remedied by an extension of the embankment (figure 2.27). The extension is usually constructed against the downstream face of the dam, though not always. Compacted earth and rock fill were formerly used and are still used for this purpose. However, roller compacted concrete is more widely used. Installation of proper drainage facilities to prevent build-up of excessive pore pressure is essential [3].

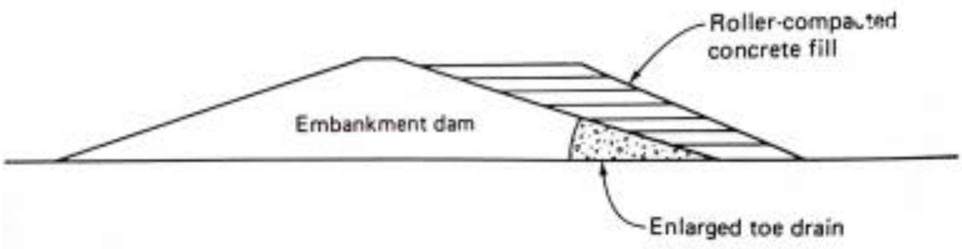

Figure 2.27 Remedial action for stability problem in an embankment dam [3]

Often concrete will deteriorate because of **chemical reactions** or in regions of severe **climatic conditions**, where concrete is tending to repeated freezing and thawing, it will crack and spall. Remedial action often involves removal of the outer layers of the deteriorated concrete followed by an application of fiber-reinforced concrete. The mortar for this concrete contains fibers, usually steel, about 1 in. in length. These fibers prevent the formation of micro cracks and provide protection against deterioration. Fiber-reinforced concrete is particularly advantageous for use where concrete is subject to erosion or damage from cavitation.

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Chapter 3

Design of flood control structures

3.1 Urban and rural measures in different river types

Urban flood control is somewhat different from rural watershed, although are even from technical point of view and when flood control problems relevant to inner rivers or when floods be transferred by channel from up-stream basins. Urban storm water management is not subject of this text and reader can find more information from other references or ref. [1,29] by the author.

Rural flood control methods, be included verity of biological, structural and systemic or optimum combined of these methods in watershed management engineering [1,30], for hazard zones or riparian of rivers and wetland or near roads, etc. We can categorize structural methods for up-stream of the small catchments (first and second small stream order) such as check dam, gabions, small rip-rap works in rivers, etc., that is useful for sediment control too, to medium and large rivers of big watersheds (river basins) such as dams, dyke, groyne, bridge, etc., that often have negative effects on environment. Other structures can use in both of the area such as weirs, culverts, retarding basins, dyke, etc. It is important that know, to use these structures, at first must evaluate criteria such as economic, environmental, and socio-technical aspects.

3.2 Dyke (levee) and roadway

A dike is an embankment constructed along the banks of a stream, river, lake or other structures for the purpose of protecting they from overflowing floodwater. Also, efficient drainage conveyance must be planned from inland to water body. Dike in some cases is used as a roadway (cause way). So, dikes generally consist of soil and sand.

3.2.1 Design criteria

In figure 3.1 shown typical parts of a dike. Design principle of embankment dikes is similar to earth dams (Chapter 2).

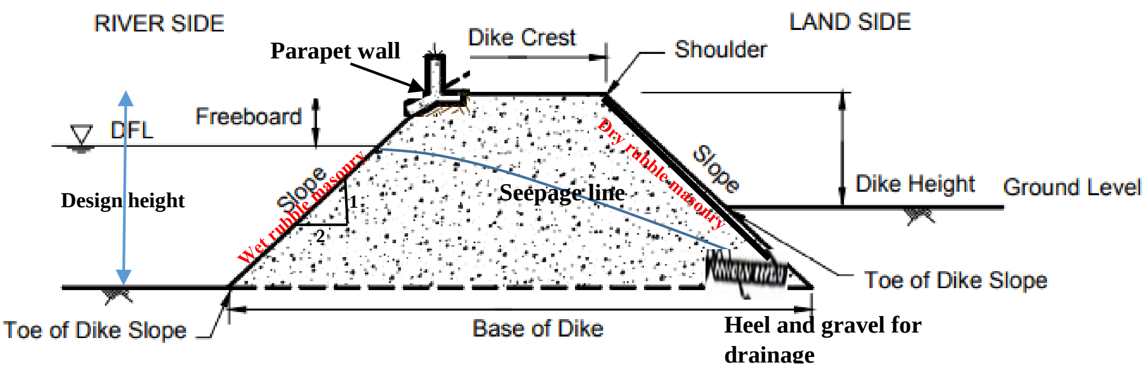

Figure 3.1 Typical of parts of dikes [1] *

*If the height of the dike is more than 5 meters, a berm shall be provided along the slopes for stability, repair and maintenance purposes. The berm width shall be 3.00 meter or more [31]

