Embankment Dams

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Course Contents

- Part I:

1. General
   - Stages of Investigation
   - Inputs
   - Questions to answer during investigation

2. Investigation
   - Importance
   - Preliminary investigation
   - Ideal Condition
   - Basic Data

Course Contents

- Part I (continued):
  3. Geology
     - Local
     - Regional
     - Karsts
     - Faults
     - Geotechnical & Geophysical Works
     - R.I.E (Reservoir Induced Earthquake)

Course Contents

- Part I (continued):

  4. Foundation
     - Settlement
     - Strength
     - Permeability
     - Jacking Test
     - Lugeon Test
     - Curtain Grouting
     - Consolidation Grouting
     - Cut-off walls
     - Clay Blanket

Course Contents

- Part I (continued):

5. Embankment Design
   - Earthfill
   - Earth & Rockfill
   - Vertical Core
   - Sloping Core
   - Asphaltic Core
   - Concrete-Faced Rockfill Dam(CFRD)
   - Asphatic Faced Rockfill Dam (AFRD)
   - Filter Criteria
   - Transition Zones
   - Freeboard
   - Riprap

Course Contents

- Part I (continued):

6. Construction
    - Construction Method
    - Test Embankment

Course Contents

- Part II:
  1. Embankment Simulation (in layers)
  2. Steady State Seepage
  3. Rapid Draw-Down Seepage Analysis
  4. Upstream/Downstream Slope Stability Analysis
  5. Stress-Strain Analysis During Various Phases
  6. Behaviour of Dam Body During Earthquake:
     - Newmark Method
     - Dynamic Analysis Using Acceleration Time History

References

1. Earth & Earth Rockfill Dams (Sherard)
2. The Engineering of Large Dams (Thomas)
3. Earthquake Engineering For Large Dams (Pris et al)
4. Embankment Dam Engineering (Casagrande volume)
5. Earth & Rockfill Dams (Kutzner)
6. Geotechnical Engineering of Dams (Fell et al)

Course Evaluation

✓ Homeworks	20 %
✓ Presentation	15 %
✓ Project	20 %
✓ Final	45 %
Total	100 %

تعدادی از بند های قدیمی ایران

نام بند	مکان	عمر(سال)
میزان	شوشتر	۱۷۰۰
گرگر	شوشتر	۱۷۰۰
شادروان	شوشتر	۱۷۰۰
امیر	فارس	۱۰۰۰
کبار	قم	۷۰۰
فریمان	فریمان	۴۰۰
کریت(قوسی)	طبس	۴۰۰
خواجو	اصفهان	۳۵۴
سلاحی	خواف(خراسان)	۳۰۰
پلدشت	پلدشت(آذربایجان غربی)	۳۰۰
عمرشاه	بیرجند	۲۰۰

تعدادی از سد های تازه تاسیس یا در حال احداث ایران

نام سد	محل	نوع سد و جنس	دبی (Q_{ave}) m^3/s	ارتفاع m	حجم مخزن $m^3 \times 10^6$	طول تاج سد	عرض تاج سد
چم گردکان	ایلام	سنگریزه ای با هسته رسی	-	۶۰	۵۹	۱۵۰	۱۰
تهم	زنجان	خاکی با هسته نفوذناپذیر	-	۱۲۰	۸۷	۴۵۱	۱۲
علویان	مراغه	سنگریزه ای با هسته رسی	۴.۶	۸۰	۶۰	۹۳۵	۱۰
میرزای شیرازی	کوار	CFRD	۷.۲	۶۵	۲۴۲	۲۳۳	۷
کرخه	اندیمشک	خاکی با هسته رسی	–	۱۲۷	۷۶۰۰	۳۰۳۰	۱۲

Storage Dams

- "They retain a significant volume of water"
  - Generally high dams as apposed to diversion dams which are of low heights and low volume

Storage Dams

- Storage Dams maybe built for:
  - Flood Control
  - Power
  - Irrigation
  - Navigation
  - Municipal & Industrial water supply
  - Recreational Benefits

❖ Generally a Storage Dam is a multi-purpose dam

Geotechnical Input During Stages of Development of Dam Project

Stage 1	• Pre-Feasibility
Stage 2	• Feasibility & Site Selection
Stage 3	• Design & Specifications
Stage 4	• Construction
Stage 5	• Operation

Geotechnical Input During Stages of Development of Dam Project

- ✓ Stage 1 Pre-Feasibility

  - ➢ Geotechnical Objectives & Activities:
    - Selection of possible sites
    - Understanding geological situation
    - Possible and Suitable dam types
    - Review of existing data
    - Air and ground inspection
    - Plan the feasibility and site selection studies

Geotechnical Input During Stages of Development of Dam Project

✓ Stage 2 **Feasibility & Site Selection**

➢ Geotechnical Objectives & Activities:

- Assess feasibility from geotechnical view point considering both local and regional situation
- Explore alternative sites for dam and other key structures adopt the most promising one
- Explore these sites further to confirm feasibility & provide sufficient data for preliminary design compare cost estimate
- Provide regular reports and final reports include "project feasibility" and purposed site, dam axes and dam type.

Geotechnical Input During Stages of Development of Dam Project

✓ Stage 3 **Design & Specifications**

➢ Geotechnical Objectives & Activities:

- Answer any questions arised from feasibility studies
- Further site investigation & testing usually necessary
- Provide regular reports
- Tender documents

Geotechnical Input During Stages of Development of Dam Project

✓ Stage 4 **Construction**

- Geotechnical Objectives & Activities:
  - Ensure the geotechnical picture is as assumed in design; if not; modify
  - Advise continuously the resident engineer
  - Records of movement , water flow , etc. in the progress report
  - Detailed mapping , color photos , instrument , etc.

Geotechnical Input During Stages of Development of Dam Project

✓ Stage 5 **Operation**

- Geotechnical Objectives & Activities:
  - Ensure structure is performing as designed
  - Design of remedial measures
  - Monitoring program
  - Monitoring report

Geotechnical Engineering Questions to Answer During Investigation

1. Source of Material

   - Earthfill ; Impervious core
   - Filters
   - Rockfill
   - Riprap
   - Concrete aggregates
   - Pavement

Geotechnical Engineering Questions to Answer During Investigation

2. Reservoir

- Water tightness
- Effect on groundwater ; levels , quality
- Stability of slopes ; inside & outside the rims
- Erodibility of soils
- Siltation rates

Geotechnical Engineering Questions to Answer During Investigation

3. Embankment

- Location ; to suit topography & geology conditions
- Alternative sites for comparison
- Depth to suitable foundation
- Nature of material to be excavated , methods
- Stability of excavations
- Permeability , Erodibility , Compressibility of foundation
- Foundation treatment required:
  - Grouting, Slurries , Blankets , …

Geotechnical Engineering Questions to Answer During Investigation

3. Embankment (continued)

    - Embankment zones
    - Stability of embankment + foundation
    - Monitoring systems ; types , siting

Geotechnical Engineering Questions to Answer During Investigation

4. Spillway , River Diversion , Inlet and outlet works

    - Location and type
    - Excavation method
    - Stability of excavation
    - Need for lining of tunnels

Geotechnical Engineering Questions to Answer During Investigation

5. Seismicity of region

    - Design earthquake (DBE)
    - Maximum Credible Earthquake (MCE)

Considerations in Choice of Site

- Water supply
  - Stream flow from hydrometric records and hydrological analysis

- Topography of the site
  - ✓ Controls:
    - Available head
    - Available storage
    - Volume of dam body
    - layout of :
      - Power house
      - Spillway
      - Outlet works

Considerations in Choice of Site

Considerations in Choice of Site

- Foundation
  - Adequate bearing capacity
  - Low permeability
  - No geological faults (Preferred)
  - No risk of seismic activity
  - Low compressibility

- Availability of material
- Land cost
- Environmental impact

Investigations

- Importance:
  - Lack of understanding how a damsite will react would endanger the project and may cause failure:
    - Reservoir
    - Dam
- "The most important task"
  - Extensive field investigation
  - Liaison with the designers

Otherwise it may misdirect and fail to reveal basic weaknesses

Preliminary Investigations

- Topography
  - Ideal Conditions Includes:
    - Narrow gorge
    - Valley opening upstream to provide for the required storage
    - High abutments , well above normal pool level
    - High flanks around the reservoir with long seepage path to neighboring valleys.
    - No depressions requiring lateral dams

Preliminary Investigations

- Using:
  - Topographical maps
  - Geological maps
  - Air photos
  - Satellite photographs (fault , geological features)
  - Helicopter flight
  - Active faults map

Preliminary Investigations

- Features to be sought:
  - Potential landslide
  - Old land slide
  - Faults (active ,inactive ,R.I.E)
  - Major joints
  - Stress relief
  - Weathering
  - Karsts
  - Strike & dip of formations & Joints (Stability purpose)
  - Springs (could provide paths for leakage)
  - Alluvium depth

Preliminary Investigations

- Models :

  - Topographical
  - Geological (to help understand in 3-D)
  - Computer 3-D

  - ✓ Must be updated as more information becomes available
  - ✓ Money required 6% of the cost of the dam
  - ✓ Time 3 years is not unusual

Preliminary Investigations

- Basic Concept :

  - Stability (Dam ,Foundation ,Abutment)
  - Watertight reservoir
  - Strength
  - Hydraulic gradient

Preliminary Investigations

✓ Hydraulic gradient

- Sound foundation is required & grout curtain to reduce the gradient
- Excessive care needed on poor foundation against piping & seepage
- Allowable gradient can only be decided with regard to geological formations and material properties
- Hydraulic gradient in the abutment is of concern inducing high pressures ,piping ,dislodgment of abutment rock

Preliminary Investigations

- ✓ Hydraulic gradient

Preliminary Investigations

✓ Watertightness of reservoir :

- Enough research should be made and if in limestone formation, detailed studies are required

Basic Data

- Topography
- Meteorology & Hydrology
- Geology & Seismicity
- Finance
- Environmental Assessment

Basic Data

- Topography :

  - To determine the reservoir volume at the various levels
  - Existance of low saddles around perimeter
  - Damsite :

    Quantities of excavation and dam material, layout of access road, setting out the dam

Basic Data

- Meteorology & Hydrology :

  - Q_{min} , Q_{ave} , Q_{max} for:
    - Dam dimension
    - Diversion tunnel
    - Cofferdams
  - Wind velocity -Freeboard
  - PMF (Probable Maximum Flood) - Spillway
  - Best accuracy is essential

Basic Data

- Geology & Seismicity:
  - Landslides
  - Limestone
  - Joint patterns
  - Faults (Active?, activity during the past 10,000 years?)

  (Information are further supplied during construction when excavations are started)

  - Color photography of cores in their boxes are recommended
  - Photographs before & after any foundation treatment
  - Install seismographs near the proposed damsite
  - Reservoir induced seismicity ; MCE, DBE, MDE

Basic Data

- Economics :

  - Comparative estimates
  - Purposes :
    - Agriculture
    - Electricity
    - Water supply

Basic Data

- Definitions:
  - ✓ Normal water level (Full supply level):

    Max. storage retention level corresponding to the crest of spillway
  - ✓ Flood level:
  - ✓ Height of Dam :

    H (ICOLD):

    Lowest point of foundation to the top of dam , excludes parpet wall , camber , guard rails …

Basic Data

- Height of Dam :
  - Freeboard:

    -Above N.W.L :

    – Pounding during flood

    – Wind setup in reservoir

    – Waves induced by wind

    – Waves induced by earthquakes or their effects such as landslide

Basic Data

- Height of Dam :
  - International Commission On Large Dams (ICOLD):

For large dams:
$$\begin{cases} H \geq 15m \\ \text{or} \\ Reservoir > 3 \times 10^6 \ m^3 \end{cases}$$

Selection of Type of Dam

a)

1. Environment
2. Weather
3. Money & time
4. Availability of material
5. Unavailability of skill
6. Seismicity : Rockfill
7. Geology
   - Strong abutment for arch dam is necessary
   - Differential deformation of foundation
8. Hydrology (Possibility of inundation during construction)
9. Cost

Selection of Type of Dam

b) Embankment Dam

- Types:
  - ✓ Earthfill , Rockfill , Hydraulic fill

- Definition:
  - ✓ A dam constructed of natural excavated material placed without addition of binding material other than those inherent in the material itself

Selection of Type of Dam

b) Embankment Dam (continued)

- Earthfill Dam:
  - ✓ Constructed primarily of compacted earth in either homogeneous or zoned areas ,containing more than 50% of earth
- Hydraulic Fill Dam:
  - ✓ Constructed of earth ,sand ,gravel or rock generally from dredge material conveyed to the site by suspension

Selection of Type of Dam

b) Embankment Dam (continued)

- Concrete Dam:
  - ✓ Arch ,Gravity , buttress, …

- Rockfill Dam:
  - ✓ An embankment type of dam which depend for its stability, primarily on rocks

    Contain more than 50% of compacted or damped rockfill
  - ✓ Also CFRD ; Bituminous concrete faced R.D

Selection of Type of Dam

c) Valley Shape

- Gorge $\frac{W}{H} < 3$
- Narrow valley $\frac{W}{H} = 3 - 6$
- Wide valley $\frac{W}{H} > 6$

W: crest width

H: height below crest

Selection of Type of Dam

✓ Valley Shape Factor:

$$K = \frac{b}{H} + \sec\phi_1 + \sec\phi_2 = \frac{b + H\sec\phi_1 + H\sec\phi_2}{H}$$

- $b > 2H$ Wide valley
- $b < 2H$ Composite; U-V Shape $\phi_1, \phi_2 > 15$
- $b < H$ U Shape $\phi_1, \phi_2 < 15$
- $b \cong 0$ V Shape

❖ Used mainly for concrete dams

Selection of Type of Dam

d) Rock Quality

- Foundation material should be strong enough
- For concrete dams & arch dam foundation strength should be over 70-100 kg/cm^2
- Existence of joints, faults, bedding, control, the load bearing capacity and deformation also on the infilling materials

e) Rock Joint Pattern

- Foundation sliding (concrete dams)

f) Other Features (length of diversion tunnel ,…)

Some Comments on Freeboard

- Objectives (USBR 1981)

  - Wind setup
  - Wave setup
  - Landslide & Seismic effect
  - Settlement
  - Malfunction of structures
  - Other uncertainties

Some Comments on Freeboard

- Other factors that may influence selection of freeboard include

  - Reliability of design flood estimates
  - Assumption in flood routing
  - Type of dam & susceptibility of erosion
  - Potential changes in design flood

Some Comments on Freeboard

- Freeboard
  - Normal
  - Minimum

✓ Normal:

1. Wind setup & wave run up for max. wind + possible settlement not included in the camber

   or

2. Landslide generated water waves + possible settlement not included in camber

Some Comments on Freeboard

- Freeboard
  - Normal
  - Minimum

✓ Normal:

1. Wind setup & wave run up for max. wind + possible settlement not included in the camber

   or

2. Landslide generated water waves + possible settlement not included in camber

Some Comments on Freeboard

- Definition
  - Vertical distance between a specified water surface & top of the dam , without allowance for camber
  - Vertical distance between reservoir water level and the crest of the dam without camber

  - Preliminary design values (USBR 1977)

Largest Fetch (km)	Normal F_B (m)	Minimum F_B (m)
< 1.6	1.2	0.9
1.6	1.5	1.2
4	1.8	1.5
8	2.4	1.8
16	3.0	2.1

Some Comments on Freeboard

- Notes

a) It is based on a wind velocity = 160 km/hr for normal F_B

b) It is based on a wind velocity = 80 km/hr for minimum F_B

c) For dams with smooth upstream surface a freeboard of up to 1.5 times the above values

Freeboard Calculation

- Figure

Freeboard Calculation

- $$F_B = S + h_L + F_S$$

S = wind setup h_L = wave run up F_S = Safety margin

$$S = \left[\frac{FV^2}{63000D}\right]\cos\alpha$$

F= fetch (km)   V = wind velocity (km/hr)

D = ave. reservoir depth α = angle between fetch direction & wind direction

Freeboard Calculation

- Significant wave height:
  - Definition:

    Average height of the highest + 1/3 of waves

$$h_w = 0.00513V^{1.06}F_e^{0.47} \quad L_0 = 0.187V^{0.88}F_e^{0.56} \quad F_e = KL$$

L_0 =wave length (m)

F_e =effective fetch (km)

  - Also :

$$(F_e)_{max} = 0.031V^2$$

To limit the fetch effect on wave height

Freeboard Calculation

W/L	0	0.1	0.2	0.3	0.4	0.5	0.6	0.8	1	1.2	1.5	2
K	0	0.26	0.4	0.51	0.6	0.67	0.73	0.83	0.9	0.94	0.98	1

- Example:

  V =90 km/hr L =10 km D =100 W/L =0.5

  - ✓ $F_e = 0.67(10) = 6.7$ km
  - ✓ $h_w = 0.00513(90)^{1.06}\,(6.7)^{0.47} = 1.5$ m
  - ✓ $(F_e)_{\max} = 0.031\left[\frac{90 \times 10^3}{3660}\right]^2 = 19km > 6.7km$

Freeboard Calculation

- Reference Level
  a) Normal water level:
     - Max. probable wind condition (V_{max}=160 km/hr)
  b) Max. flood level (PMF):
     - Lesser wind condition (<80 km/hr)

- Design wave height:

Rockfill dams	$h_d = h_w$	For rockfill slopes
Earth dams	$h_d = 1.1h_w$	Cemented
	$h_d = 1.2h_w$	Moderately cemented
	$h_d = 1.3h_w$	Not cemented
	$h_d = 1.25h_w$	average

Freeboard Calculation

✓ Now knowing h_d , wave run-up h_L is now determined

➢ Safety Margin: $2' - 10'$

- Depending on:
  - Reservoir size
  - Dam height
  - Dependability of data
  - Risk of settlement due to earthquake

Freeboard Calculation

✓ Japanese Standard (allowance above flood level) :

Height of dam(m)	Concrete dam(m)	Embankment(m)
< 50	1	2
50-100	2	3
>100	2.5	3.5

- Above N.W.L > 5m - 6m normally
- The extra allowance for freeboard when reference is the flood level is less than when it is based on N.W.L
- Short term overtopping of the core maybe allowed in many cases but a min safety surplus to the dam crest must be respected

Geology

- Safety of dam
  - Foundation:
    - Stability
    - Faults
    - Joints
    - Seepage
  - Abutments
  - Reservoir watertightness (large dams)
  - Borrow pits

Geology

- ✓ Engineering geologist ←→ Design engineers
  - – Rock strength
  - – Rock mass strength & Stability
  - – Infillings
  - – Discontinuities (dip & dip direction)
  - – Seams, Faults, etc.
  - – Depth of weathering
  - – Distinguishing different regions of differential mechanical properties for using appropriate lab. parameters
  - – Landslides (Vaiont dam Italy ; 40×10^{6} m^{3} of reservoir water, overtopped the earth dam filled 1900 people d/s)
  - – Karst & Caverns (Lar dam) ; remedial action very costly & time consuming (abandon the site?)