Dike design height (h_{dike}) can gain by sum level of design flood (h_{df}) [1,2], with freeboard height (h_f) (chapter 1,2), height of wind effect on producing water wave in width rivers (h_{wind}), height of effect of river bank curve or backwater by simultaneous joining of tributary on the water elevation (h_c) [32,31] (Figure 3.2), height of body and foundation subsidence (h_s) based on dam material, compaction and land conditions (Table 3.3).

$$h_{dike} = h_{df} + h_f + h_{wind} + h_c + h_s \qquad \text{(Eq. 3.1)}$$

$h_{wind,}$ can gain from Eq. 2-12.

Freeboard height can be calculated by uncertainty analysis [1] in design phase and can relevant to any subject that has not clear hydraulic response due to lack of data in project such as h_c or by table 3.1 in preliminary phase.

Table 3.1 Minimum required freeboard [31]

Design flood discharge Q (m^3/s)	Freeboard (m)
Less than 200	0.6
200 and up to 500	0.8
500 and up to 2,000	1.0
2,000 and up to 5,000	1.2
5,000 and up to 10,000	1.5
10,000 and over	2.0

For the backwater effect in a tributary, the height of the dike in the transition stretch shall not be lower than that of the main river or even higher at the confluence in order to prevent inundation in the subject areas. In general, the dike's height of the main river at the confluence point is projected following its design flood level [31].

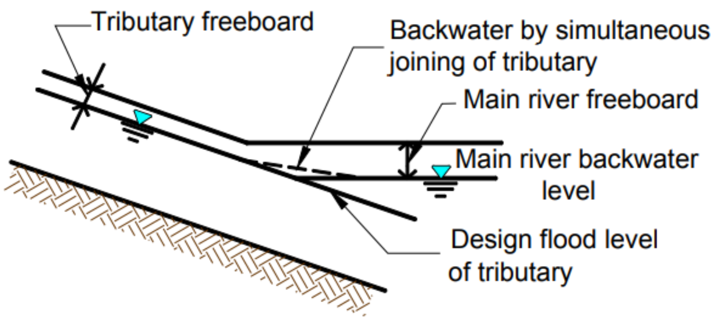

Figure 3.2 Free Board due to backwater effect [31]

Crest width. The crest width can be 3 to 7 meter depending on design flood discharge especially for wide rivers (Table 3.2) and it must be 3 m or more when the landside ground level is higher than the design flood level regardless of the design flood discharge.

Table 3.2 Crest width of dike [31]

Design flood discharge, Q (m^3/sec)	Crest Width (m)
Less than 500	3
500 and up to 2,000	4
2,000 and up to 5,000	5
5,000 and up to 10,000	6
10,000 and over	7

Crest width may be designed for multi-purpose use, such as for patrolling during floods and emergency flood prevention works or OMR activities. The base of the dike is fixed by the width of its crest and slope. Also, the body slops are depending on rest angle of construction material and percentage of compaction. Likewise, the dike shall be designed to prevent from possible collapse due to seepage (Figure 2.22) which is also dependent on the width of the dike's crest, slope, method of construction and appropriate selection of material.

Slop. In principle, the slope of the dike must design based on seepage behavior of the dike body. in this condition, seepage line will pass from correct path of the inner dike body and gravel in heel for drainage excess water (Figure 3.1).

In general, the slope of the dike shall be as gentle as possible at least lesser than 2:1 (Figure 3.1). When the crest height from riverbed is more than 6.00 meters, the slope of the dike shall be gentler than 3:1. A slope of 4:1 is usually used for a dike consisting of sand and shall be protected by providing a cover of good soil sodded at least 300 mm thick. When the surface of a dike is covered by a revetment (rip-rap), the slope of dike could be steeper than 2:1 [31].

Height of body and foundation subsidence (h_s). Extra-embankment shall be planned due to Consolidation of the dike. The standard for extra-embankment height is shown below (Table 3.3) for preliminary studies.

Table 3.3 Height of h_s in Eq. 3.1 [31]

Dike Height (m)	Dike Foundation Materials			
	Ordinary Soil		Sand/ Sand & Gravel	
	Extra Embankment Materials			
	Ordinary Soil cm	Sand/ Sand & Gravel cm	Ordinary Soil cm	Sand/ Sand & Gravel cm
≤ 3 m	20	15	15	10
3 m – 5 m	30	25	25	20
5 m – 7 m	40	35	35	30
≥7 m	50	45	45	40

In design phase of projects, it must be analyzed by geotechnical numerical methods and must pay attention to slope of the body of dike for drainage purpose.

3.2.2 Material and compaction

Suitable materials for the dike are selected based on economic consideration, workability during construction, and stability of the dike such as mechanical stability, shearing strength, resistance, compaction, durability against slope failure and cracks of dike body and without possibility of compressive deformation or expansion and toxic organic matter and water-soluble material. These cases can study by field samples and lab by ***geotechnical - soil mechanic criteria***. The following are some suitable materials, which meet the basic requirements based on experiences [31]:

- Well-graded materials: Those materials can be compacted sufficiently. Coarse-grained fractions contribute to the strength of the materials, while the fine-grained fractions contribute to the increase of the impermeability of materials.

- Materials with maximum grain-size diameter of 10 to 15 cm. Maximum grain-size diameter is determined considering the limitation of the rolling thickness during construction. If the maximum diameter is too large, the materials could not be compacted sufficiently.

- Materials with fine-grained fractions (0.075mm or less) content ratio from 15% to 50%. Fine-grained fraction is a requirement to secure the impermeability of dike. However, if fine-grained fraction is more than 50%, there is a high risk of cracks during dry condition.

- Materials with less silt fraction: Silt fraction contributes to the erosion of slope surface and slope failure due to decrease of shear strength caused by high permeability and increase of water content ratio.

3.2.3 Dike breaching and special consideration

Floodwall. Although, dikes are economical, and it will last for a long period, could be easily mixed with the ground materials, follows the ground deformation/settlement of foundation, it is easy to improve if the design flood level increase or damaged by flood, earthquake, flood walls (concrete, steel or woody body) are appropriate in urban area or in areas close to important facilities, if land acquisition is a problem (Figure 3.3)

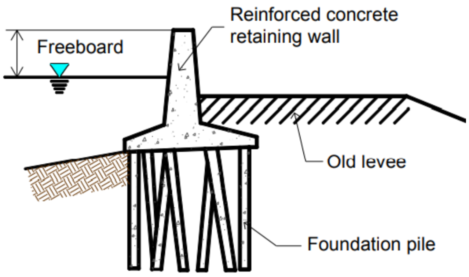

Figure 3.4 Flood well - Self-Standing Retaining Wall [31,1]

Principle of floodwall design is similar to dams in chapter 2.

Parapet wall. A return wall or parapet (Figure 3.1) is a very efficient construction built to reduce wave overtopping over see and river structures.

Main causes of **damage/breaching of dike.** The best important of dike break are wave erosion, overtopping, sand boiling, slope or subsoil failure, leakage and human activity (Chapter 1,2). Some important causes and its countermeasures are as follows in Table 3.4:

Table 3.4 Main causes of damage/breaching of dike [31]

Causes of Damage	Countermeasures
Erosion (Scouring)	The surface of the dike on both sides shall be covered with vegetation for protection against erosion. The riverside should be protected with revetment, if necessary.
Overflow	Sand bagging for emergency measure. For long term measure, provide concrete and asphalt covering for the crest and the landside slope.
Seepage	To prevent the collapse of dike caused by seepage, embankment materials for the dike should consist of impervious materials (e.g. clay) in the riverside, and pervious materials in the inland side. Drainage structures and related facilities works should be provided at the inland side to drain accumulated water.
Earthquake	Immediately repair/restoration after the earthquake.

Design new dike. Dike construction may include new dike and widening or raising existing dike (Figure 3.4).