KARSTS

- Carbonate Rocks
  - ✓ Having significant amount of soluble minerals
    - Calcite $CaCo_3$
    - Dolomite $CaMg(Co_3)_2$

    in their fabrics
  - ✓ There are other soluble material that maybe present in other types of rocks, they include
    - Gypsym
    - Anhydrites
  - ✓ Karsts occur in limestone beds, marble, metamorphose carbonate rocks; rich in calcite or dolomite

KARSTS

- Engineering properties of sedimentary carbonates rocks(Dearman 1981)

% Carbonates: 0, 10, 50, 90, 100

Predominant grain size (mm): 2, 0.06, 0.002

<table>
<tr><td colspan="4"></td><td colspan="3">limestone</td></tr>
<tr><td colspan="2">conglomerate</td><td colspan="2">Calcareous conglomerate</td><td colspan="2">Gravelly limestone</td><td>calcirudite</td></tr>
<tr><td colspan="2">sandstone</td><td colspan="2">Calcareous sandstone</td><td colspan="2">Sandy limestone</td><td>calcarenite</td></tr>
<tr><td rowspan="3">mudstone</td><td>Silt stone</td><td>Calcareous siltstone</td><td colspan="2" rowspan="2"></td><td>Silty limestone</td><td>Calcisiltite</td></tr>
<tr><td rowspan="2">Claystone</td><td>Calcareous claystone</td><td>Clayey limestone</td><td rowspan="2">Calcilutite</td></tr>
<tr><td colspan="4">marlstone</td></tr>
</table>

KARSTS

% Calcite

0	10	50	90	100
Dolomite	Calcitic Dolomite	Dolomitic Limestone	Limestone	
100	90	50	10	0

% Dolomite

✓ If the rock mass contains over 90% carbonates:

- When fresh they have very low K and porosity, therefore, flow is concentrated at joints, defects, … and the weathering and solutioning and cavities follow these patterns

- Large cavities will form & the 10% non-soluble material fill some cavities or form residual soil at surface

KARSTS

- On the other hand if it contains less than 10% carbonates, the rock is weathered, next to cavities and they have lower density because of removal of soluble material

✓ In general:

- The lower % of carbonates:
  - The less cavities
  - The higher proportion of weathered rock compared to cavities
  - The higher rate of infilled to open cavities

KARSTS

- Significance of solution effects

  - ✓ Need for treatment of dam foundation & reservoir
    - To fill cavities:
      1. Cement grouting
      2. Concrete curtain (diaphragm wall)
      3. Mining & Backfilling
      4. Backfill grouting
      5. Cut-off walls

- Presence of clay infilling in cavities presents a problem, they obstruct grouting and they may be removed later by underseepage

KARSTS

- Significance of solution effects

  - High pressure grouting designed to cause hydraulic fracture is shown to give significant improvements
    { Zhang & Huo (1982), Eadie (1986), McMalon(1986) }

  - Where cavities are numerous & largely or wholly filled with soils, cement grouting alone is not relied upon

  - Mining and backfilling with concrete,… must be adopted

  - ✓ In Khao Laem Dam in Thailand: These methods with (3.5km and up to 200m deep) grout-curtain was practiced

KARSTS

- Significance of solution effects

  - ✓ El Cajón Dam in Honduras:
    - The gorge from dam up to 200m into the valley sides and for at least 180m beneath the flow was caverneous limestone
    - The curtain adopted was in the form of the trough extending from dam to basaltic rock (Both sides and beneath the floor)
    - Construction involved:
      - 14km of galleries
      - 514,000 meters of holes drilled and grouted
      - 83,700 tonnes of cement
      - 14,930m^3 of backfilled concrete

KARSTS

- Significance of solution effects

  - ✓ Dams which failed to store water

Dam	Country	
Civitella Liciana	Italy	Cretaceous limestone
Cuber	Spain	-
Kopili	India	Eocene limestone
May	Turkey	Mesozoic limestone
Motejagne	Spain	Jurassic limestone
Perdikas	Greece	Miocene limestone
Villetle Berra	Italy	-
Lar*	Iran	Miocene limestone

* Leakage from deep inside the hight abutment emerging in two Springs d/s.
Discharge: – 0.5 m^3/s before construction of dam

– 5-10 m^3/s after construction of dam

KARSTS

- Other problems that may result

  – Possible collapse of cavities

  – Sinkholes

  – Dewatering of excavations

    (more continuous piping at higher rate is required)

KARSTS

- Dissolution Mechanism

  - Depends upon:
    - Solubility of the mineral
    - Rate of solution of the mineral

    (Speed at which it reaches equilibrium)

    - The solution rate K is, in turn, a fraction of:
      - Flow velocity
      - Temperature
      - Concentration of other dissolved salts

KARSTS

- Dissolution Mechanism (continued)

  - ✓ Governing equation:

$$\frac{dM}{dt} = KA(C_s - C)$$

dM= mass dissolved in time 'dt'

A = area exposed to solution

C_s = solubility of material (saturates concentration)

C = concentration of mineral in solution at time 't'

K = solution rate constant

KARSTS

Dissolution Mechanism (continued)

✓ Base on studies on carbonates, it was concluded:

- Joints aperture ≤ 0.5mm will not result in dangerous progressive solution

❖ If large cavities are backfilled, then; cement grouting which can fill joints down to 0.2mm aperture is adequate to prevent progressive solution of limestone

KARSTS

- Dissolution Mechanism (continued)

  - ✓ Important factors:
    - – Chemical composition of inflowing water (increase or decrease in solubility)
    - – Size & Distribution of open joints
    - – Flow velocity
    - – Gypsum surcharge? ; for countermeasures?

KARSTS

- Dissolution Mechanism (continued)

  - ✓ Carbonates are good for:
    - Concrete aggregates (If meeting the requirements ; specially Alkali reaction)

      Generally they perform very well
    - Good rockfills
    - Random fills
    - Riprap

    } if not shaley or argillaceous

  - ✓ Carbonates are not suitable for filter zones because of their susceptibility to dissolution and cementation

FILTERS

- ✓ Experience:
  - Particles size of base material, d_{85} , is a characteristic grain size

- ✓ Terzaghi & Peck (1948)
  - Both for fine & coarse filters (Initially for cohesionless soils as base material)

$$\frac{(D_{15})_f}{(d_{15})_{soil}} \geq 4 \; to \; 5 \qquad \text{(Permeability)}$$

  - Coarse particles of based are prevented from moving into filter, these coarse particles will then block movement of fine particles (self filtering in the base material)

$$\frac{(D_{15})_f}{(d_{85})_{soil}} \leq 4 \; to \; 5 \qquad K \propto D_{15}^2$$

FILTERS

- ✓ USBR (1973)

$$(D_{15})_{filter} = (5 - 40)\ (d_{15})_{soil}$$

$$\frac{(D_{15})_{filter}}{(d_{85})_{soil}} \leq 5$$

Fines content of filter $< 5\%$

FILTERS

- Moreover, filter grain size distribution should be roughly parallel to that of base soil

✓ Suggestions:

- Max. grain size ≤ 75 mm to minimize segregation
- They are very conservative if applied to clay base material. In that case these rules need not to be satisfied
- The reasons to limit fine content of filters ≤ 5%:
  - Must be non-cohesive (resist different without cracking)
  - fine material of filter may be washed out which affect the retaining potential

FILTERS

- ✓ Note :
  - If the base material ranges from gravel (over 10% > 8.75mm) to silt (over 10% P_{200}), the base material should be analyzed based on fraction smaller than 4.75mm (No.4 sieve)

➢ Critical Filter

- Where erosion might start (contact with base material ; d/s of core) is called *critical filter*
- ✓ Sherard (1984):
  - Investigated 36 types of silt & clay
  - 20% clay content (<0.002mm)
  - 7 soils were dispersive

FILTERS

- ✓ Sherard & Dunnigan (1989) investigated further, the results of their studies were:
  - Initial study
  - Figure
  - Further study ; four soil groups (no erosion filter test; NEF test)

	Soil group	Fines content	Max. D_{15} of filter (mm)
1	Fine silts & clays	85 - 100	$7d_{85}$ to $12d_{85}$ (mean $9d_{85}$)
2	Silty & clayey sands	40 - 85	0.7-1.5
3	Silty & clayey sands and gravelly sands	0 - 15	$7d_{85}$ to $8d_{85}$ (round grains) $9d_{85}$ to $10d_{85}$ (crushed grains)
4	Between 2 and 3	15 - 40	Intermediate between 2 and 3

FILTERS

- ✓ Honjo &Veneziano (1989)
  - For broadly-graded base material

$$\frac{D_{15}}{d_{85}} \leq 5.5 - 0.5\frac{d_{95}}{d_{15}} \qquad \text{for} \quad \frac{d_{95}}{d_{15}} \leq 7$$

  - A uniform filter reduces problem of segregation. Uniform sands C_u= 2 to 5 and appropriate D_{15} are always satisfactory filters
  - However :

    A broadly graded is:

    – Cheaper

    – Single filter, instead of multiple
  - Some guides limit C_u< 20 to prevent segregation. Segregation may be avoided if max. size < 75mm and if it contains 40% sand (pass sieve No.4)

FILTERS

- Internal stability of base material if broadly graded is required (to make for ?, filter criteria to be applicable at the interface with filter) if not, only small particles moves toward filter & passing through filter, leaving the coarse part behind: also self-filteration.

✓ To check it out:

- Divide the grain size into two parts at any size exceeding 0.2mm.Then the filter criteria must apply between the coarse and fine fractions
- If not possible to meet the above criteria, a two zone filter may be required. The zone next to core a fine to medium sand designed for soil matrix of the core.

✓ Perfect filter concept:

- A filter to retain smallest particle even if they arrive at filter, after complete segregation of coarse material

FILTERS

- RipRap

Max. Wave Height (m)	D_{50} (cm)	Max. Rock (kg)	Layer Thickness(cm)
0 - 0.3	20	45	31
0.3 - 0.6	25	91	38
0.6 - 1.2	31	227	46
1.2 - 1.8	38	680	61
1.8 - 2.4	46	1134	76
2.4 - 3	61	1814	91

- Well-graded: $2.5^{cm} \leq D \leq 1.5D_{50}$
- Thickness $\geq 1.5D_{50}$
- If the quality of riprap is not very good, thicker riprap must be considered

FILTERS

- Significant wave height

$$\frac{\delta H_S}{V^2} = 0.0026\left(\frac{\delta F}{V^2}\right)^{0.47}$$

- Filter under riprap

$$(D_{15})_F < 5(D_{85})_{Embankment}$$

$$(D_{15})_{RipRap} < 10(D_{85})_F$$

FILTERS

- ✓ Thickness 9″ - 30″

- ➢ US. Corps of Engineering (Min. Thickness)

Max. wave height (m)	Filter thickness (cm)
0 - 1.2	15
1.2 - 2.4	22.5
2.4 - 3	30

- ✓ Downstream face protection:
  - RipRap with no under filter layers of coarse & sand & gravel with max. size of 3in. or more is satisfactory in most cases

Geology

- ✓ Availability of natural materials for construction will affect:
  - Its cost
  - Type of dam

- ✓ Geology
  - – Safety
  - – Availability
  - – Cost & type of dam

Geology

- Rock classification

1. Uniaxial compression strength
   - Weak <35 MPa
   - Strong 35-115 Mpa
   - Very strong >115 Mpa

2. Prefailure deformation Elastic, Viscous
3. Failure characteristic Brittle, Plastic
4. Gross homogeneity Massive, layered
5. Continuity in formation Solid, Blocky, Broken

Geology

- Rock classification (continued)

6. Weathering:

   – Fresh — no visible sign
   – Slightly — in open discontinuity surface
   – Moderately — extends throughout the rock mass friable (and easily crumbled)
   – Highly — extents throughout & partly friable
   – Completely — wholly decomposed
   – soil

Geology

- Foliation

  - Rocks subjected to heat and deforming pressure during metamorphism process, parallel layers will develop (along which are new minerals such as mica, talc,…) and tend to expand due to coming out

Geology

- Fualts

  - ✓ Recognized by:
    - Offset of beds
    - Gouge
    - Brecciation or crushing
    - Topographic features
      - Escarpment
      - Offset alignment of vegetation

Geology

- Geology
  - Global
  - Regional
  - Local (⟷ surface)
    - Geophysical
      - Seismic
      - Electrical
    - Drilling
    - Adits

- Global
  - Crust movement, seismic history,…
- Regional
  - Age, location of faults, landslides, karsts, weaknesses, aerial photos, satellite,…
  - General geological formation of the region
  - Exposed rocks, ridges, possible reservoir leakage

Geology

- The regional report covers the whole reservoir area. It must consider reservoir tightness & slope stability before and after ? . may require special investigations such as tracer,… in this respect regional and local geology may overlap

- At most sites geological interpretation is by inference and its accuracy will be proportional to the amount of work done

Geology

- Local Geology

  - Continuation and detailing of regional report
  - Damsite& whole work area, borrow areas, access roads,… (more experienced engineers are required)
  - For the dam foundation:
    - Knowledge is required to a depth ≅ height of the structure. If some cases more depth may be required; but, not all boreholes need be as much deep
    - Reasonable understanding to this depth is necessary

Geology

- Local Geology (continued)

  - Must include

    - Description of rock types & soils
    - Geotechnical surface mapping
    - Mapping joints & faults
    - Graphical presentation of strikes & dips of discontinuities
    - Evaluation of risk of landslides
    - Borehole & test pit profiles
    - A mapping that shows location of BH, TP,…

Geology

- Surface Geology
  - Discontinuities:
    - Wedges?
    - Gouges, fillings
  - Weathering
  - Overburden (alluvium)

    Not require to be removed if it is well consolidated

Geology

- Surface Geology
  - It may be required to sluice the site to have features exposed
  - Gneiss, Mica Schist are good for strength and watertightness; but, excess mica in the foliation drop the friction angle from 40° to 30°
  - Such weaknesses are sometimes ⟶ intense folding
- ✓ Trenching
  - – Faces, mapped, sampled & photographed
- ✓ Possibility of leakage
  - – Along smooth contacts
    - Rock
    - Conduits in the embankment
  - – Hydraulic gradient

Geology

- Geophysical
  - To supplement surface and subsurface investigations
  - Rapid & Cheap

✓ Must be well planned to get the most amount of information

- – Seismic
- – Electrical
- – Base of weathered layer for stripping
- – Base of cutoff
- – Correlate seismic readings with BH profiles
- – Geophone spacing depends on No. of layers, homogeneity, 20^m - 50^m?

Geology

- Drilling
  - Up to 1.5^{m} diameter, is possible diamond drilling
  - Core recovery
    - Nature of strata
    - Equipment
    - Method of drilling
    - Experience & Skill
  - Usually 100^{mm} core, less often 50^{mm} double tube core barrel drilling + split core tube
  - Life of diameter bits: 2^{m} (Quartzite) ⟶ 44^{m} (Mudstone)
  - Monthly drilling rate in the order of 300^{m} to 500^{m}
  - In developing countries the same equipment ⟶ 30^{m} - 50^{m}

Geology

- Drilling (continued)

  - Taken color photos of cores in the core boxes
  - Extension of borrow areas
  - Location of BH
    - Axes
    - Spillway
    - Abutment (grout curtain)
    - Reservoir rims
    - shell (if needed)
    - Borrow areas
    - …

Geology

- Test Pits
  - Bulldozer trenches

✓ Gives

a)

– Geotechnical soil description and sampling

– Mapping of the wall

– Suitability of material in dam body

  - Core
  - Filter
  - …

– Disturbed and undisturbed samples for lab tests

Geology

- Test Pits

b) Field tests

- Cohesive soils
  - ω_n (moisture meters)
  - Shear strength
- Coarse grained soils
  - Gradation test

c) Lab tests

Geology

- Adits

Positive information obtained by going underground

- Min dimension: $1.5^{m} \times 2.5^{m}$

  $2.5^{m} \times 3.1^{m}$
- Mapping the geological features
- All geological features are mapped
- Can be used for insitu testing plate load test (Jacking) vertical and horizontal ⟶ modulus of deformation
- May be used for foundation treatment later

Geology

- Permeability
  - Piping
  - Erosion & Collapse
  - Stability (foundation & abutment)
  - Soluble rocks (Gypsum, Anhydrite)

Geology

- Recording & Presentations
  - Logging
    - Standard geological terms understandable to the engineers
  - Drilled cores
    - Must be retained
    - Must be photographed (color)
  - Geological map

Geology

- Seismic Activity
  - Earthquake (history), Regional
  - R.I.E

- Natural Events
  - History of region
  - Recording of all faults
  - Installation of strong motion seismograph
    - On rock at dam base
    - Crest
    - On rock at the short distance (papers be added here)

Geology

- Reservoir filling

  - ✓ Hoover Dam, reservoir capacity 42×10^9 Tons
    - No earth tremor recorded prior to construction
    - Filling began "1935"
    - 1^{st} shock "1936" ; water level 100^m
    - "1937" over 100 tremor
    - Largest shock magnitude "5"
    - Assumed earthquakes induced by load on crest probably on faults or regions of weakness

Geology

- Reservoir filling (continued)

  - ✓ Kariba Dam, Rhodesia; began to fill in "1958“ ; total reservoir capacity 170 × 10^9 Tons
    - No information available of past activity
    - Shocks observed 6 months filling began
    - Greatest magnitude "5.8" ; 4 years later

  - ✓ Koyna Dam, India; 103^m high dam; reservoir capacity 2.8 × 10^9 Tons
    - Strongest record "6.4" magnitude
    - "0.5g" acceleration

Geology

- Reservoir filling (continued)

  - ✓ Eucumbene Dam, Australia; reservoir capacity 4.8×10^9 Tons
    - In region of known seismicity
    - During 1st filling two shocks "4" & "5" magnitude
    - Many shocks up to "4" magnitude

  - ✓ Talbingo Dam, Australia; reservoir capacity 0.9×10^9 Tons
    - No record in past 13 years before filling; 1st filling; May 1973
    - June ⟶ recorded seismic activity

      As water level increase ⟶ Increase in activity

Geology

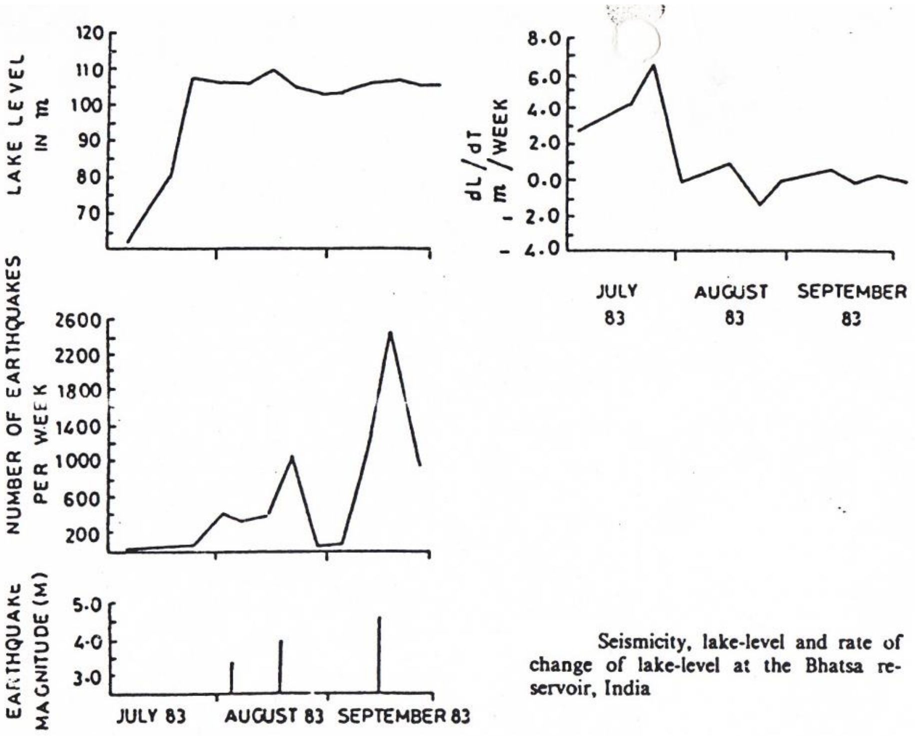
- Reservoir filling (continued)

  - August ⟶ Rate of fill dropped sharply ⟶ Decrease in activity

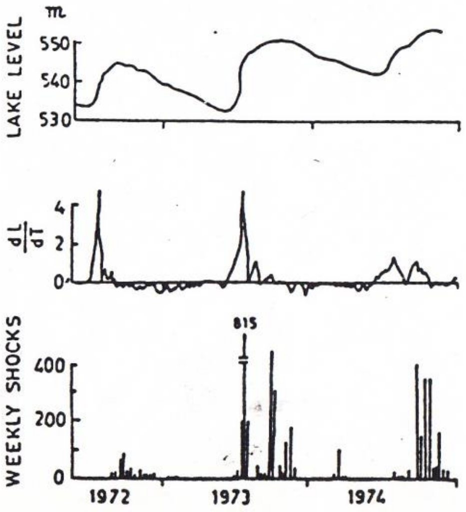
    All magnitude < "2.4"
  - Up to 1972 ⟶ 2000 weak events; strongest "3.5"
  - Most events within 7^{km} radius U/S on west bank

✓ A US seismologist studied 3 dams:

  - At Hoover frequency of seismic events related to level of water in the lake, while at two other dams filling apparently lessened seismic activity



Seismicity, lake-level and rate of change of lake-level at the Bhatsa reservoir, India



(after Powar et al.,1986)

Seismicity, lake-level and rate of change of lake-level at the Mula reservoir, India

Geology

- Reservoir filling (continued)

  - ✓ Conclusion

    - ❖ It is a possibility monitor before & after filling.