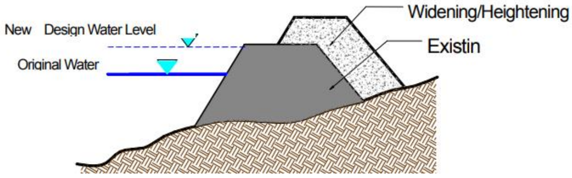

Figure 3.4 Flood well - Self-Standing Retaining Wall [31,1]

Whenever there is a necessity to heighten/widen the dike on the landside or riverside, the position depends on the alignment *as straight as possible*; however, ideally the rive side and landside is preferred especially for the toe and other standpoint technical such as distance from river, direct attack of flow, scouring, and permeable foundation should be avoided.

Maintenance Road. The dike shall be provided with a maintenance road for patrolling the river during emergency flood prevention activity. When a permanent road is to be built and the difference in height between the dike crest and the landside is below 0.6 meter; maintenance road is no longer necessary. However, the dike's crest itself can be used as a maintenance road. The maintenance road shall be 3.0 meters or more (Figure 3.5).

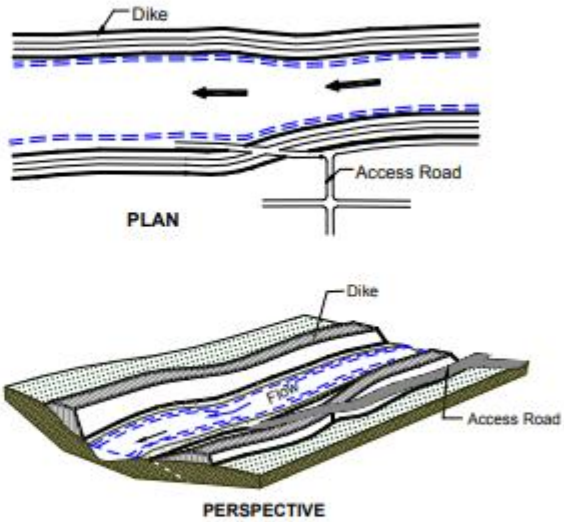

Figure 3.5 Access road and levee alignment or project line [31]

Also, access road shall be provided in portions of dike where there are human activities (i.e., quarry, fishing, agriculture, etc.). Access road shall be built properly in consideration to flood control function of a dike. Whenever possible, access road shall be constructed near the existing peripheral and/or riverside road with its entrance facing downstream side. For maintenance and other purposes, a built-in stair is also necessary. Stairway shall be strong enough to withstand the expected external forces acting on it [31].

Foundations. Investigation stages to find appropriate foundation is vital. Weak and permeable foundations van cause ***dike breach***. So, the ground condition of the proposed project line must be investigated in consideration of the foundation. Weak and permeable foundations are major issues which should be verified during the preparation and the *survey and investigation stages* (chapter 2).

Weak foundation can cause damages in and around the dike, such as sliding failure and large subsidence of the dike during construction, decrease in function of dike due to the continuous subsidence after the construction, deformation of surrounding foundation and structures, etc.

The following are the possible location of weak foundation and shall be considered during field investigation by *geotechnical field and lab tests* [31]:

a) Flat swamp/damp or paddy field area

b) Flat paddy field extending into plateau or mountainous area

c) Inland side of natural levees, sea sides, or sand dunes

Permeable Foundation. A permeable foundation can make dike prone to *breach* due to seepage at the foundation, boiling, and piping at the landside toe of the dike. The following are the possible location of a permeable foundation and shall be considered during field investigation [1]:

a) Area near a river, which may be a fan, a natural levee or a delta.

b) Traces of old rivers.

c) Landslide area with spring water or rise in groundwater level during floods.

The following can be considered as permeable foundations:

a) Surface layer consisting of sand, gravel or coarse sand.

b) Continuous sand gravel layer or coarse sand layer under a thin and impermeable surface layer.

The relationship between appropriate soil classifications of standpoint of permrabilty and the coefficient of permeability is important criteria that must gain from field activities. Generally, foundation with coefficient of permeability more than $k = 5\times 10^{-3}$ cm/s can be judged as permeable foundation.