  - ✓ Prediction of Magnitude

  - Baoqi (1992):

$$M=1.317+0.995E\pm1.201$$

Where: E: "Comprehensive effective parameter" $=SH_{max}/V$

S: Reservoir surface

V: Reservoir Volume

H_{max}: Reservoir Maximum depth

PII: S 0 2 6 7 - 7 2 6 1 (9 7) 0 0 0 2 5 – 0

Soil Dynamics and Earthquake Engineering **17** (1998) 53–56

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ELSEVIER

Reservoir induced earthquakes analyzed via radial basis function networks

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(Received 5 June 1997; accepted 17 June 1997)

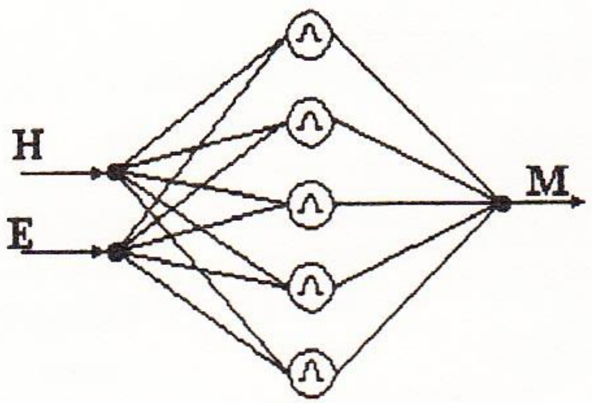


Fig. 2. Architecture of the RBF network used in this study.

$$\phi(r) = e^{-r^2/\beta^2}$$

$$O_k = \sum_{j=1}^{m} \lambda_{jk}\phi(\|x_i - c_{ij}\|)$$

β: Spread of function
λ: Connection weights
Cij: Centers of RBFs

$$\|x_i - c_{ij}\| = \sum_{i=1}^{n} (x_i - c_{ij})^2$$

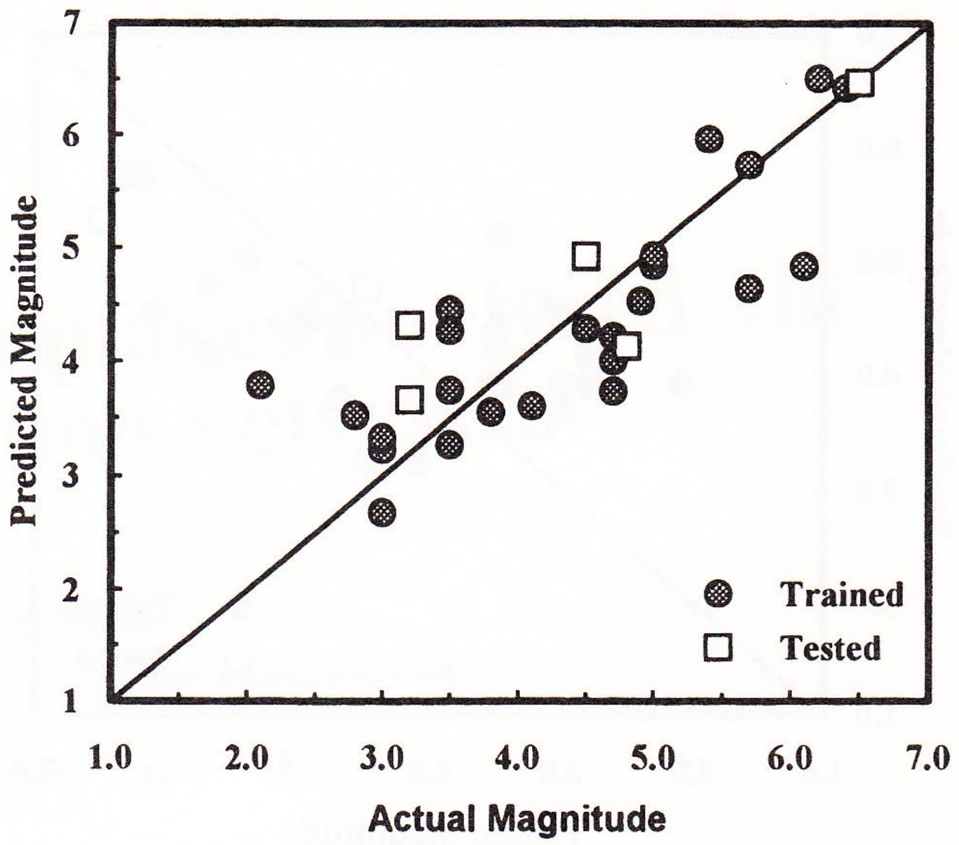


Fig. 3. Predicted magnitude of RIE versus actual magnitude (RBF network).

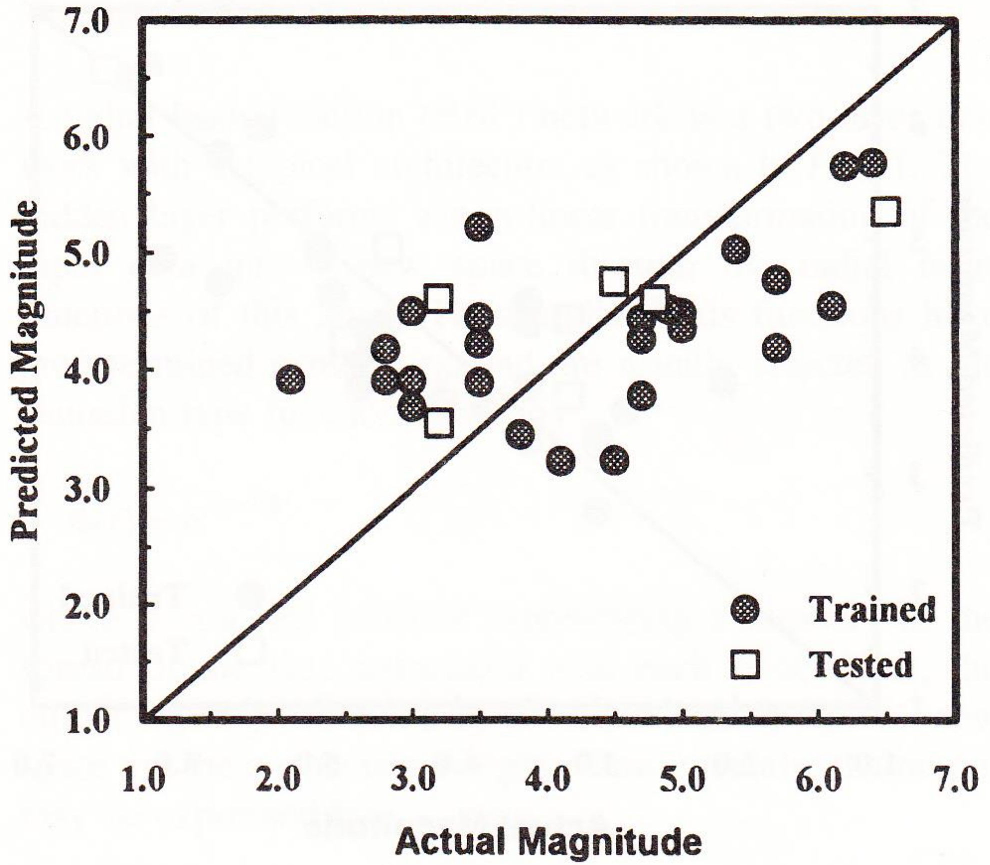


Fig. 4. Predicted magnitude of RIE versus actual magnitude (statistical approach).

Geology

- Stability of valley wall

  - Influence of P.W.P on stability
  - Different modes of failure
  - DRM

Geology

- Material Investigation (continued)

  - Availability of suitable material (as soon as possible)
  - For embankment ⟶ Impervious, semi pervious, free draining
  - For concrete aggregate ⟶ Sound, inert rock or gravel or sand

Geology

- Material Investigation (continued)

a) Reconnaissance

✓ Evaluation:

- Type, approximate quantities within reasonable distance aerial photos, regional geology, local information, aerial inspection
- No subsurface work, samples taken for preliminary property testing as well as photographic and mineralogical examination
- A suitable plan indicating sources should be prepared

Geology

- Material Investigation (continued)

b) Feasibility

- Selected area are explored with sampling on a grid, say, 100 m^2 ; γ_d , ω_{opt} , c , φ
- Alluvium, sand and gravel studied (depth and likely quantities of usable materials)

Geology

- Material Investigation (continued)

  b) Detailed investigation

  - Trenches (more accurate picture)
  - Auger (fine grained material)
  - In coarse material dozers are often used
  - Back hoe below water table
  - Seismic methods to supplement

Geology

- Material Investigation (continued)

  - Material 50% more than what is needed further allowance for material grading and reblending such as filters
  - Mech. properties ⟷ Geotechnical Engineers
  - Mineral & ? properties ⟷ Geologist (explain anomalous behavior)
    - Warn against possible alkali reaction in concrete
    - Warn against dispersive characteristics
    - Warn against soundness

Foundations

- Acceptable deformation
  - Elastic
  - Consolidation after reservoir filling
  - Change of strength due to saturation
- Stability
- Any change in modulus of deformation of foundation along dam axis
- Adequate strength
  - Weathering
  - Clay seams

Foundations

- Large excavation may result in:
  - Upward heave
  - Crack in the abutment wall (due to distress ⟶ increase in water seeping through abutment)
- 10^m – 20^m of rock immediately under the dam is of greatest importance

Foundations

- Properties to be tested
  - Crushing strength
  - Shearing strength
  - Elasticity of rock mass
  - Tectonic stresses
  - Permeability
  - Crushing strength

    Depends on:

    – Quality

    – Degree of weathering

    – Micro cracks

Foundations

- Intact rock

Rock type	Strength (MPa)
Silt stone	24 - 120
Shale	35 - 110
Sandstone	40 - 200
Limestone	50 - 240
Dolomite	50 - 150
Granite	90 - 230
Basalt	200 - 350
Gneiss	80 - 330

Foundations

- Rock mass

  Depends on number of joints, infillings, roughness

  – Shear strength

  • Basic friction angle

Rock type	Friction angle (°)
Basalt	31 - 38
Conglomerate	35
Dolomite	27 – 31
Gneiss	23 – 29
Limestone	33 – 40
Sandstone	25 – 35
Shale	27
Silt stone	27 – 31
Slate	25 - 30

Foundations

- Patton

$$\tau = \sigma \tan(\varphi + i)$$

$$i = f(\sigma)$$

- Barton

$$\tau = \sigma \tan(\varphi + JRC \log \frac{\sigma_j}{\sigma})$$

  - σ_j: *uniaxial strength*
  - *JRC: joint roughness coefficient*
  - $5 < JRC < 20$

Moduli

- – Tangent modulus
- – Secant modulus
  - ❖ Secant modulus often used

✓ Modulus of deformation

- Since it is not isotropic or homogeneous
- In situ tests measures modulus of deformation

Moduli

- ✓ Order of magnitude (Intact Rock)

Rock type		($\times 10^3$ MPa)
Limestone		3 – 27
Dolomite		7 – 15
Very hard limestone		70
Sandstone		10 – 20
Siltstone		3 – 14
Gneiss	Fine	9 – 13
	Coarse	13 – 24
Schist	Micaceous	21
	Biotite	40
	Granite	10
	Quartze	14
Granite	Very altered	2
	Slightly altered	10 – 20
	Good altered	20 – 50
Basalt		50
Andesite		20 - 50

Moduli

- ✓ Effect of loading direction

  E_P : Parallel to stratification

  E_N : Perpendicular to stratification

Rock Type	E_p/E_N
Sandstone	2.3
Granite	1.3
Schist	1.9
Sandstone	1 – 1.6
Sandstone	1 – 1.7

Moduli

- ✓ Mudstone

Stress range (MPa)	E ($\times 10^3$ MPa)	
	Perpendicular to bedding	Parallel to bedding
0 – 1.4	24	24
0 – 2.8	25	42
0 – 5.6	31	46
0 – 8.4	28	46
Dynamic	41	34
In situ	17	27

Moduli

- Modulus of deformation
  - The in situ modulus of deformation is needed due to presence of joints & fillings it may be as low as half the lab values or even 1/10 as was the case with Nagawado Dam in Japan (155^m high Arch Dam)

- ✓ Geological hammer

A ring like steel	70×10^3 (MPa)
Solid ring	7×10^3 (MPa)
A low pitched note	700 (MPa)
A dull clunk	$\cong$ 70 (MPa)

Moduli

- ✓ Foundation deformation ⟶ Additional settlement of the embankment
  - Reasonable "E" value for analysis is required

- ✓ Consolidation Grouting

  E_M : In situ modulus of deformation

  E_{ML} : Lab. Modulus in the first loading cycle

  E_D : dynamic modulus (Lab.)

– Modulus at Mossy rock Dam

	(MPa)	(MPa)	(MPa)
In situ jacking	16500	9000	5500
Lab. Core test	30300	20700	24000
Geophysical	25000	30300	33000

Moduli

- Poisson's Ratio

$$\vartheta = 0.25 - 0.5$$

- Can be determine seismically from shear wave velocity
- For both rock specimen in Lab. or rock mass in the field

- In situ tests

✓ Shear test

In galleries

Generally continued to large strains to measure residual parameters; τ_r , φ_r

Moduli

- In situ tests (continued)
  - ✓ Residual Rock Stresses

    - A rosette strain gauge fixed on galleries' wall and "overcored" the final reading of gauges ⟶ deformation due to the de-stressing ⟶ original stress pattern in the rock
    - Flat jack may be used to determine the stress in one direction
    - A slot is pressurized till the pins on the two slides of the slot are at their original position

    (A narrow slot is made by using a saw)

Moduli

- In situ tests (continued)
  - ✓ Deformation Modulus

Jacking

- In galleries wall to wall or floor to floor
- In boreholes using anchore cable
- Assessing effectiveness of treatment by F.E studies

Permeability

– Piping
– Stability (material moved –soft)
– Water loss

- Construction ⟶ close the joints ⟶ K ⇩
- Filling the reservoir ⟶ tend to open cracks ⟶ K ⇧

✓ Possible modes of leakage

– Piezometers Hydraulic gradient (direction acceptable?)
– Radioactive Isotropes (tracing)
– Dyes Fluorescein

Permeability

- Leugeon Method (1933)
  - Using packers, permeability is measured for a length "l" of the drill hole:

$1^m \leq l \leq 5^m$ less fractured rock →

highly fractured rock ←

a) Saturated the section with low pressure until seepage is stabilized at constant rate

b) Flow measured at few consecutive intervals of 5 min. till the deference between two consecutive measurement is less than 10%

c) Procedure is repeated with increasing pressure up to 10 Bars. and then in descending order (Exp. 4,7,10,7,4)

Permeability

- Leugeon Method (continued)

  d) Pressure at the middle of "l" :

$$P = \left(P_m + \frac{\Delta H.\gamma_w}{10}\right) - \Delta P$$

ΔH : Different in elevation between pressure reading monometer and grand water table / or middle of "l" if no water table exists

P_m : Measured pressure (Bars)

ΔP : Hydraulic pressure loss along the pipe and fitting from monometer to packers which is a function of "Q"

Permeability

- Leugeon Method (continued)
  - d) LU (Lugeon Unit) is defined as:

$$N = \left(\frac{10\,Q}{Plt}\right)$$

Q: flow of water (lit)

l : section length (m)

t : time during which Q is measured (min)

P : testing pressure (bars)

✓ When $l = 5^m$

$K = 1.5 \times 10^{-5}\ cm/s \qquad r = 4.6^{cm}$

$K = 1.3 \times 10^{-5}\ cm/s \qquad r = 7.6^{cm}$

Permeability

- Permeability tests
  - Pump in or Pump out (sand & gravel)
  - Leugeon Test

    1 Leugeon is the acceptable value unless the river flow is high enough and generally depends on the value of water in that project. The grout mix should be designed for the appropriate soil under study

Grouting

- Single
- Staggered
- 3-D arrangement
- Depth

$$D = \frac{H}{3} + C$$

$8^m \leq C \leq 25^m$ depending on dam size, foundation type, significance of seepage

If : $C = 25^m$, $H = 60^m$

$$D = \frac{60}{3} + 25 = 45^m$$

Another suggestion ; $\frac{H}{2} \times 1.2 \cong 0.6H \quad 0.6 \times 60 = 36^m$

Suggest $\longrightarrow \cong H$ For $H = 60^m$ (Seepage Analysis)

❖ Note: Not good for karstified foundation

Grouting

- Dokan Arch Dam in Iraq

  Extends into abutments, total length 24^{km} , area 450,000 m^2, as deep as 200^m holes (Exceptional case)

- CFRD
  - Require special attention due to the very steep gradient
  - 107^m Cethana Dam on quartzite & conglomerate 10^m -12^m spacing along the plinth (Depth?)
  - Resistivity may be used to check effectiveness

Grouting

- Pressure
  - $1\ {}^{psi}/_{ft} \cong 0.25\ {}^{kg/cm^2}/_{m}$
  - Pressures $> 20\ {}^{kg}/_{cm^2}$ should be applied under engineering supervision

- Mixture
  - Start at their mixture say $5:1$; thicken if it is consumed freely:
    $4:1\ , 3:1\ , 2:1\ , 1:1\ , 0.8:1\ , 0.6:1$

- Chemical grouting
  - Water/Cement mix, fills cracks up to 0.6^{mm} wide

Grouting

- Karstic Foundation

  Filling should be at stages with remedial works if require

  - It may require extensive grouting
  - Effectiveness

    Grouting against head?