Tidal Fluctuation (wind setup). The dike height affected by high tide (section at which design high-tide level is higher than the design flood level) shall be designed in consideration of the high-tide level plus the surge height due to wave action (Chapter 1,2). The dike affected by high tide should be generally covered on the respective three faces by concrete or similar material, taking into account the wave overtopping action. It is necessary to provide drainage at the dike's heel in order to collect local runoff and the floodwaters resulting from the wave overtopping action.

Multiply functions. The dike for special purpose, such as overflow levee, guide levee, separation levee, etc. shall be planned to allow sufficient demonstration of the functions. The height, length, width, etc. the especial dike depends on the place of construction, purpose, etc.; and therefore, must be thoroughly analyzed on a case to case basis. In some cases, *hydraulic and geotechnical model* tests, etc. must be conducted to confirm the appropriateness of the design of each structure.

Toe protection (landside). When the dike is constructed along the road or the drain, provide toe protection work, which shall have a height of 0.5 – 1.0 m and shall be made of dry stone masonry to secure the drainage in the dike body [31,1] (Figure 3.6).

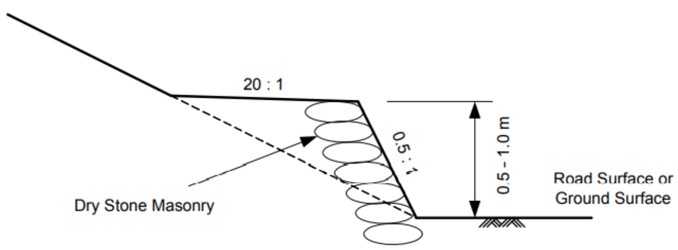

Figure 3.6 Toe protection work [31]

Piping and slope failure. During flood, the pore pressures of the dike will increase due to the seepage of the floodwater, which eventually decrease the shearing strength of dike. As a result, the safety of the dike will be decreased. ***Verification*** of the dike safety shall be carried out by the following items:

1) Safety against slope failure during the flood by the slip-circle method.

2) Safety against piping of the foundation by seepage analysis.

3.3 Revetment (Rip-rap and Gabion)

The external force which contributes to erosion depends on the river flow velocity. The revetment (Rip-rap and Gabion-Figure 3.7) protects the collapse of riverbank due to erosion, scouring and/or riverbed degradation [33,31,1].

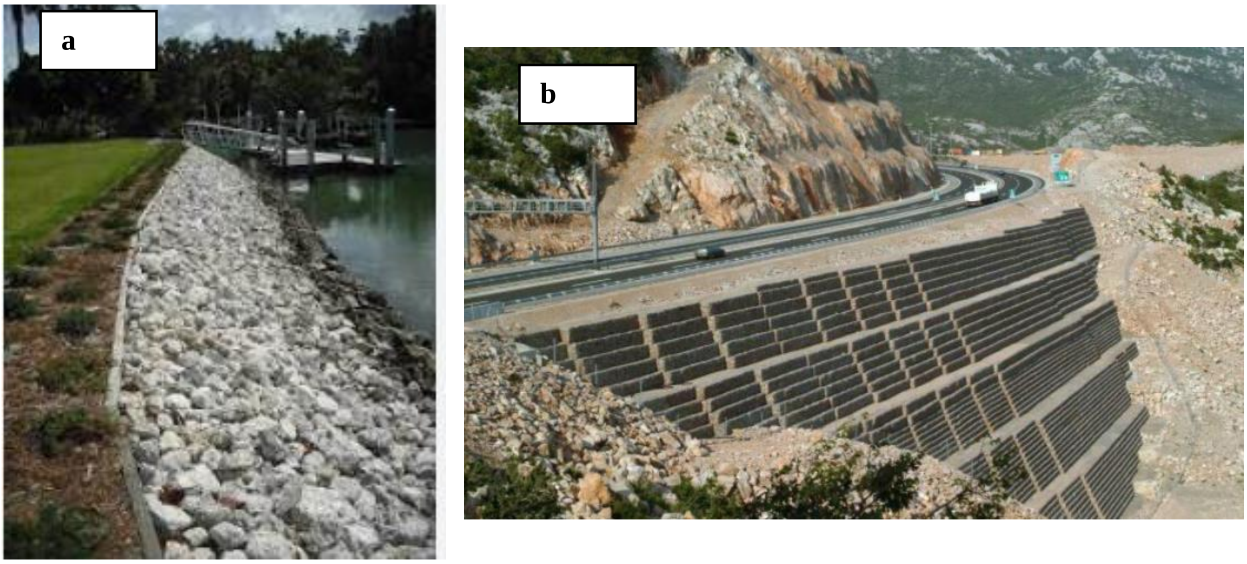

Figure 3.7 a. Rip-rap and b. Gabion. protection work [31] *

***gabion** is a cage, cylinder or box filled with rocks, concrete, or sometimes sand and soil for use in civil engineering, road building, military applications, erosion control and landscaping.