  - May-Dam in Turkey

    Never filled; 10^m – 15^m alluvium over limestone & marl

    ⟶ Over 36 sinkholes near the dam

  - Tarbela in Pakistan
  - Lar in Iran
- Soluble Material
  - Gypsum, Anhydrite

Foundation Improvement

In order to:

- Decrease deformation
- Decrease permeability
- Increase strength
- Protect against erosion
- Increase stability of abutments

Consolidation Grouting

- To consolidate, increase resistance to erosion of infilling material in the zone of max. hydraulic gradient
- Low pressure to prevent heave
- Oriented to intersect as many seams as possible
- Increase "E"

Embankment

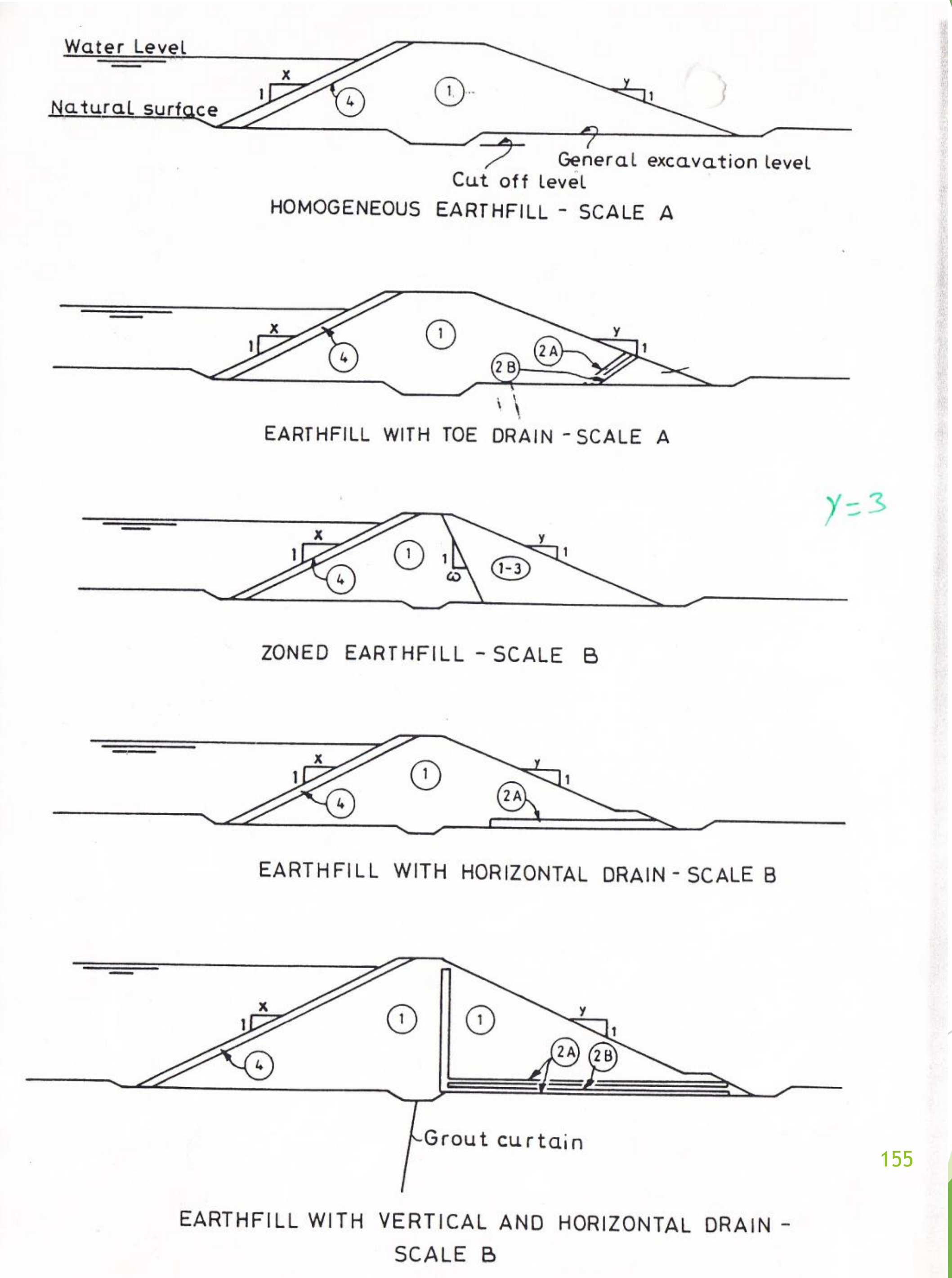
- Nurek Dam; 312^m high in Tajikestan
- Masjed Soleyman: 177^m heigh, Crest 488^m , Reservoir: 228×10^6 m^3
- Karkheh Dam: 127^m high, Crest 3030^m , Reservoir: 7.3×10^9 m^3
- Marun: 165^m high, Crest 345^m , Reservoir: 1.2×10^9 m^3
- Gotvand: 180^m high, Crest:760^m , Reservoir: 5×10^9 m^3

- CFRD high dams have been built
  - Anchicaya; up to 152^m
  - Area; 160^m Brazil
  - La Miel; 180^m Colombia
  - Siah Bisheh: 82.5 m

Embankment

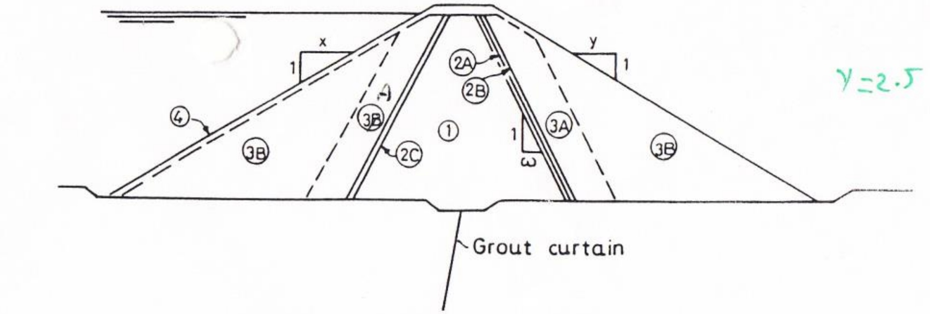
- Reinforced Earth
  - To provide support for the spillway reduce the cost

Embankment

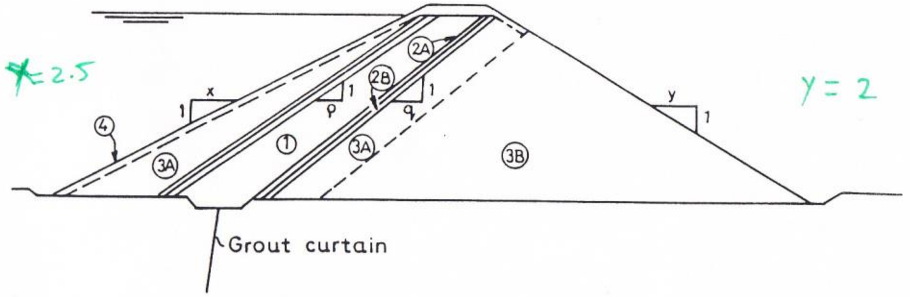


Water Level
Natural surface
x
1
y
1
4
1
General excavation level
Cut off level
HOMOGENEOUS EARTHFILL - SCALE A
2A
2B
EARTHFILL WITH TOE DRAIN - SCALE A
y=3
1-3
ZONED EARTHFILL - SCALE B
EARTHFILL WITH HORIZONTAL DRAIN - SCALE B
Grout curtain
EARTHFILL WITH VERTICAL AND HORIZONTAL DRAIN - SCALE B

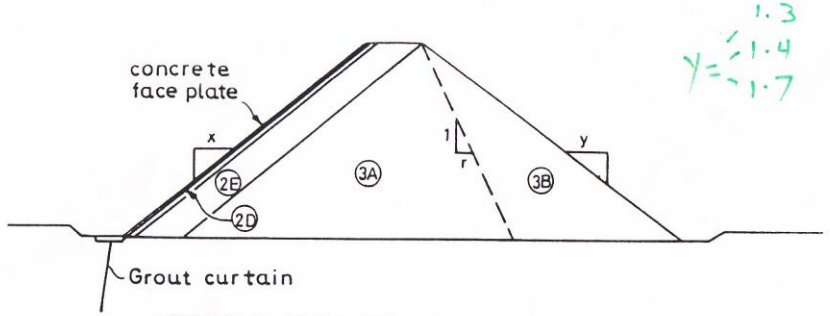
Embankment



EARTH AND ROCKFILL - CENTRAL CORE - SCALE B



EARTH AND ROCKFILL - SLOPING UPSTREAM CORE - SCALE B



CONCRETE FACE ROCKFILL - SCALE B

NOTES

1. Crest detailing and downstream slope protection not shown.
2. Scales relate to overall size, details are not drawn to scale.

Scale A 0 5 10 m

Scale B 0 10 20m

Embankment

a) No internal erosion control must be limited to low heights ($\leq 5^m$) in low hazard locations

b) If $\frac{k_h}{k_v} \ll 1 \longrightarrow$ seepage on D/S face! (uncontrolled seepage may occur)

($\leq 10^m$) in low hazard locations

c) Zone (1): Impermeable (low permeability) zone

Zone (1-3):

Same material with less compaction

Weathered or low strength rock with sufficient amount of fines to provide internal stability against erosion. Not necessarily meeting the filter criterion

($\leq 20^m$) in medium to low hazard sites

Embankment

- d) Good if $\frac{k_h}{k_v} \cong 1$; if $\frac{k_h}{k_v} \gg 1 \longrightarrow$ seepage may bypass the filter & horizontal drain ; ($\leq 10^m$) in medium to low hazard sites
  - Cases a, b, c, & d have been constructed for large dams in the past

e) Seepage control is independent of $\frac{k_h}{k_v}$

Suitable for construction of large dams; mostly 30^m – 50^m high

g) Less earthfill Thin Central core

Placement of D/S rockfill in wet season followed by core material & filter

Steeper D/S slope but flatter U/S slope

h) CFRD

Embankment

- Factors Affecting Selection of Dam Type
  - Amount of seepage permitted through dam and through foundation; value of water, stability
  - Settlement of dam and foundation (integrity and hence watertightness)
  - Freeboard allowance
  - Effect of ambient weather

Embankment

- Dam Design
  - ✓ Zoning

    To insure safety in terms of

    - Strength
    - Seepage control
    - Cracking control
  - ❖ The final selection is the most economical utilization of the materials available

Embankment

- Stability
  - ✓ Circular Failure
    - – Simplified Bishop
    - – Spencer
    - – Morgenstern & Price
    - – Janbu
  - ✓ Wedge Failure
    - – Strength parameters { Void Ratio? / Stress Ratio? }
    - – Reservoir Elevation?

Embankment

- Type of dam selected based on
  - Foundation
  - Available material

- Slope angle depends on
  - Characteristics of materials
  - Core thickness

Thin core ($t < \frac{H}{2}$) and well compacted rockfill

Slope $\longrightarrow$ angle of repose ; $1{:}\frac{4}{3}$ (CFRD)

For AFRD for effective ? Of asphaltic facing ; 1: 1.7 U/S

❖ In seismically active regions slope not steeper than 1:2

Embankment

- Possible forms of sealing under considerations
  - A central core of hot asphaltic concrete
  - A core of cold placed bituminous emulsion
  - A central core of lean concrete, by grout intrusion
  - A central core of foamed concrete
  - A plastic, rubber sheet, corrugated metal membrane in a thin clay core
  - Concrete face
  - Asphaltic face
- Req. :
  a) Flexibility
  b) Watertightness
  c) Performance
  d) Construction convenience

Embankment

- Decrease in slope angle of few degrees may increase the cost by as much as hundreds of million Rials (1370!)
- Plane strain analysis may not provide suitable representation, for steeped wall valleys because of ?
- Dynamic analysis will be discussed later. Factors increasing resistance to earthquake:
  - Compaction for max. density
  - Ample freeboard

    8^{m} for 160^{m} high Dartmouth in Australia
  - Provision that a wave could pass safety over (Mica Dam Canada)
  - Generous transition zones in zoned dam, flared toward abutment

Embankment Dams

- Non-cohesive filter to encourage self-healing of cracks
- Protection of D/S face, particularly at its toe, against overtopping

Embankment

- Crest width
  - Japanese Code

$$W = 3.6\sqrt[3]{H} - 3$$

H (m)	W (m)
30	8
50	10
70	11
100	13
200	18

W Governed by:

- Construction procedure
- Access requirement

- More suitable:

$$W = 3.6\sqrt[3]{H} - 4$$

Except for seismically active zones

Embankment

- Slope Protection

  D/S : Erosion by rain water

  U/S : Wave, Ice, Impacting of floating debris

- Riprap Size

Max. wave height (m)	D_{50} (m)	Max. rock (kg)	Layer thickness (m)
0 – 0.3	0.2	45	0.3
0.3 – 0.6	0.25	90	0.38
0.6 – 1.2	0.31	227	0.46
1.2 – 1.8	0.38	680	0.61
1.8 – 2.4	0.46	1134	0.76
2.4 – 3	0.61	1814	0.91

Embankment

- Alternative:
  - Mass concrete
  - Reinforced tetrahedral
  - Soil cement compacted in layers

Dangerous with drawdown due to P.W.P behind which may destroy it and endanger the dam.

➡ A drainage layer

Homogeneous Embankments

- Req. for the material in a homogeneous dam or core of a rockfill dam:
  - Sufficiently impervious
    - to restrict water loss
    - Safety
  - Capable of being placed and com pacted free from potential paths of percolation through the fill or along the contact
  - Should develop max. practical shear strength and maintain it after reservoir filling
  - It must not settle, soften or liquefy upon saturation (Earthquake?)

Homogeneous Embankments

- Water content:

$< \omega_{opt}$:

   – Higher rigidity, good adherence to foundation
   – Susceptible to cracking

A compromise in water content to suit the particular condition

- Standard Proctor Criteria

- A chimney Drain $\longrightarrow$ Stability of D/S ⇧
- Importance of Rapid Drawdown

Homogeneous Embankments

- Types of controlling underseepage:
- -Open trench
- Slurry trench/Cut-off wall
- Grout Curtain
- U/S Blanket

Earth-Rockfill Dams

"Most used"

- ✓ Core
  - To form impermeable barrier the rest of dam to insure stability
  - From natural material: clay, gravel, etc
  - From prepared material: cement, asphaltic
  - Metal, Plastic, Rubber, etc

Earth-Rockfill Dams

- ✓ Core width
  - Depend on :
    - Material available
    - Type of foundation
    - Permissible at contact
  - Thinner core
    - Steeper upstream face
    - Less material in the dam and core

$$W \cong \frac{H}{2} \quad \Rightarrow \quad i \cong 2 \qquad , K \leq 10^{-5}\, cm/s$$

Earth-Rockfill Dams

- Dispersiveness
  - Should be checked blockage of filter (may be treated with line?)
    - Inclined core are thinner

– Vertical cores $1 \leq i \leq 4{,}5$

– Inclined cores $1.2 \leq i \leq 5{,}9$

Thin cores should be provided with generous well designed filters on each side

– The foundations of closely jointed rock require thicker filter width

– Low plasticity clay; PI? Is more suitable? Cracking?

Water content:

Higher the opt.?

Lower the opt.?

Embankment

- Dispersiveness (Continued)
  - Normally high plastic clay is used as a cover on foundation & abutment
  - Max. core size material:

$<50^{mm}$	Lower cabin creek
$<120^{mm}$	Mattmark
$<115^{mm}$	Talbingo
>5%	Finer than 4.7^{mm}
>15%	Finer than 0.074^{mm}

Embankment

- Other types of cores
  - Early dams were built using a external core of cement concrete. It fractured in many instances and repair was costly
  - Most suitable is a “flexible“ core
  - Concrete core: Grouting the rockfill of no fines.
  - Grouting at different elevation during construction
  - A *P.E.* sheet 0.6^{mm} thick as a central impervious membrane for a 75^{m} high dam on the Atboohy river in Russia

Embankment

- Other types of cores (Continued)
  - Symmetrical location at center since 1940 different zones:

    U/S ⟶ D/S

    Riprap, Transition, Rockfill shell of good quality, Filter zone, Core, Transition (filter), Good rock, Poorer rock, D/S face
  - Moving core U/S ⟶ more economy greater quantity of rockfill could be placed in one operation, Nantahala (1942) (inclined core) very thin $i = 9\,;H = 80^m\,, W = 8.8^m$
  - Near vertical core ensures max. contact pressure on the foundation
  - Inclined cores as far U/Sas possible if stability is not jeopardized. D/S rockfill is placed first, so a large proportion of settlement has occurred before the transition zones and core are superimposed

Embankment

- Seismic Resistance

  - ✓ Sherard:

    Inclined core has better resistance

  - ✓ Thomas:

    A near vertical core provides the greatest stability under earthquake. Def. would be less and less serious

- The core may be widened toward the abutments to mitigate the tensile strains

- Remis modification of inclined core

Embankment

- Cracking of core

"Post Construction" Crest Settlement (mm)	Kind of cracking
< 50	No cracking
$50 \leq \ < 100$	Transverse cracking of dams compacted dry of opt. may appear
$100 < \ < 130$	Longitudinal cracking between core & shell may appear R.C. slab without perimetral joints may crack
$> 160 - 180$	Longitudinal cracking of core compacted dry of opt. hydraulic fracture occur
> 220	Transverse cracking of core compacted wet may appear, longitudinal crack between shell and core compacted wet of opt.
$350 - 400$	Asphaltic concrete facing may crack longitudinal cracking of core wet of opt. R.C. facing with perimetral joints may crack
$1000 - 1200$	No uncracked dam, all dams exhibit transverse cracking
≥ 1400	Serious cracking of asphaltic concrete facing
≥ 3800	Cracking needing substitution of R.C. facing

- From J.Justo ; Based on a study of 180 dams

Embankment

- Cracking of core (Continued)
  - Cracks perpendicular to the axis appears at crest due to non uniform settlement

    Crack may penetrate deep & it should not be neglected

  - Cracks parallel to axis, due to different settlement between core & the rockfill shell

    Generally they are not dangerous, so long as they are discovered and backfilled with a fine grained non-cohesive material. It may be recommended not “covering“ the crest of the dam until most of settlement has occurred

Embankment

- Cracking of core (Continued)

  – Horizontal crack in the core may develop due to saturation (Saturation Collapse) when it is compacted dry of opt. It does not appear at the surface and hence it is serious. It may happen between core & shell due to unequal settlement. It may appear in a narrow gorge (arching)

1. Remedy : Backfill after trenching
2. If transition zone of non-cohesive material and adequate thickness is provided it may be self-healed.

Filter and Transition Zone

- It is good practice to widen the transition zone towards each abutment where tension and shear cracking may develop

- The thickness may be designed based on the filter requirement, but usually the thickness is controlled by placing equipment.
- Generally it is much cheaper to use ordinary equipment and it is good practice to be liberal with their thickness

Rockfill

- Strong sound rock is recommended Decked dams perform very satisfactorily if they are build of and rest on sound strong rock
- Petrographic studies should be made of material proposed for use in embankment dams to understand their physical & chemical properties
    - Strength loss due to saturation
- Under high confining pressures the angle of friction is lower than under low confining pressures
- Each rockfill layer is heterogeneous due to variation of gradings between trucks and due to the process of damping and spreading

Rockfill

- ✓ Rockfill must be "free draining"

- Simple test
  - Excavate a hole
  - Fill with water
  - If it falls at a rate exceeding $^{75mm}/_{10min}$ it can be accepted

- Grading of coarse rock zone
  - At least 10% > layer thickness (not very big)
  - At least 25% average dimension > $^{3}/_{4}$ layer thickness
  - Less than 25% average dimension < 30% layer thickness

Rockfill

- Compaction
  - Steel drill vibratory rollers of dead weight $10^T - 15^T$
  - Over compaction may cause loss of strength due to crushing
  - Generally vibration effect is between $1^m - 1.2^m$ deep; optimum at 0.8^m
  - In large dams

    Trial embankment is desired

    – Layer thickness

    – Number of passes

    – Settlement compatibility between core and shell

Decked Rockfill Dams

- Decked Rockfill Dams
  - Timber face
  - Steel face
  - CFRD
  - AFRD

- ✓ CFRD
  - Shuibuya; China; 233^{m} height
  - Bakun; Malaysia; 205^{m} height
  - Siahbishe Dam; 82.5^{m} height

CFRD

- ✓ CFRD (continued)

  It can allow considerable flow without damage

  "List of Dams From Cook Paper"

  Knight Creek Dam; 34^{m} high

  Settlement 12^{mm} , due to hard igneous rock and unyielding foundation

  ✓ Plinth &Face slab

  The concrete plinth → support for face slab

  → grout cap { consolidation / grout curtain }

  • Gallery may be provided for future access

CFRD

- ✓ Plinth &Face slab (continued)

Gradient < 20 For very sound rock

< 10 For good rock

- It should be determine with cautions for foundation of "Not Sound Rock"
- Plinth depth:

Enough to reach sound non-erodible rock

Min. Steel 0.5%

Transverse joints at 6^m – 10^m intervals (with water stops)

CFRD

- ✓ Face Slab

$$t = 0.3 + 0.002h \leftrightarrow t = 0.3 + 0.005h$$

Even

$$t = 0.3 + 0.0075h$$

❖ See table from B.Cook

$$h = 70^m \quad \rightarrow \qquad t = 0.3 + 0.14 = 0.44$$

- vertical construction joint spacing controlled by construction method $\cong 12^m$?
- Steel $\cong 0.5\%$ both ways

CFRD

✓ Face Slab (continued)

- Near the abutments, joints interval is decreased and where tension expected reinforcement may increase
- Use of P.V.C. under the facing for controlling leakage in Pozo de los Ramos Dam in Spain

Asphaltic Face

- Asphaltic concrete face

  Greater Flexibility

  Requirements:

  - Stability on the selected slope
  - Durability
  - Impermeability
  - Resistance to water pressure, wave action and impact by flowing debris
  - Safety against hydrostatic uplift
  - Adequate drainage from beneath the facing and from the fill
  - Strength & elasticity to withstand local deformation without fracture

Asphaltic Face

- Ghrib Dam, Algeria → heaving of face slope 1:1
- BouHanifia Dam, Algeria → without incident (55^m high)

✓ Normally in layers

$$4\ years \leftarrow 75^{mm} \Rightarrow 90^{mm} \rightarrow 2\ layers$$

- Asphalt content enough to ensure stability
- The most common heights $50^m - 70^m$

– East Side Dam, Hong Kong $> 100^m$ high

Slope 1:1.6 → 1:1.7 , 1:1.5 , 1:1.4

– Scot Peak Dam, Australia; 46^m high

Facing two layers $60^{mm} - 75^{mm}$ thick

Addition layer $37^{mm} - 50^{mm}$ where $H > 30^m$

Asphaltic Face

- ✓ For high dams exceeding 100^m height

  Requirements to be considered carefully:

  - Stability of facing

    Composition of asphaltic concrete

    Mineral filler to reduce air voids

  - Durability

    In Algeria: Applied heat-reflecting paint

    In Northern Latitude: To prevent ice adhering to asphaltic concrete

  - Impermeability $< 4\%$ voids
  - Structural strength & Flexibility

Foundation treatment

- Foundation treatment
  - Quality of abutments often decreases with height above river bed, ∴ no relaxation of standards or attention as the dam rises
  - A conservative approach to foundation treatment is recommended. Once the fill is placed, there is no second chance

✓ Requirement

1. The rock under the core, materials in faults, joints,… must be non-erodible or must be protected from erosion
2. Core material prevented to enter joints, cracks,… and then may be back into the shell
3. Core contact must remain tight, after initial filling and long term despite the distortion in the dam due to weight of dam & water

Foundation treatment

4. Seepage through foundation must be controlled and discharged so that excessive pressure do not develop within D/S shell or foundation beneath the D/S shell

Special attention if the rock dips D/S :

1. Remove all weak unconsolidated materials that
2. may cause excessive settlement or instability

- If permeable, a cutoff is necessary
- Its dimension controlled by the mech. excavators
- Depth controlled by the required hydraulic gradient
- If blanket is to be placed again, items 1 & 2 above applied for the whole area under blanket

Foundation treatment

- ✓ If it is founded on rock
  - – Under core
  - – Under transition zone
  - – Under plinth

    All materials other than "satisfactory rock" must be removed

- For CFRD's plinth foundation a higher quality rock is required unless additional provisions are considered.
- Use of explosive if necessary must be "rigidly" controlled

Foundation treatment

- No overhang in foundation or abutments
- Even stepped foundation may result in stress relief. Dwarf walls may be built

✓ Dental concrete

✓ Foundation slopes; under the core should preferably converge toward the river D/S or be ⊥ to the Dam axis, however, divergence may be accepted if in limited areas (7^m to 10^m vertically) and not exceeding more than 10° from ⊥ to the dam axis

Divergence up to 15° over 30% – 40% of core width

Divergence over 20° not longer than $1^m – 2^m$

✓ No continuity between faults or weaknesses

Foundation treatment

- After final clean up a plastic clay layer is placed under and around the core

  Thickness depends on the dam height (between 1^m – 2^m thick)

- Consolidation grouting

- Piping cut-off slab

  for contact of core & foundation; may be used as grout cap

Foundation treatment

- Grout curtain
  - Single line
  - Double line
  - Multiple line

- In situ test

  Later injections will be expensive and less effective

  Min. curtain depth $\geq \frac{H}{2}$

- Aswan Dam 220^{m} deep curtain

  Lugeon Test?

  Value of water lost

Foundation treatment

- ✓ Foundation beneath the rockfill shell:
  - All unsuitable materials should be removed
  - Normally desirable to remove all earth and clay
  - Pervious consolidated gravel is left in place, provided stability is checked for a lower friction angle

- ✓ Nothing should be left which endangers

  Stability or excessive settlement or induce water seepage

Foundation treatment

✓ At Bellfield Dam, Australia

- U/S rockfill foundation contained relief joints up to 25^{mm} width filled with unconsolidated clay to depths $4^{m} – 5^{m}$
- It was costly to remove and replace all
- It was converted to rockfill by blasting in situ to a depth of 6^{m}

Settlement

- Settlement during construction
  - ✓ Estimate

$$S = \frac{\gamma}{E}(H - x)x$$

S: settlement at a particular level (Note: S=0 at x=0 and x=H!)

E_r , γ_r : rockfill modulus of deformation and density, respectively

H: dam height

Modulus & Density vary with the state of stress

Settlement

- At the end of construction:

$$S^{(m)} = 0.035(H - 13) \qquad \textit{by Speedie}$$

After construction:

$$\log S_5 = 0.017\mathrm{H} - 1.35$$
$$\log S_{10} = 0.0156H - 1.16$$

S_5 and S_{10} : settlements five and ten years later

- **Settlement**

- $S_l = S/[1000 * H * \log(t_2 / t_1)]$

- where s is the crest settlement measured in mm between times t_1 and t_2 since the completion of the embankment at a section of the dam H meters high (Charles, 1986).
- Values of $S_l > 0.02$ indicate that mechanisms other than creep or secondary consolidation contribute to the dam settlements (Tedd et al., 1997).

Settlement

- Some camber may remain ≅ 0.5% of crest length
- In free draining rockfill, settlement will occur following crushing of the points of contact and is ∴ function of rock hardness
- Particle breakage → change in shear strength
- In a zoned dam; differential settlement
- Rule of thumb [2% H] : post construction settlement (no foundation settlement)

Settlement

Post-construction settlement of rockfill dams analyzed via adaptive network-based fuzzy inference systems

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Received 10 August 2000; received in revised form 14 June 2001; accepted 6 July 2001

Based on field measurements of 82 rockfill dams with
1.Vertical core 2. sloping core 3. compacted membrane faced 4.Dumped membrane faced

Settlement

$$I_C = 1 - i_E \times i_F$$

Table 2
Embankment compaction index

Compaction method	Lift thickness (m)		
	< 2	2–3	> 3
Compacted with roller	1.0	0.5	0.25
Dumped, sluiced	0.2	0.15	0.1
Dumped, not sluiced	0.1	0.05	0.0

Table 3
Foundation quality index

Sound bedrock	Poor or weathered bedrock	Thick riverbed deposit (> 10 m)
1.0	0.5	0.1

Settlement

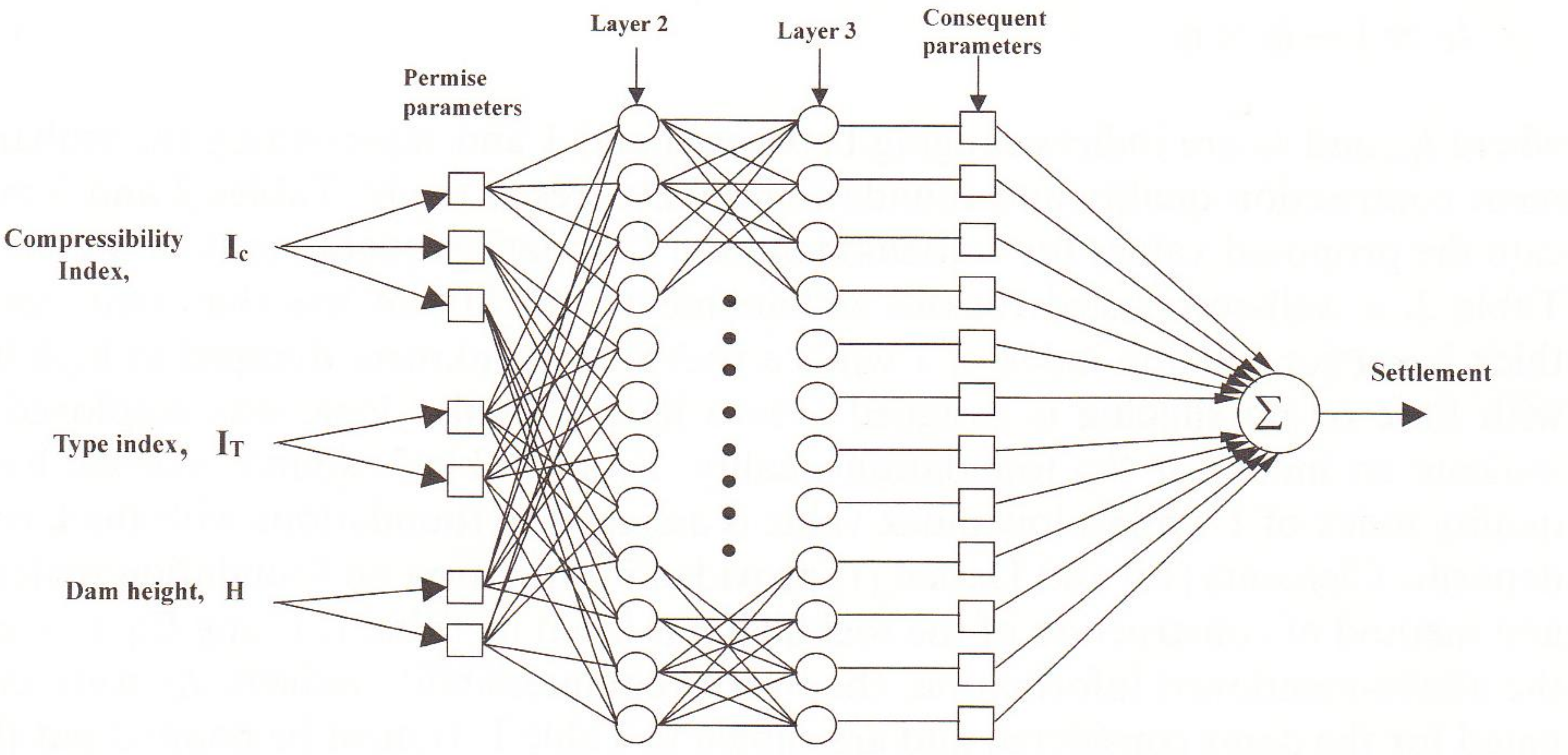

Fig. 3. Architecture of adaptive neurofuzzy network used for predicting dam settlement.

Settlement

G. Habibagahi / Computers and Geotechnics 29 (2002) 211–233

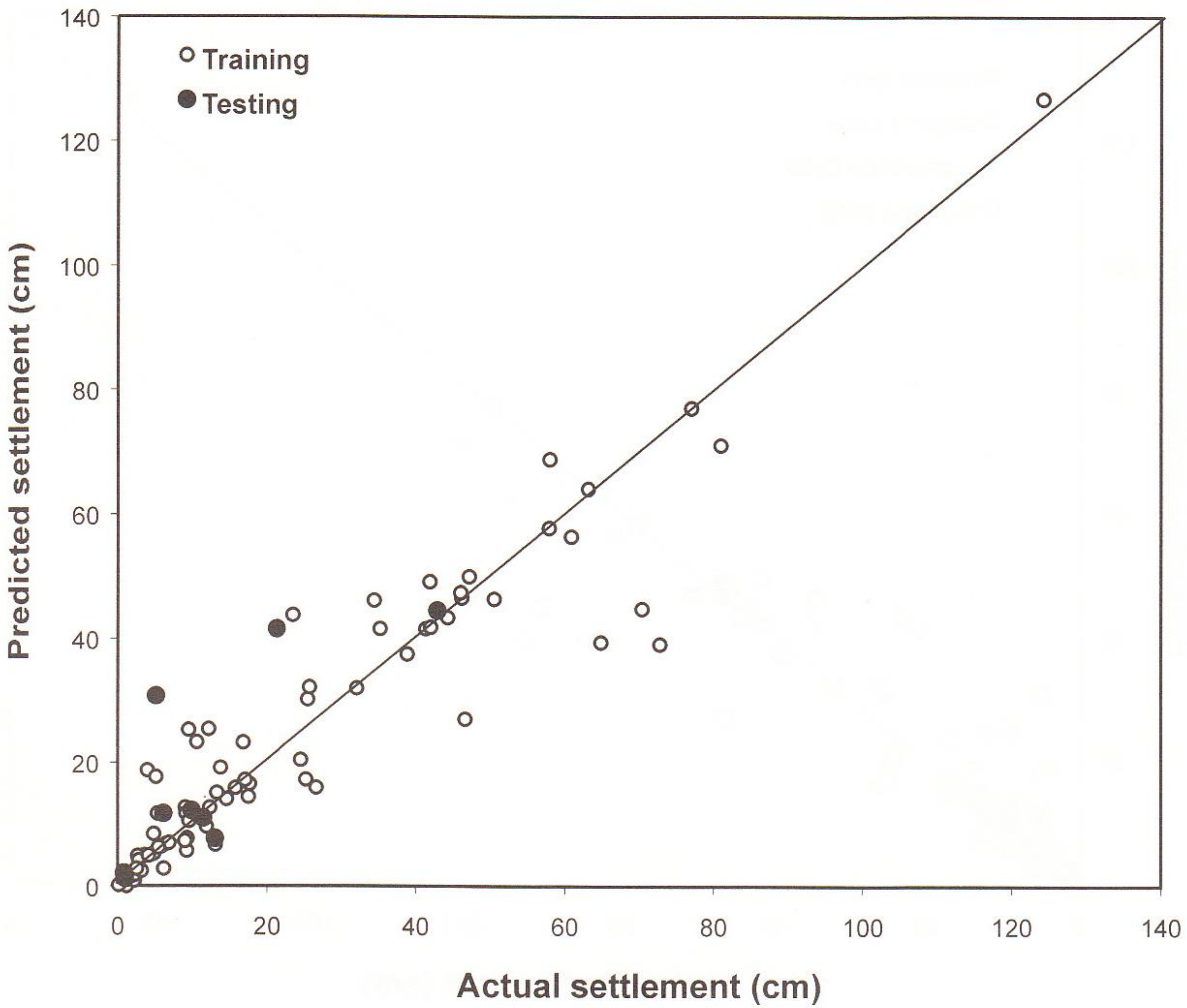

Fig. 8. Training and testing patterns predicted using neurofuzzy inference system.

Settlement

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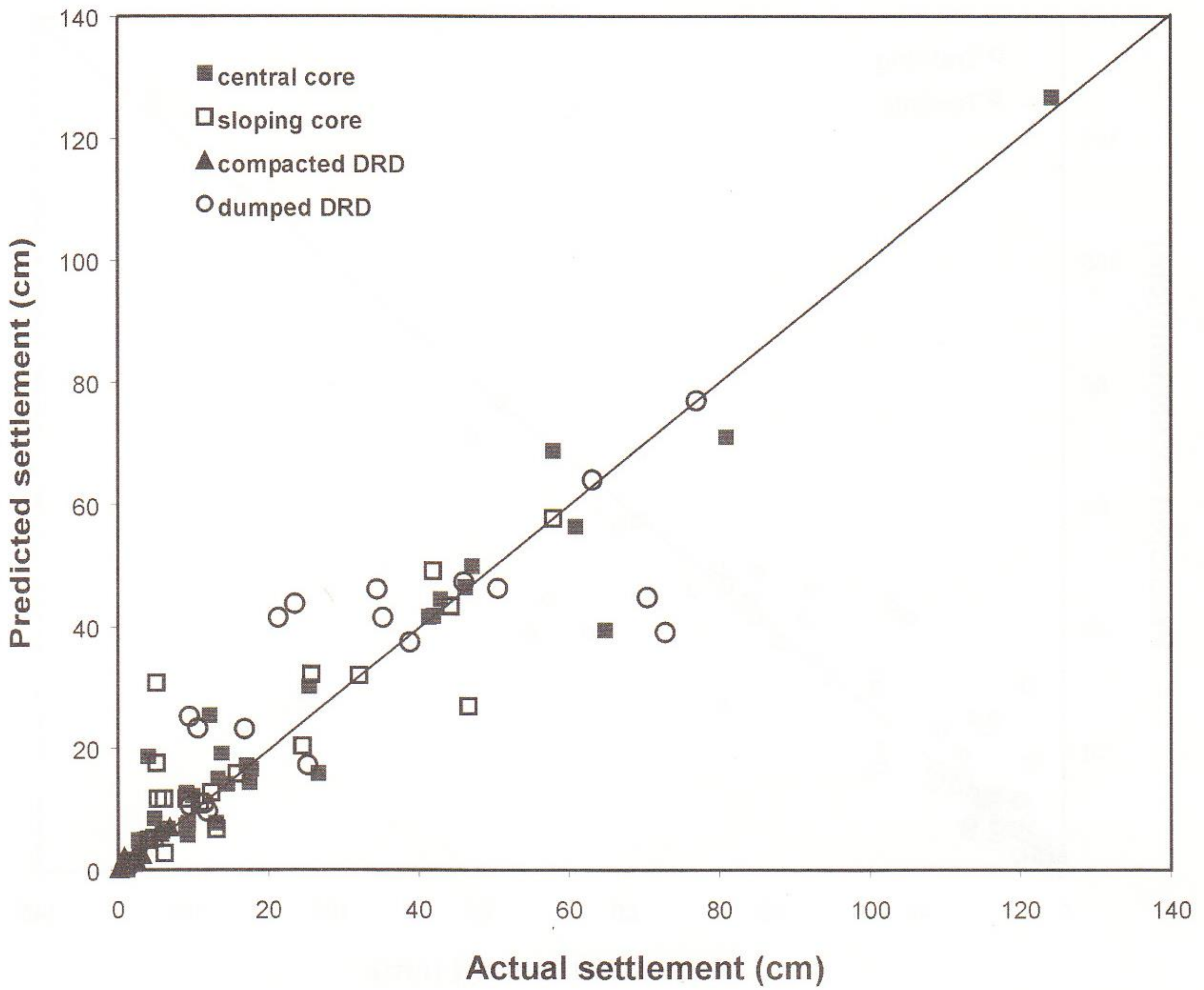


Fig. 9. Predicted versus measured values of crest settlement for different types of rockfill dams.