Revetment shall be designed based on the existing site conditions, such as river flow velocity and direction, embankment material, topographical, morphological, and geological conditions of the riverbank, aims of project etc. Further, revetment shall be designed to withstand the lateral forces due to high velocity flow, when located in flow attack zone, on a weak geological condition of riverbank, and with poor embankment materials or head of the ***groyne*** in water. The revetment structure shall consist of slope covering works, foundation works and head and foot protection works. The components of revetment are illustrated below, figure 3.8 and 3.9 to 3.11:

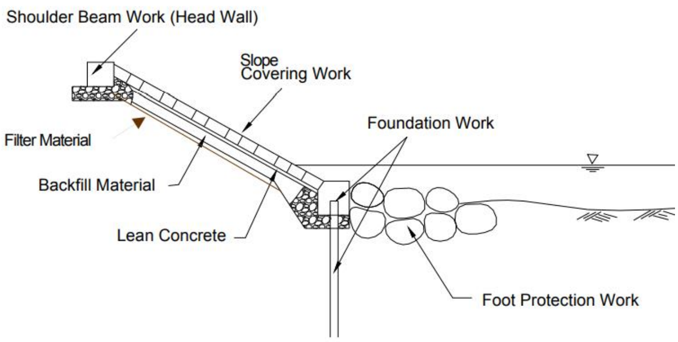

Figure 3.8 Component of typical revetment [31]

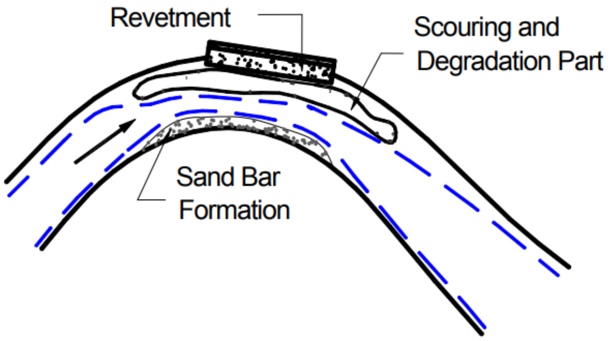

Figure 3.9 Example of construction a revetment for river Bend Erosion Control

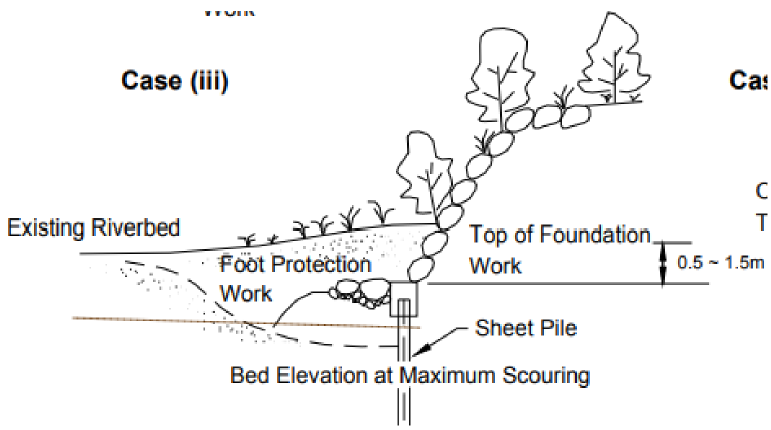

Figure 3.10 Example of construction a revetment for Foundation Work

The flow velocity and density of stones is an indispensable factor in the selection of the type of slope covering work. The mean velocity derived in the uniform flow calculation is not equal to the velocity of flow in front of the revetment, which is influenced by the effects of sand bar, bend and foot protection work, etc. To design the revetment, calculate the design velocity of the revetment using the average mean velocity as detailed describe in ref. hydraulic text books [32].