Dynamic Analysis of Earth Dams

A. Pseudo-Dynamic Analysis
B. New-Mark Method
C. Seed, Sarma, Ambraseys Method
D. Finite Element Simulation

Dynamic Analysis of Earth Dams

A. Pseudo-Dynamic Analysis (Pseudo-Static?):

- Same as limit equilibrium approach, but an inertia force is included equal to: mass × Max. base ground acceleration
- No information is available on magnitude of displacements
- Due to alternating change in direction of earthquake forces and due to the short period of Max. load applied, a $F_s = 1$ may not represent complete failure of the slope

Dynamic Analysis of Earth Dams

- Dynamic analysis (Seismic design)

Possible ways in which an earthquake may cause failure of an earth dam:

1. Disruption of dam by major fault movement
2. Loss of freeboard
3. Slope failure
4. Sliding of dam on weak foundation material
5. Piping failure through cracks induced by ground motion
6. Overtopping due to seiches in reservoir
7. Overtopping due to slides or rock falls into reservoir
8. Failure of spillways or outlet works

Dynamic Analysis of Earth Dams

- Defensive measures

1. Ample freeboard to allow for settlement, fault movement, …
2. Wide transition zones of materials not vulnerable to cracking
3. Use chimney drain near central portion of the embankment
4. Ample drainage zone to allow for possible flows through cracks
5. Use wide core zones of plastic materials not vulnerable to cracking
6. Use a well graded filter U/S as a crack stopper
7. Crest details to prevent erosion in the event of overtopping

Dynamic Analysis of Earth Dams

- Defensive measures (continued)

8. Flare embankment core at abutment contacts
9. Locate the core to minimize the degree of saturation of materials
10. Stabilize slopes around the reservoir rim to prevent slides
11. Relocate the dam if danger of fault movement or provide special details
12. Double dam system in special situations (Los Angeles Dam) to protect people D/S

Dynamic Analysis of Earth Dams

- Pseudo static analysis:

Equivalent acceleration coefficient	n_g
Severe earthquake	0.1
Violent, Destructive	0.25
catastrophic	0.5

- ✓ Actual peak acceleration on the sliding mass may be much more than the above values but due to transitional nature (short duration of max. acceleration) and allowable deformation, it is considered adequate

In US	$0.05 \leq n_g \leq 0.15$
In Japan	$0.15 \leq n_g \leq 0.35$

Dynamic Analysis of Earth Dams

- It does not indicate stability even if $F_s > 1$ is obtained
- Examples:

Dam	n_g	F_s	description
Sheffield	0.1	1.2	Complete failure
Lower San Fernando	0.15	1.3	Upstream slope failure
Upper San Fernando	0.15	2 – 2.5	Downstream slipped $6'$
Tailing (Japan)	0.2	$\cong 1.3$	Failure with release of tailing

❖ Reasons to be explained later

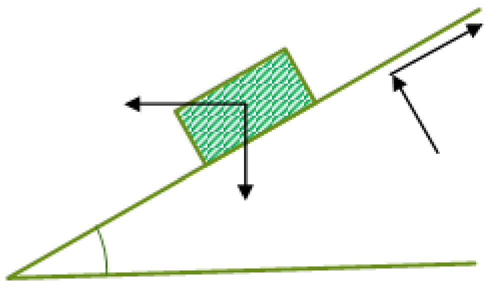
✓ Leshchinsky& San (1994)(ASCE, J.G.E. vol.120, PP 1514-1532)

Proposed a variational limiting approach and provides appropriate design charts

Dynamic Analysis of Earth Dams

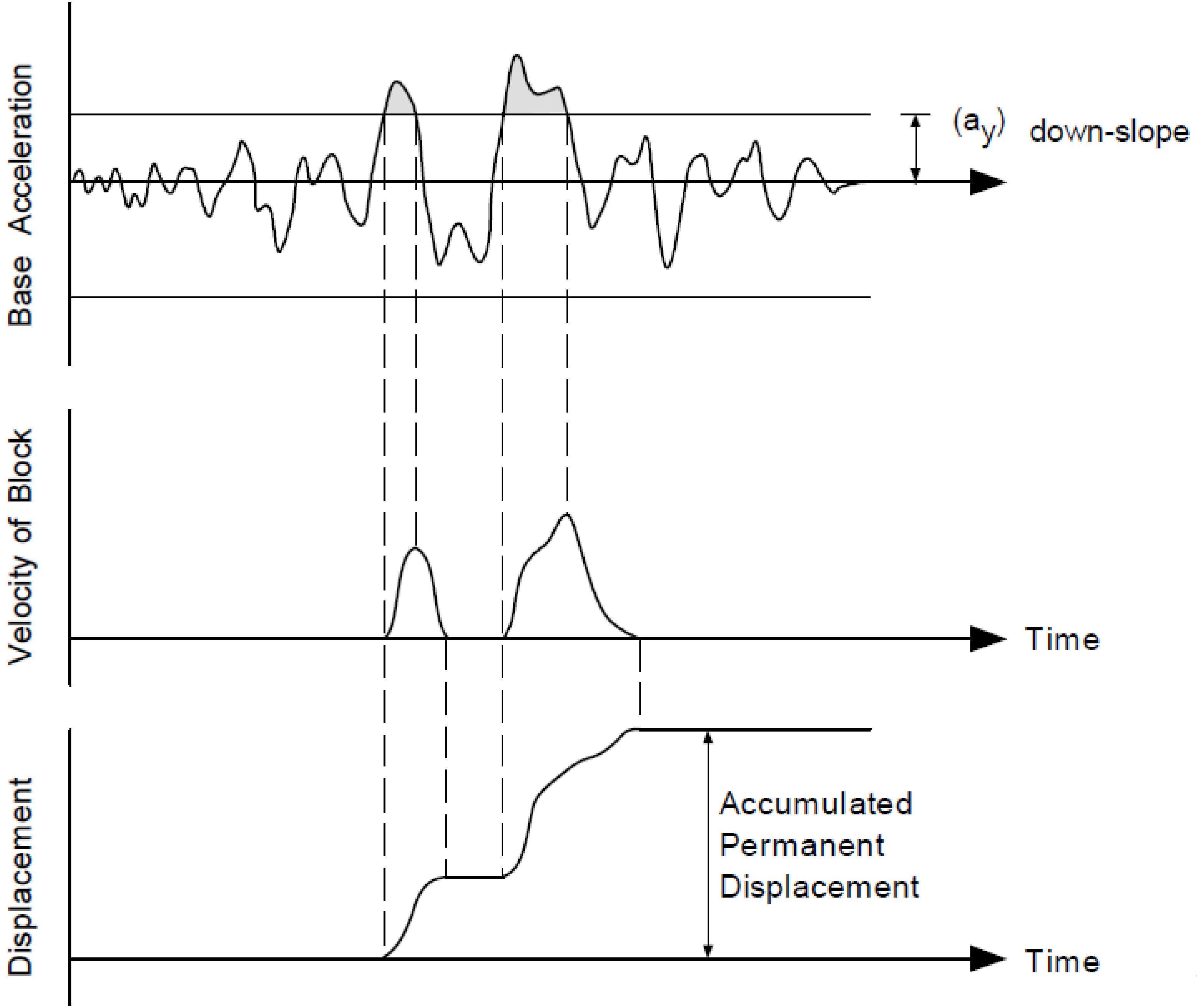
B. Newmark Method

- Newmark (1965) in Rankine lecture proposed the method
- The soil slides along a failure surface and is assumed "rigid"



"Block Slide"

- Displacement are determined for assumed failure surfaces by double integration of acceleration which exceed the yield acceleration


Base Acceleration
(a_y) down-slope
Velocity of Block
Time
Displacement
Accumulated
Permanent
Displacement
Time

Dynamic Analysis of Earth Dams

- C. Newmark Method Modified by various authors (Seed, Sarma, Ambraseys)

  - ✓ Ambraseys& Sarma (1967) (Geot. Vol. 17 ,PP 181-213)
    - Fundamental period of oscillation, T_0:

$$T_0 = 2.61\left(\frac{h}{s}\right)$$

S: shear wave velocity $= \sqrt{\frac{G}{e}}$

- Periods of higher modes

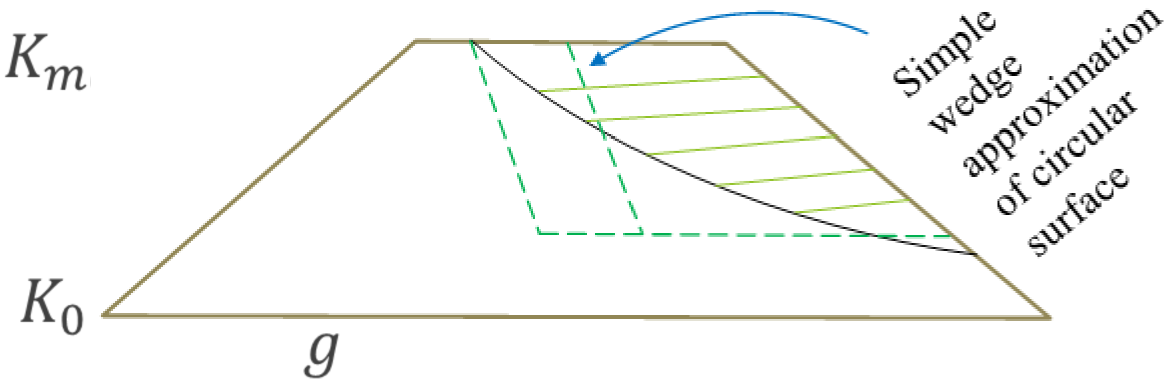
$$T_n = T_0\left(\frac{a_0}{a_n}\right)$$

a_n: n^{th} root of the Bessel function $J_0(z) = 0$

Dynamic Analysis of Earth Dams

- ✓ Average acceleration coefficient for a given failure surface

$$\overline{K_n}\,.\,K_0 = K_{max_{ave}}$$



$$\overline{K_n} = \frac{\sum m_i a_i}{(a_{max})_{base}\sum m_i}$$ K_0: *Max base ground accel. coef.*

Figure 18 of the paper:

$Y = nh$ as 'n' is smaller $\Longrightarrow$ higher values of

$\overline{K_n}(\Longrightarrow K_{max_{ave}})$

Dynamic Analysis of Earth Dams

- Figure 19

  Average curves for a number of strong earthquakes

- Figure 20

  Correction of *Fig.19* for damping values other than $\upsilon = 20\%$ given in *Fig.19*

- For $n > 1.0$ in this paper new results are presented: *Fig.23* , *Fig.24*

❖ Strong ground movements from near earthquakes will cause smaller acceleration in high dams than in low dams

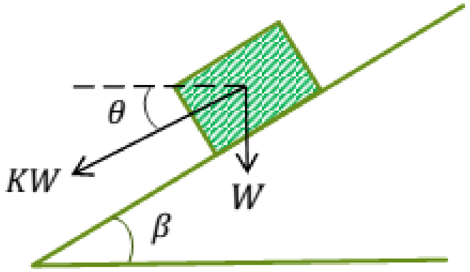
❖ A deep slide will be subjected to smaller overall acceleration than a small slide near the crest, or free surfaces

❖ Near the crest is the most vulnerable location

Dynamic Analysis of Earth Dams

- Sarma (1975)

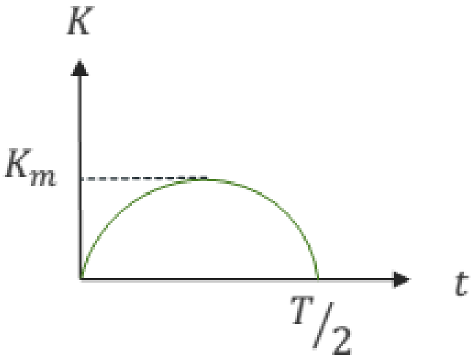
$$\ddot{x} = g\frac{\cos(\beta - \theta - \acute{\varphi})}{\cos\acute{\varphi}}(K - K_c)$$



- Solution:

- For Half Since pulse:

$$\Rightarrow \frac{4x_m}{K_m g T^2}\left[\frac{\cos\acute{\varphi}}{\cos(\beta - \theta - \acute{\varphi})}\right]$$



$$= \frac{(\frac{K_c}{K_m} - \sin q)^2}{(2\pi^2\frac{K_c}{K_m})}$$

"For $0.725 \leq \frac{K_c}{K_m} \leq 1$"

Dynamic Analysis of Earth Dams

- $$= \left[\frac{K_c}{K_m} + \alpha - \pi + cos^2(\alpha/2)\cot(\alpha/2)\right] / \pi^2$$

  "For $0 \leq \frac{K_c}{K_m} \leq 0.725$"

– Where

$$q = \alpha + \frac{K_c}{K_m}(\cos\alpha - \cos q)$$

$$\alpha = \sin^{-1}(K_c/K_m)$$

K_m: Max. value of seismic coefficient Of Earthquake record

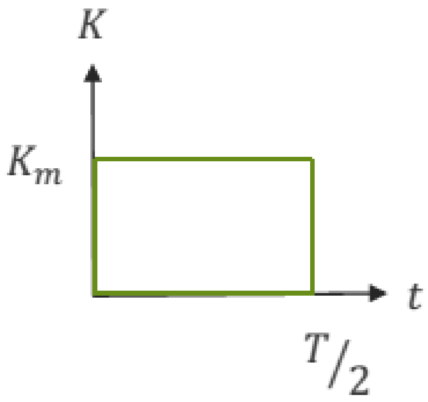
T : Predominant period of Earthquake acceleration

K_c : Critical seismic coefficient ($F_s = 1$)(yield coefficient)

Dynamic Analysis of Earth Dams

- ✓ Sarma (1975)

- For Rectangular pulse:

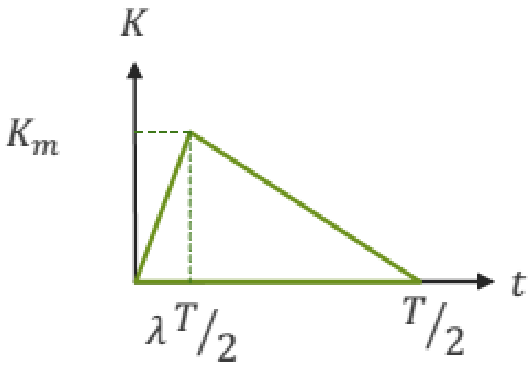


$$\Rightarrow \frac{4x_m}{K_m g T^2}\left[\frac{\cos \acute{\varphi}}{\cos(\beta-\theta-\acute{\varphi})}\right]$$

$$=\frac{1}{2}\left(\frac{K_m}{K_c}-1\right)$$

Dynamic Analysis of Earth Dams

- ✓ Sarma (1975)



- For Triangular pulse:

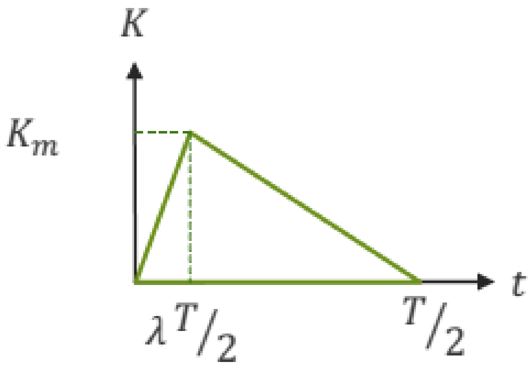
$$\Rightarrow \frac{4x_m}{K_m g T^2}\left[\frac{\cos \acute{\varphi}}{\cos(\beta-\theta-\acute{\varphi})}\right]$$

$$= \frac{1}{24}\left[4\left(1-\frac{K_m}{K_c}\right)\left(1-\lambda\frac{K_m}{K_c}\right)-\left(1-\lambda\left(\frac{K_m}{K_c}\right)^2\right)\right] \Big/ \left(\frac{K_m}{K_c}\right)$$

"For $\quad 0 \le \frac{K_m}{K_c} \le [1-\sqrt{1-\lambda}]\,/\lambda$"

Dynamic Analysis of Earth Dams

- ✓ Sarma (1975)



- For Triangular pulse:

$$\Rightarrow \frac{4x_m}{K_m g T^2}\left[\frac{\cos\acute{\varphi}}{\cos(\beta-\theta-\acute{\varphi})}\right]$$

$$=\frac{1}{6}\left[\left(1-\frac{K_m}{K_c}\right)^3\left(2-2\sqrt{1-\lambda}-\lambda\right)\right]$$

"For $\left[1-\sqrt{1-\lambda}\right]/\lambda \leq \frac{K_m}{K_c} \leq 1$"

Dynamic Analysis of Earth Dams

- Fig. 8 , PP. 754 ; the solution is shown graphically
  - For $\frac{K_m}{K_c} > 0.5 \quad \Rightarrow Triangular\ curve$
  - For $\frac{K_m}{K_c} < 0.5 \quad \Rightarrow Rectangular\ curve$

- Example:

$K = 0.46g$ $Max.\, earthquake\ record$

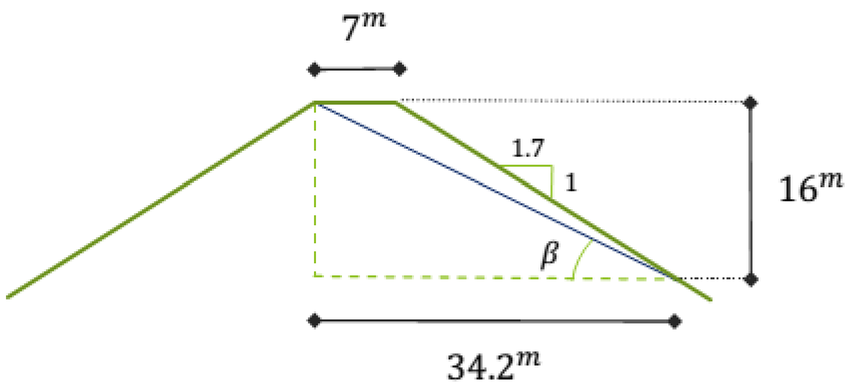
$K_c = \sin(\acute{\varphi} - \beta)$ $(Approx.)$

$\beta = 25^\circ$ $\acute{\varphi} = 49^\circ$ (near surface mechanism)

$$H = 80^m \qquad n = \frac{16}{80} = 0.2 \qquad T_0 \cong \frac{H}{200} = 0.4\ sec$$

Dynamic Analysis of Earth Dams

- $K_c \cong 0.4$
  $K_m \cong 0.46 \times 1.8 = 0.83$
  $\frac{K_c}{K_m} = \frac{0.4}{0.83} = 0.48$



$$\Longrightarrow \frac{1}{C} \cdot \frac{4x_m}{K_m T^2 g} = 0.15 \qquad \text{(Triangular)}$$

$$x_m = 0.15 \times \frac{\cos(25-49)}{\cos 49} \times (0.46 \times 1.8) \times 9.81 \times 0.4^2 = 0.27^m$$

- If using the Rectangular curve:

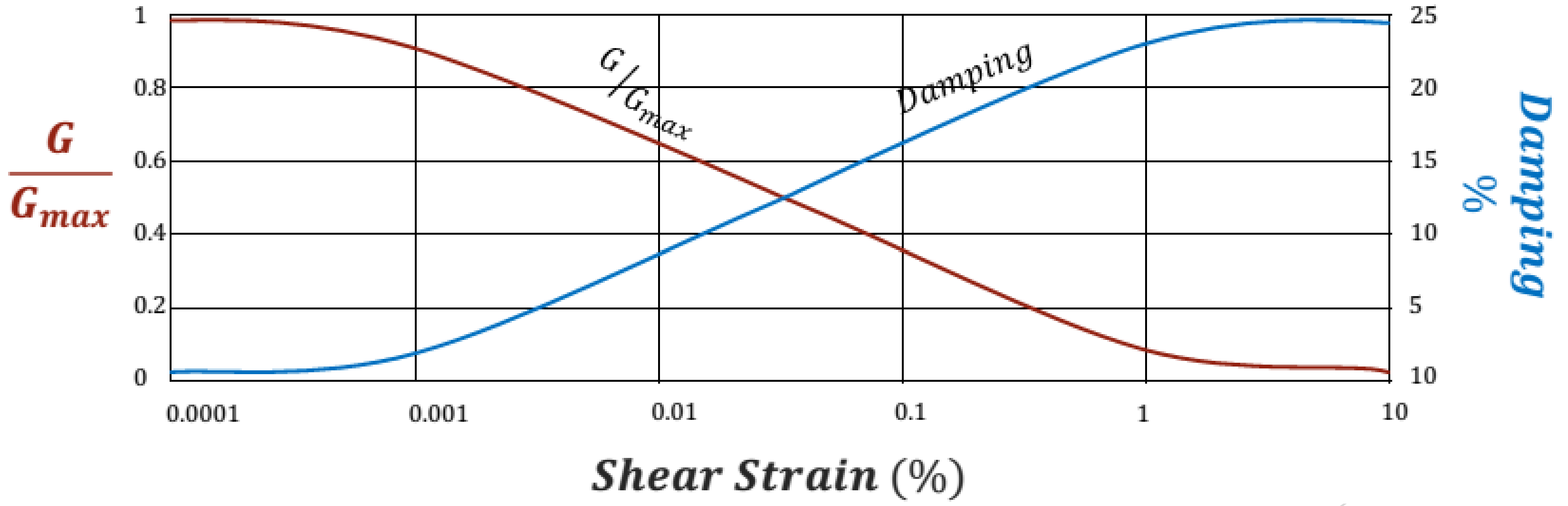
$$x_m \cong 0.27 \times 3^m = 0.81^m$$

Vertical displacement: 0.34^m

Dynamic Analysis of Earth Dams

✓ Makdisi & Seed (1978) (ASCE, Vol. 104, PP 849-867)