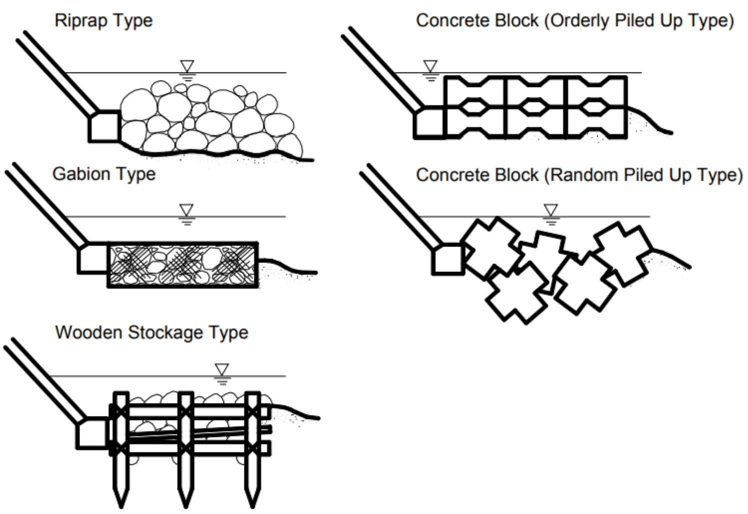

Figure 3.11 Example of construction a revetment for Foot Protection Work

Although, hydraulic parameters must calculate for design flood, in preliminary phases, the minimum diameter of the boulder for rip-rap shall be determined based on the table 3.5. The larger boulder shall be used at the toe of slope and slope surface. Outflow of materials from the foundation is unavoidable; therefore, proper maintenance shall be carried out.

Table 3.5 Minimum Diameter of Boulder (Riprap Type) [31]

Design Velocity (m/s)	Diameter (cm)
2	-
3	30
4	50
5	80
6	120

For gabion type (Figure 3.11) shall not be used for rivers with saline water intrusion and for rivers with riverbed and banks consisting of boulders. The gabions shall be connected to each other. The diameter of the filling boulders shall be determined based on the table 3.6.

Table 3.6 Diameter of Filling Boulder (Gabion Type) (Unit: cm) [31]

Water Depth (m)	Design Velocity (m/s)					
	1.0	2.0	3.0	4.0	5.0	6.0
1.0	5-15	5-15	5-15	10-20	-	-
2.0	5-15	5-15	5-15	5-15	15-20	-
3.0	5-15	5-15	5-15	5-15	15-20	15-20
4.0	5-15	5-15	5-15	5-15	5-15	15-20
5.0	5-15	5-15	5-15	5-15	5-15	15-20
6.0	5-15	5-15	5-15	5-15	5-15	15-20

for identify diameter other type in figure 3.11 ref. to [31].

3.4 Groyne (Epi) or spur dike. Functions of epi are increasing of flow roughness and so reduces the flow velocity, increasing sedimentation and progressing riverbank toward river, increasing water level and redirects river flow from riverbank. So, it can has following purposes: 1. prevents of bank erosion and deepens water depth for navigation (Figure 3.12).

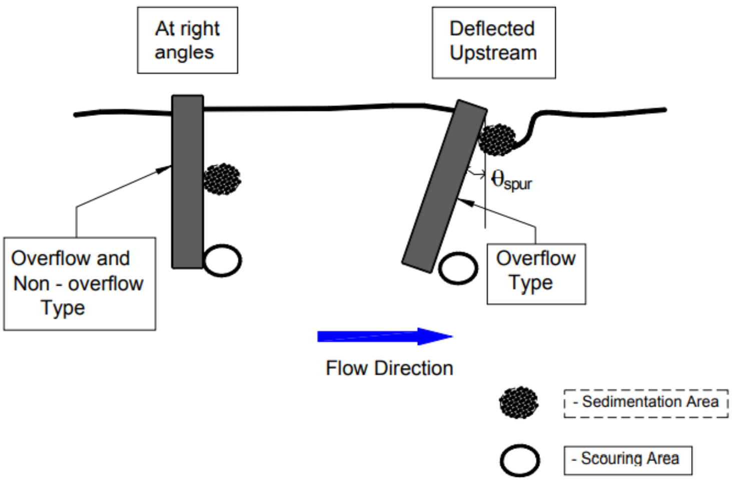

Figure 3.12 Relationship between spur dike alignment and resulting sedimentation scouring [31]

Basically, spur dikes are classified into ***permeable and impermeable/semi-permeable, overflow and non-overflow*** types and depending on problem and river condition it can design based on flow velocity and density of materials [31].

3.5 kinds of check dam

A check dam in watershed management (usually made from concrete or gabions) is a barrier placed in an actively eroding gully or minor streams to trap sediment carried down the gully during periodic flow events (Fig. 3.13).

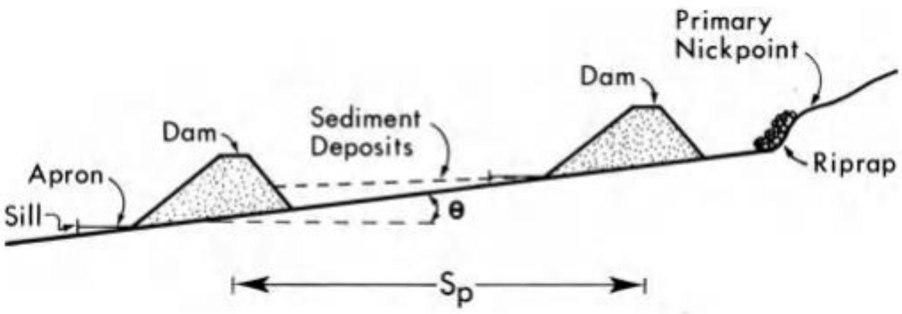

Figure 3.13 Diagram of placement of check dams: S_p: spacing, θ: angle of gully [34]

Sediment accumulations behind a check dam function to [34]:

- develop a new channel bottom with a gentler gradient than the original gully bottom and hence reduces the velocity and the erosive force of gully flow;
- stabilize the side slopes of the gully and encourages their adjustment to their natural angle of repose, reducing further erosion of the channel banks;
- promote the establishment of vegetation on the gully slopes and bottom; and
- store soil water so that the water table can be raised, enhancing vegetative growth outside the gully.

Principle of design of one check dam is similar to gravity dams (Chapter 2). But design of series of check dams (Figure 3.13) based on concept of initial of surface soil erosion (Figure 3.14) that is relevant to soil condition, slope, and intensity of rain. Also, Heede and Mufich (1973) developed the methodology to calculate the spacing of check dams [34].

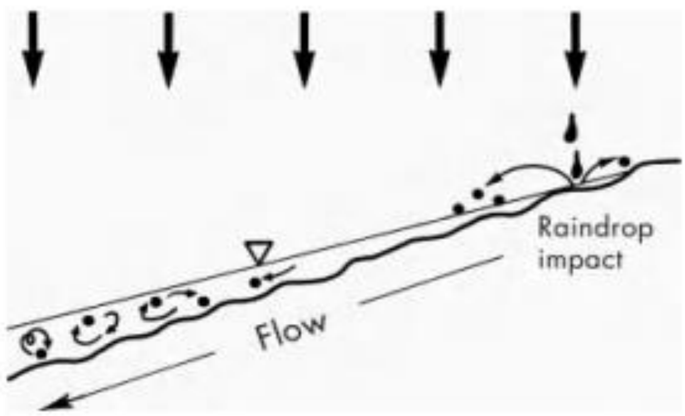

Figure 3.13 Surface soil erosion as a result of raindrop impact and turbulent surface [34]

With the exception of gully plugs, an effective check dam should consist of three essential elements:

1. A spillway adequate to carry a selected design flow.

2. A key that anchors the structure into the bottom and sides of the gully.

3. An apron that absorbs the impact of water from the spillway and prevents undercutting of the structure.

3.6 Other river structures (fuse plug, bridge, hard point, sill, weir, bypass, retarding basins, ...

A ***fuse plug*** is a collapsible dam installed on spillways in dams to increase the dam's capacity. There are many hydraulic text books about bridge hydraulic, culverts, hard point, etc. So, the reader can use them for more information [1].

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The hydrologist is guided introduction of damming and different aspects of water planning, design, construction and maintenance, embankment and concrete dams and appurtenant hydro-mechanical equipment and remediation by special schemes. This book is categorized in 3 main items.
I. Dam Hydrologic Design
II. Principle of Damming
III. Design of river flood control structures

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