1. Earthquake Induced Acceleration:
   - Time history of earthquake induced acceleration on the slipping mass considered
   - Proposed using Quad-4 ; F.E. Program, using equivalent linear, strain dependent properties (Modulus & Damping)



Dynamic Analysis of Earth Dams

- 2. Average Time history of the sliding mass is calculated:

$$K_{ave}(t) = \frac{\sum m_i a_i}{g \sum m_i}$$

$\ddot{U}_{max}$: Crest acceleration

$(K_{ave})_{max}$: Max. average acceleration for a potential sliding mass

Dynamic Analysis of Earth Dams

3. They proposed a relationship between $\ddot{U}_{max}$ and $(K_{ave})_{max}$ with depth ratio $^{y}/_{h}$ (Fig 7)

- For the previous Example:

$$n = {}^{y}/_{h} = 0.2 \implies \frac{K_{max}}{\ddot{U}_{max}} \cong 0.85 \implies K_{max} = 0.85\ddot{U}_{max}$$

$$\ddot{U}_{max} \cong g \quad (estimated\ from\ F.E.calculations)$$

$$\therefore\ K_{max} = 0.85g$$

Which compares well with $K_m = 0.46 \times 1.8 = 0.82g$

Dynamic Analysis of Earth Dams

4. Yield acceleration assumed constant throughout the earthquake. Embankments height considered between $23^m - 46^m$. Response studied using ground acceleration representing 3 different earthquake magnitudes: $6\frac{1}{2}, 7\frac{1}{2}, 8\frac{1}{4}$

(Fig 10, Fig 11)

- For the previous Example:

$$\frac{K_y}{K_{max}} = 0.48$$

From Fig:

$$\Longrightarrow \frac{U}{K_{max} g T} = 0.1 \quad \Longrightarrow U = 0.1^{sec} \times 0.83 \times 9.81^{m/s^2} \times 0.4^{sec} = 0.32^m$$

Compare with 0.34^m

Dynamic Analysis of Earth Dams

- ✓ Summary:

Find
$$\begin{cases} \ddot{U}_{max}\ (crest),\ T_0\ (First\ natural\ period) \\ K_y \end{cases}$$

$Fig.\ 7 \Longrightarrow (K_{max})_{ave}$ for the specified slide

$Fig.\ 11 \Longrightarrow$ displacement

- ❖ Note:

"No reduction in strength is allowed due to cyclic loading"

Dynamic Analysis of Earth Dams

- ✓ Seed (1979)

  "Rankin Lecture" (PP 215-263)

  - As explained in pseudo-static approach, some dams with $F_s > 1$ had failed
  - But Seed (1979) mentions 33 earth dams within 35 miles ($\cong 57^{\mathrm{km}}$) of San Andreous fault and 15 within 5 miles ($\cong 8^{\mathrm{km}}$) on which a $8\frac{1}{4}$ magnitude earthquake occurred

  Distance $\leq 57^{km}$: Estimated peak ground acceleration $> 0.25g$

  Distance $\leq 8^{km}$: Estimated peak ground acceleration $> 0.6g$

  "Non Suffered Any Significant Damage"

  - They were constructed of clayey soils on rock or clayey soil foundation
  - 2 Dams were constructed of sand, and the sand was not saturated

Dynamic Analysis of Earth Dams

- Akiba &Semba(1941)

Studied 12 cases of complete dam failure, 40 cases of slope failure and concluded:

- Most failures occurred few hours up to 24^{hrs} after earthquake
- Majority consisted of sandy soils
- Clayey embankments even close to epicenter did not fail

The time lag between the earthquake and failure:

- Due to piping through cracks induced by the earthquake
- Slope failure from P.W.P redistribution

Dynamic Analysis of Earth Dams

- For dams constructed of saturated cohesionless soils, and subjected to strong shaking, a primary cause of damage is the build up of P.W.P in the embankment and possible loss of strength as the result
- This type of failure can not be predicted by a pseudo-static type analysis
- All cases of slope failure reported involved sandy soils

Dynamic Analysis of Earth Dams

- Seed (1979) used the method proposed earlier based on Newmark approach to calculate deformation for a yield acceleration coefficient: $K_y = 0.05 , 0.1 , 0.15 , F_s = 1.15$

❖ (Considered less than 15% strength loss)

$\Longrightarrow$ Computed displacements for different magnitudes of earthquake $6\frac{1}{2} , 8\frac{1}{4}$, and crest acceleration less than 0.75g are within acceptable limits

- For the above conditions it suffices to analyze for the following conditions:

  Magnitude $6\frac{1}{2} : F_s = 1.15 , n_g = 0.1$

  Magnitude $8\frac{1}{4} : F_s = 1.15 , n_g = 0.15$

$\Longrightarrow$ "Reasonable displacements"

(Up to $3' \cong 1^m$ displacement (Max.))

Dynamic Analysis of Earth Dams

- Soils do not lose more than 15% of original strength:
  - Many clayey soils
  - Some dense saturated sands
  - Clayey sands
  - Dry sands
  - Saturated sand or gravel with high permeability ($K > 1\ {}^{cm}/_{s}$)

Dynamic Analysis of Earth Dams

- CFRD (Gazetas&Dakoulas) (1992)

1. Filter zone beneath the face
   - 40% passing No.4 sieve to limit $K < 10^{-3}\ {}^{cm}/_{s}$
   - Others question, it may remain saturated and may have detrimental effects during shaking

2. Rockfill
   - $C_u > 20$
   - about 30% finer than 1"

10% finer than 1" for $K > 1\ {}^{cm}/_{s}$

$\therefore$ not allowed to be saturated

$$E_H > 3 \times E_V \ \ (measurements)$$

Dynamic Analysis of Earth Dams

3. Hydrodynamic effect may be safely ignored

4. Generally considered that the crest settlement in modern CFRD would not exceed $1\% - 2\%$ of the dam height under the most severe earthquake shaking (side slope? Effect on settlement?)

$2\% \times 65^m = 1.35^m$

average $= 1^m$

$1\% \times 65^m = 0.65^m$

Sherard& Cook (1987):

This is acceptable, since a sudden crest settlement of $0.01H$ will not threaten the safety of a modern CFRD

Dynamic Analysis of Earth Dams

5. 3-D effect

   Rigid abutment $\Longrightarrow$ decrease in T

   $\therefore \Longrightarrow$ crest acceleration $\uparrow$

6. $\nu \cong 0.25$

7. Water pressure acts externally, $\Longrightarrow$ increase in T_m and $\Longrightarrow$ stiffer Dam $\Longrightarrow$ not large nonlinearity

8. Displacements of up to $1^m - 2^m$ do not pose any threat to CFRD's overall integrity

Dynamic Analysis of Earth Dams

9. To minimize problems:
   - Flatter slopes in the upper part
   - Filter sufficiently permeable
   - Increase freeboard
   - Flexible waterstopes
   - To use clay sand and gravel or soil cement or rollcrete in the $\frac{1}{4}$U/S under the parpetwall.

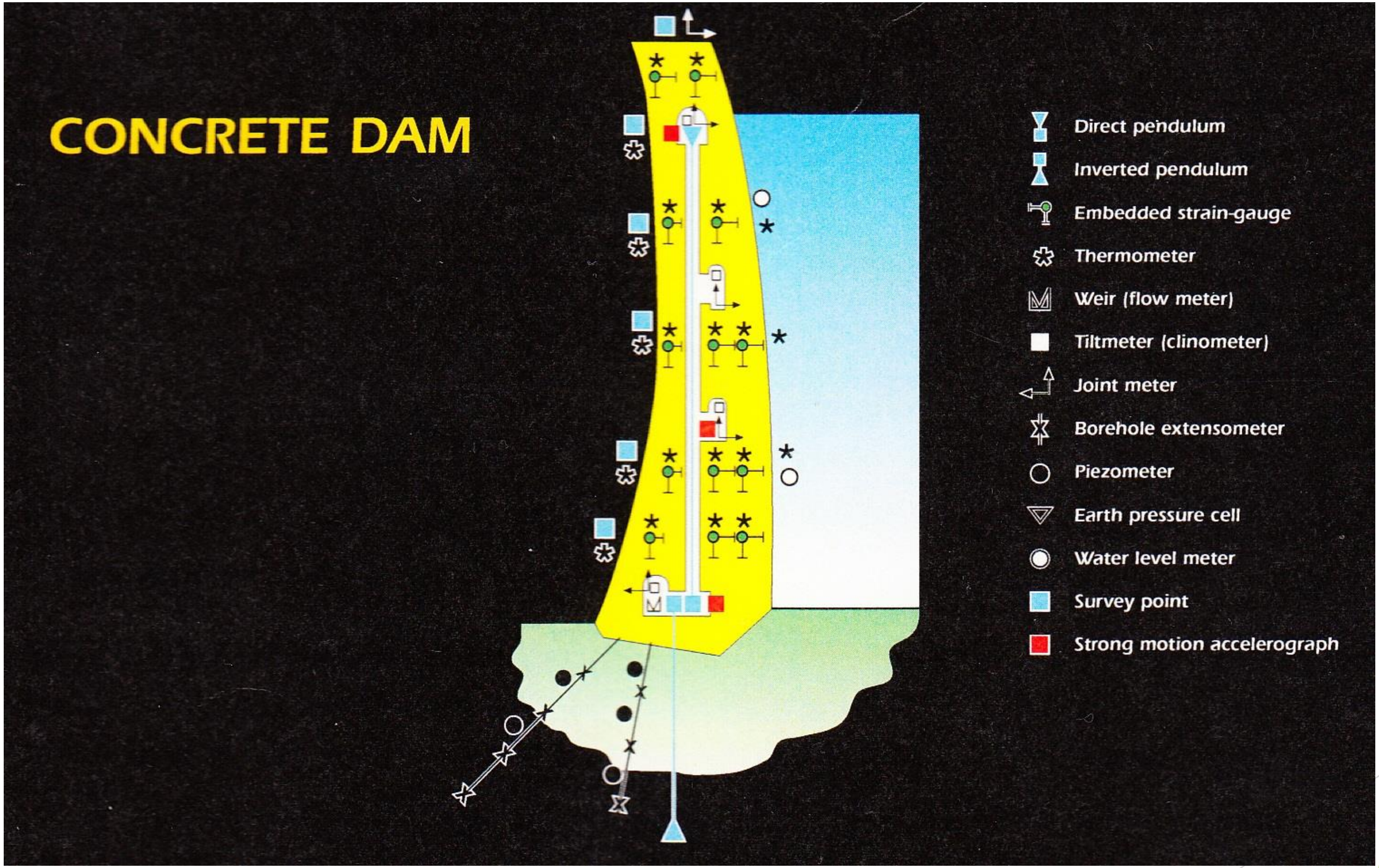
10. Stability of parpet wall is endangered during severe earthquakes

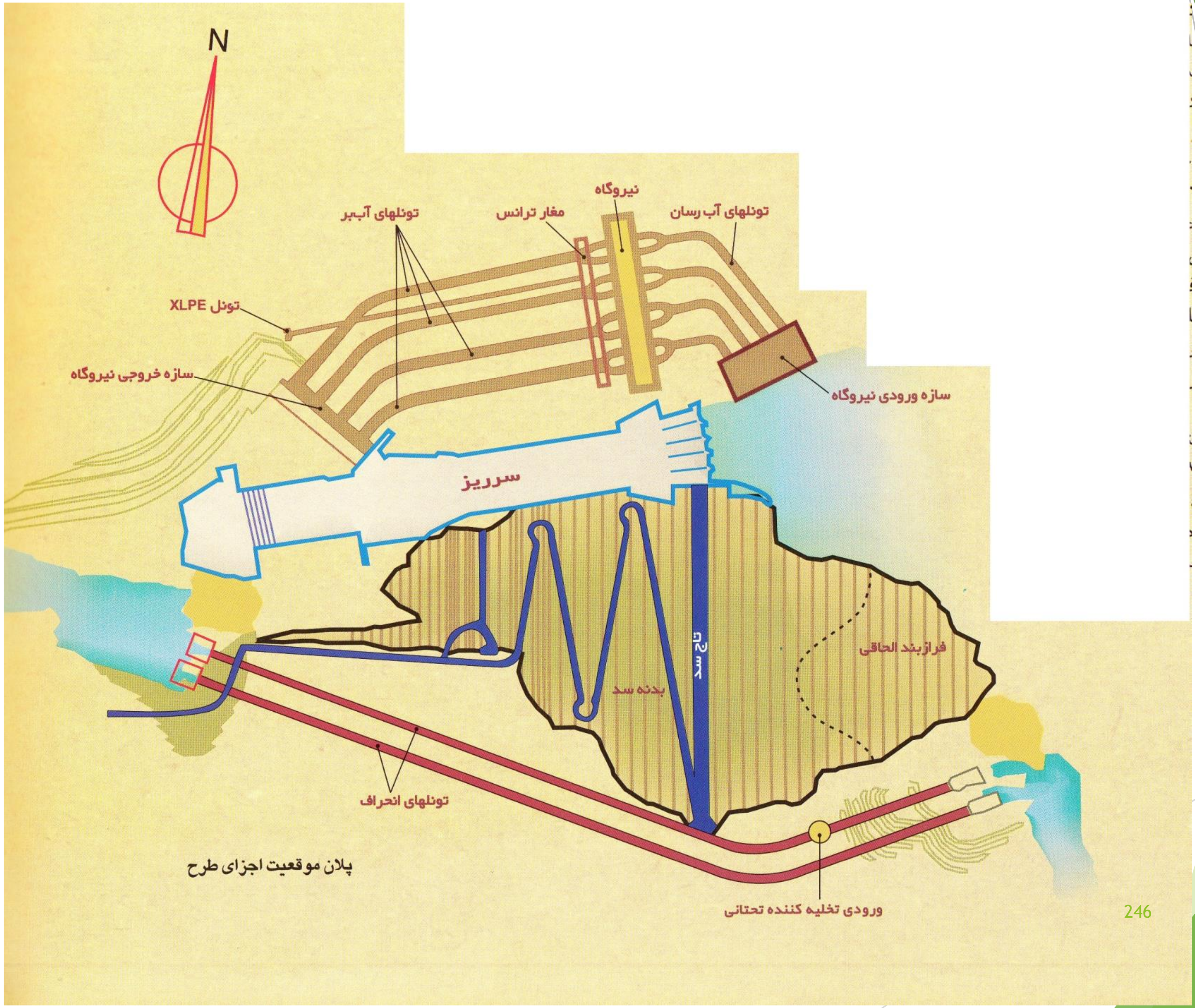
Dynamic Analysis of Earth Dams

- Uddein &Gazetas(1995) (ASCE, PP 185, Vol. 2)

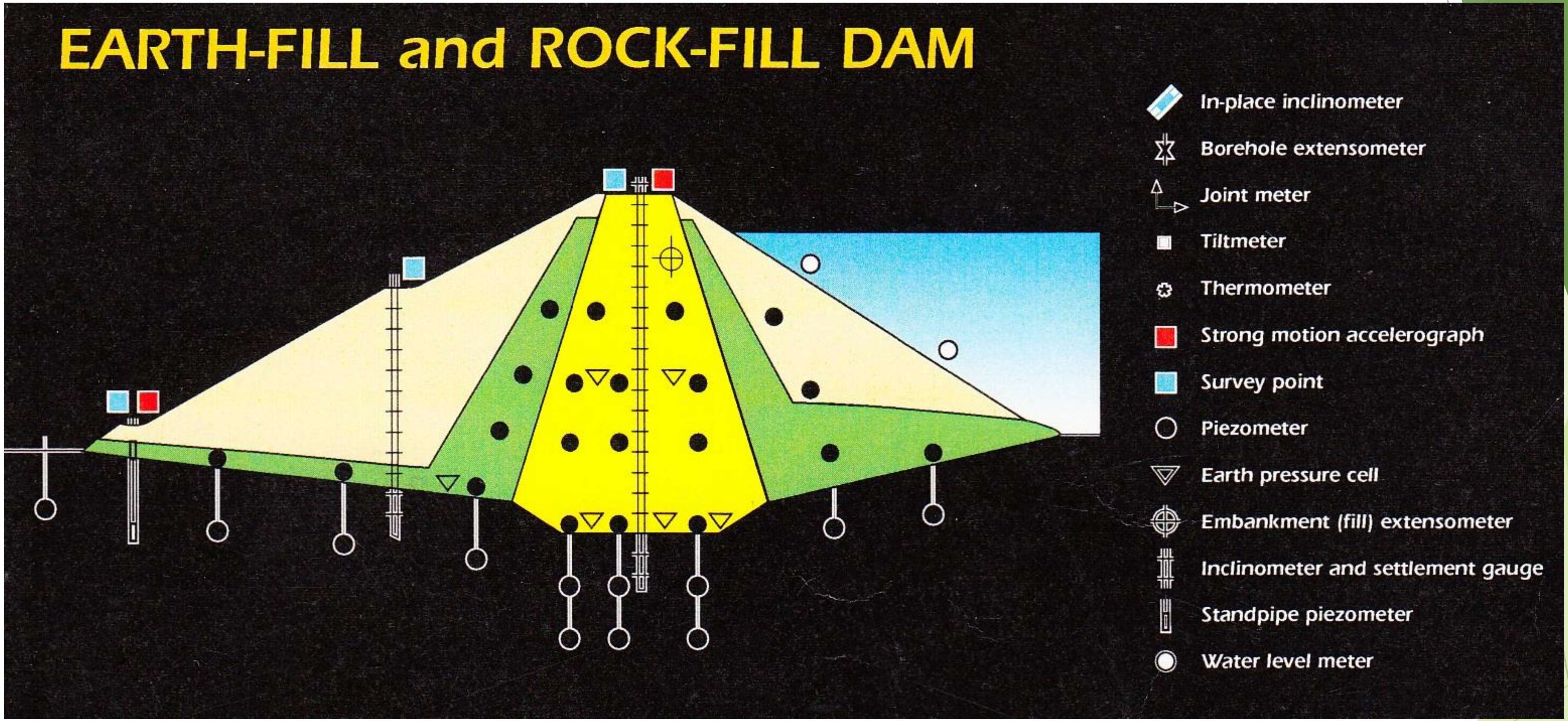
1. Vibration characteristics can be estimated, ignoring the slab
2. Crest acceleration $1.5-3 \times$ PGA depending on the frequency content of motion relative to the dam's natural frequency
3. Tensile stress in concrete facing exceeding tensile strength may develop $\Longrightarrow$ cracking of concrete and failure of joints

- Parpet wall: caution


CONCRETE DAM
Direct pendulum
Inverted pendulum
Embedded strain-gauge
Thermometer
Weir (flow meter)
Tiltmeter (clinometer)
Joint meter
Borehole extensometer
Piezometer
Earth pressure cell
Water level meter
Survey point
Strong motion accelerograph

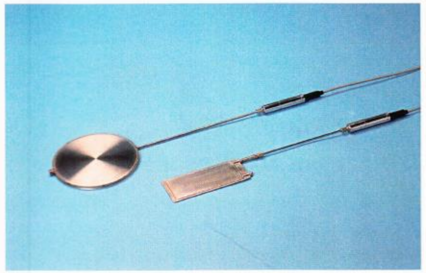


پلان موقعیت اجزای طرح


EARTH-FILL and ROCK-FILL DAM
In-place inclinometer
Borehole extensometer
Joint meter
Tiltmeter
Thermometer
Strong motion accelerograph
Survey point
Piezometer
Earth pressure cell
Embankment (fill) extensometer
Inclinometer and settlement gauge
Standpipe piezometer
Water level meter

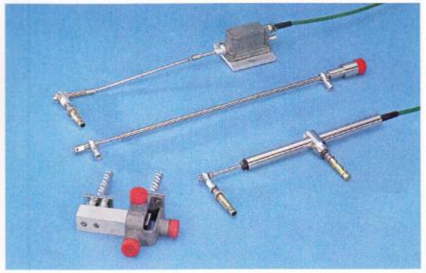
Vibrating wire piezometers

پیزومتر تار مرتعش



Earth pressure cells

فشار سنج تار مرتعش



Crackmeters and jointmeters

سیستم ترک سنج

Pendulum systems

سیستم پاندول

SINCERT

SIS GEO

Sisgeo was founded in 1993 to carry on the work of S.I.S. Geotecnica, which was set up in 1973.

Sisgeo manufactures and supplies a complete range of instruments for geotechnical (soil and rock), structural and environmental survey.

Sisgeo has its own Quality System certified by Det Norske Veritas in compliance with UNI EN ISO 9001 Standards .

Sisgeo has a staff who includes university graduates, engineers, geologists, workers and skilled technicians.

شرکت سیسجئو در سال ۱۹۹۳ میلادی بمنظور ادامه فعالیت شرکت اس ای اس ژئوتکنیکا که در سال ۱۹۷۳ تاسیس گردیده بود ، اغاز به کار نمود . شرکت سیســجئو دارای گواهینامه کنترل کیفیت Det Norske Veritas زیر نظر ایزو ۹۰۰۱ استاندارد (UNI ISO 9001) سازنده لوازم ابزار دقیق در کلیه زمینه های ژئوتکنیک (خاک و سنگ) ، سازه ها و سیستم پایداری می باشد . کادرشرکت سیسجئو شامل فارغ التحصیلان دانشگاه ، مهندسین ، زمین شناسان و تکنسینهای ماهر می باشد .

SAD AFZAR

شرکت سد افزار در سال ۱۳۷٤ بعنوان نماینده انحصاری شرکت سیسجئو ایتالیا (تولید کننده ابزار دقیق ژئوتکنیک و ژئومکانیک) جهت ارائه خدمات فروش و پشتیبانی فنی در جمهوری اسلامی ایران تاسیس گردید . این شرکت علاوه بر فعالیتهای فوق الذکر با همکاری شرکت سیسجئو و استفاده از متخصصین مجرب ایرانی در زمینه طراحی و تولید بخشی ازابزار رفتار سنجی و پایداری سدها در داخل کشور نیز فعال میباشد .

Inclinometer probe and readout

سیستم انحراف سنج و دستگاه قرائت

Sisgeo and

Repubblic of

- KARUN III
  Diversion
- TAHAN D
  Earth-fill
- IZADKHA
  Earth-fill
- SYVAND
  Earth-fill
- SALMAN-
  Concrete
- REIS-ALI D
  Concrete

Table 4.4. Recommended minimum diameters of drill cores.

Substratum	Uniaxial compressive strength (MPa)	Minimum drill core diameter (mm)	Borehole diameter (mm)[1]	Drilling procedure
Rock, very strong, slightly jointed	> 80	≥ 56	≥ 76	Rotary, diamond bits
Rock, moderately strong, moderately jointed	50 to 80	≥ 66	≥ 86	Rotary, diamond bits
Rock, strongly jointed and/or broken	20 to 50	≥ 80	≥ 101	Rotary, diamond or carbide bits
Rock, weak, friable	10 to 30	≥ 80	≥ 101	Rotary, carbide bits
Conglomerates, slightly cemented, without coarse gravel	10 to 15	≥ 90	≥ 116	Rotary, carbide bits
As above, with coarse gravel	5 to 15	≥ 120	≥ 150	Rotary, diamond bits
Cohesive material, very stiff (e.g. siltstone)	5 to 15	≥ 66	≥ 86	Rotary, carbide bits
Cohesive soil, plastic (silt and clay)	<.5	≥ 120[2]	≥ 150	Rotary, carbide bits or pipe driving without rotation

[1]Borehole diameter compatible with minimum core diameter at the use of double core barrel

[2]Undisturbed sampling for laboratory tests

Table 4.1. Investigations of the substrata and the natural constructio[illegible]aterials.

Type of investigations	Result	Sampling	Field tests
Geological mapping	General overview, identification of material deposits	–	–
Core drilling	Stratification of soils and rock	Rock and soil samples for lab tests	Water pressure tests and test grouting
Penetration tests	Stratification of soils, identification of material deposits	–	–
Test pits and test trenches	Stratification of soils, identification of material deposits	Undisturbed and disturbed soil samples for lab tests	Moisture content, moist unit weight, gradation
Adits and shafts	Rock conditions	Rock samples for lab tests	Rock mechanical tests
Geophysical tests (calibration by core drillings required)	Stratification, thickness of overburden	–	–
Large scale tests, desirable prior to the elaboration of tender documents			Blasting test, compaction tests, grout test

Table 4.2. Exa[illegible] of field investigations for a rockfill dam with earth core at favourable geological c[illegible]nditions.

Item	Work to be done	Responsible reporter	Period of performance (months)	
			Beginning	End
1	Site mobilization with two rigs, equipment for borehole tests, workshop, site office, housing, all accessories	Geologist, engineer	Beginning of 1	End of 3
2	Preparation of access to 30 to 35 drill hole locations and 20 to 30 test trench locations	Geologist	Beginning of 4	End of 5
3	Geological mapping	Geologist	Beginning of 5	End of 7
4	Identification of material deposits in close vicinity	Geologist, engineer	Middle of 7	Middle of 9
5	200 m of core drilling in mapped area of quarries, no tests	Geologist, (engineer)	Beginning of 6 Drill rig A	End of 7 100 m per month
6	1400 m of core drilling in 20 to 25 boreholes with complete water pressure testing. The location of boreholes covers the area across the valley and about 300 m d/s and 300 m u/s of the dam	Geologist	Beginning of 6 Drill rig B Beginning of 16 Drill rig A	End of 19 80 m per month End of 19 80 m per month
7	400 m of core drilling in 6 selected boreholes with complete water pressure testing and test grouting	Geologist	Middle of 8 Drill rig A	Middle of 15 60 m per month
8	Excavation of 20 to 30 trenches in mapped borrow areas for core material, filter and concrete aggregates incl. necessary auger drilling and penetration testing	Geologist, engineer	Middle of 8	Middle of 12
9	Soil sampling from all material deposits	Geologist, engineer	Middle of 10	Middle of 13
10	Drafting of complete reports on items 3 through 9	Geologist, engineer	Beginning of 19	End of 20
11	Geodetical survey of all borehole and trench locations	Surveyor	Middle of 19	End of 20
12	Wrapping and shipping of rock and soil samples: – to a local laboratory, – to a laboratory abroad for special testing	Engineer	 Middle of 13 Middle of 13	 Middle of 14 Middle of 16
13	Laboratory testing and reporting: – local laboratory, – laboratory abroad	Engineer	 Beginning of 15 Beginning of 17	 End of 20 End of 2[illegible]
14	Period to cover delays and unforeseen works		Beginning of 21	End [illegible]

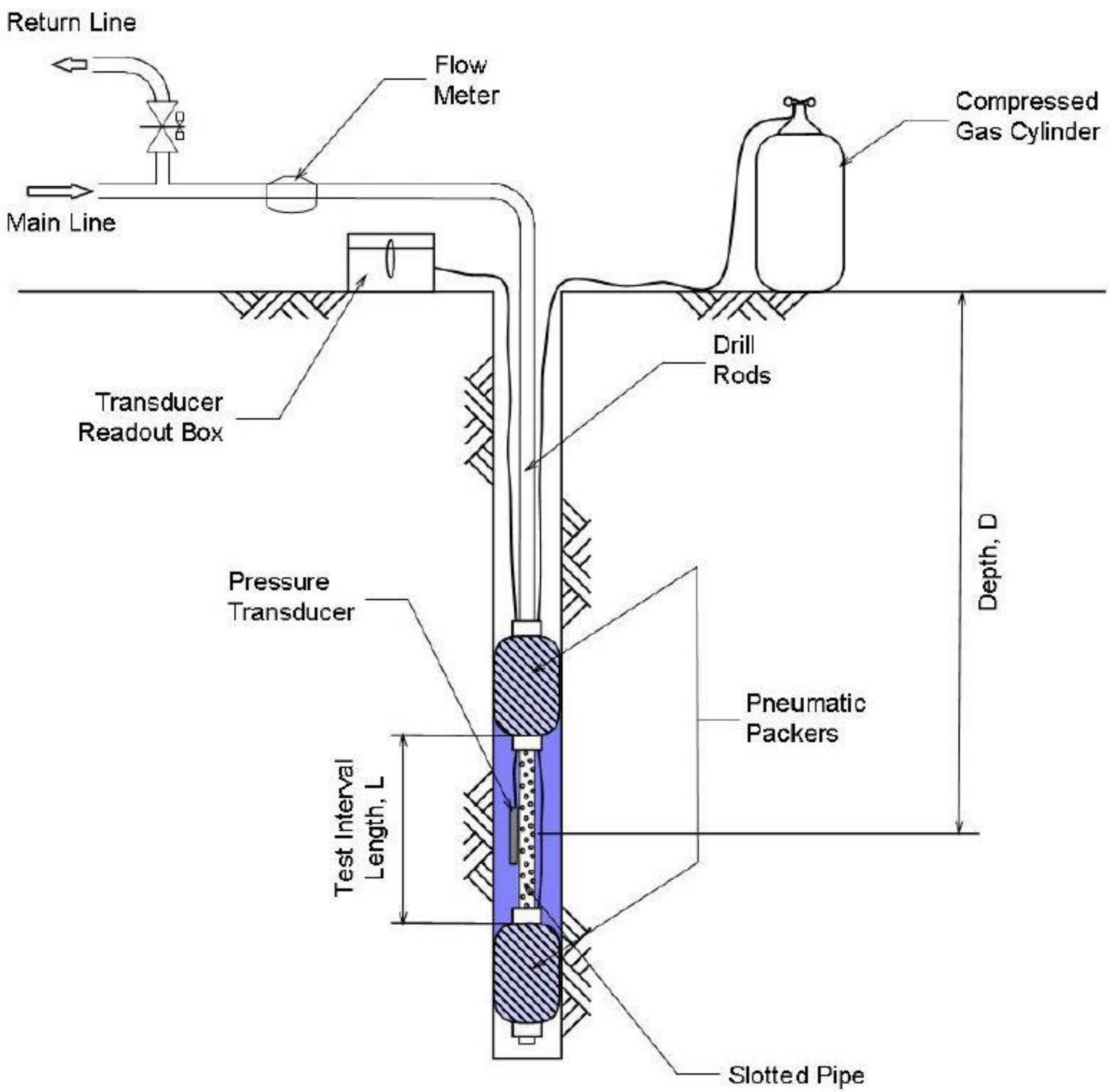
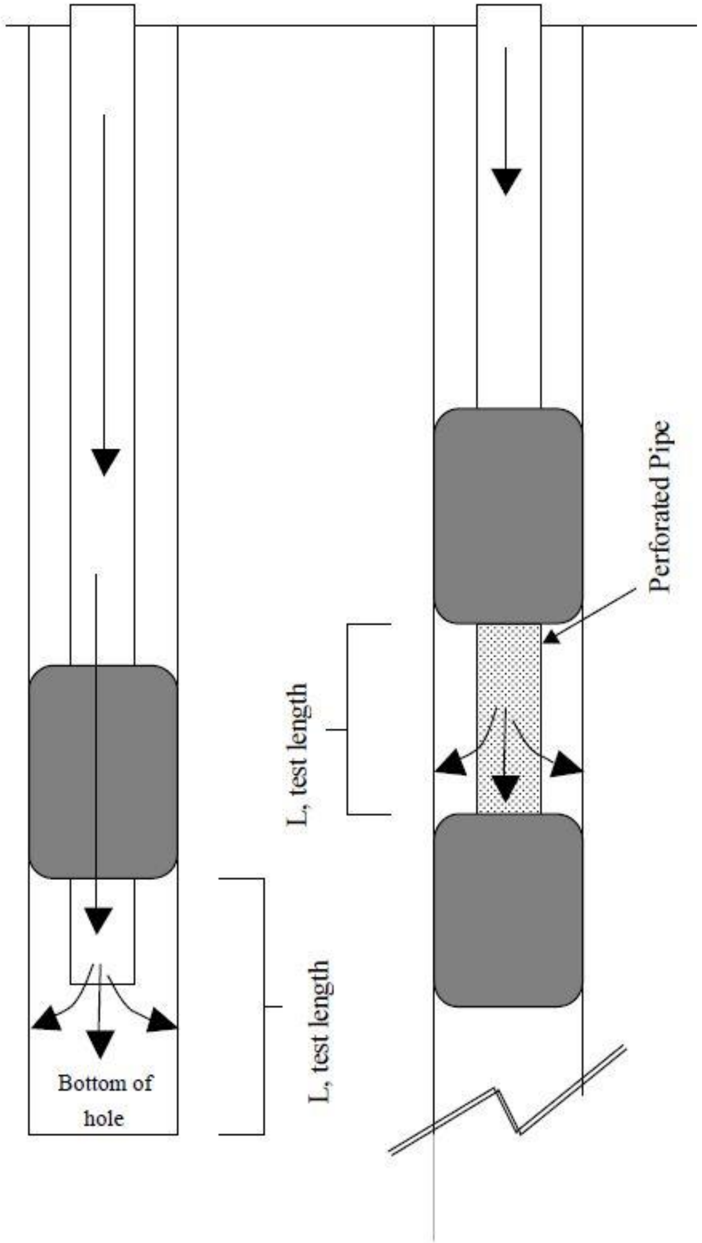


Figure 1. Lugeon test configuration



1A:Single Packer Test.
Open Borehole

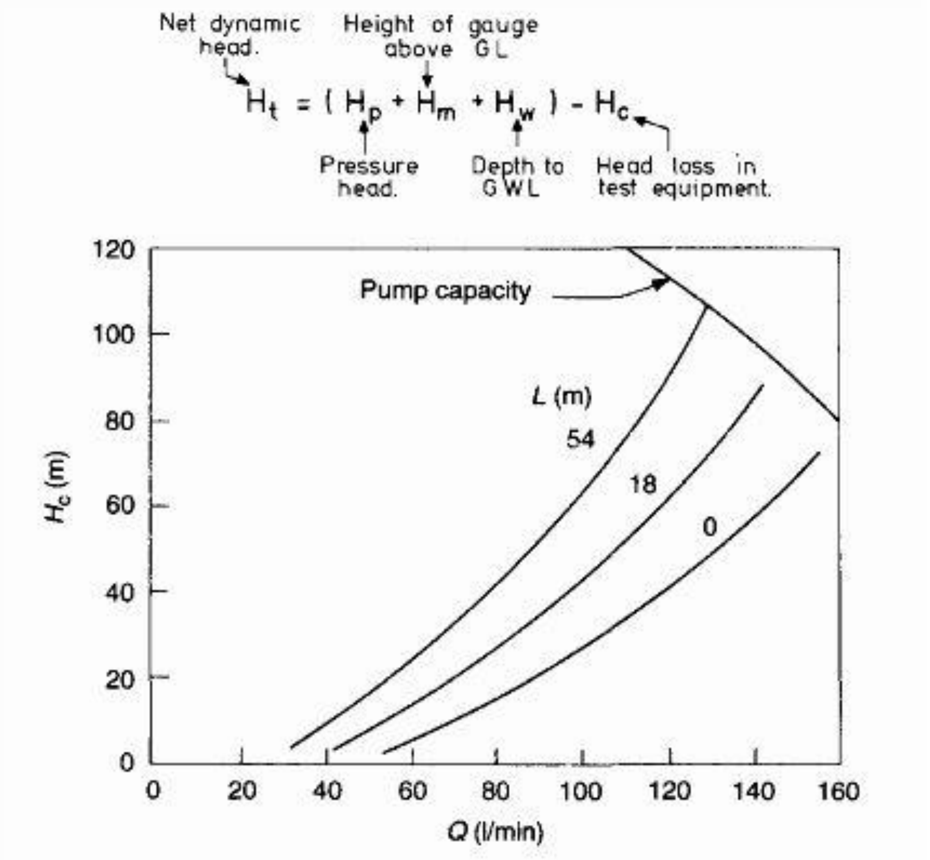
1B: Double Packer Test
Open Borehole

Table 1. Pressure magnitudes typically used for each test stage

Test Stage	Description	Pressure Step
1st	Low	$0.50 \cdot P_{MAX}$
2nd	Medium	$0.75 \cdot P_{MAX}$
3rd	Maximum (peak)	P_{MAX}
4th	Medium	$0.75 \cdot P_{MAX}$
5th	Low	$0.50 \cdot P_{MAX}$

Table 2. Condition of rock mass discontinuities associated with different Lugeon values

Lugeon Range	Classification	Hydraulic Conductivity Range (cm/sec)	Condition of Rock Mass Discontinuities	Reporting Precision (Lugeons)
<1	Very Low	$< 1 \times 10^{-5}$	Very tight	<1
1-5	Low	$1 \times 10^{-5} - 6 \times 10^{-5}$	Tight	± 0
5-15	Moderate	$6 \times 10^{-5} - 2 \times 10^{-4}$	Few partly open	± 1
15-50	Medium	$2 \times 10^{-4} - 6 \times 10^{-4}$	Some open	± 5
50-100	High	$6 \times 10^{-4} - 1 \times 10^{-3}$	Many open	± 10
>100	Very High	$> 1 \times 10^{-3}$	Open closely spaced or voids	>100



Calibration curves for packer test equipment with various rod lengths (Dick 197

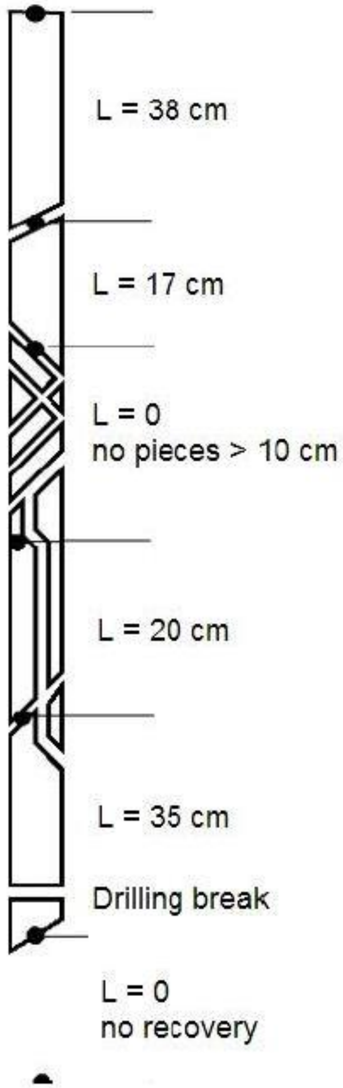
Table 4: Rock Mass Rating System (After Bieniawski 1989).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS					
Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Type of formation	P wave velocity (m/s)	S wave velocity (m/s)	Density (g/cm^3)	Density of constituent crystal (g/cm^3)
Scree, vegetal soil	300-700	100-300	1.7-2.4	-
Dry sands	400-1200	100-500	1.5-1.7	2.65 quartz
Wet sands	1500-2000	400-600	1.9-2.1	2.65 quartz
Saturated shales and clays	1100-2500	200-800	2.0-2.4	-
Marls	2000-3000	750-1500	2.1-2.6	-
Saturated shale and sand sections	1500-2200	500-750	2.1-2.4	-
Porous and saturated sandstones	2000-3500	800-1800	2.1-2.4	2.65 quartz
Limestones	3500-6000	2000-3300	2.4-2.7	2.71 calcite
Chalk	2300-2600	1100-1300	1.8-3.1	2.71 calcite
Salt	4500-5500	2500-3100	2.1-2.3	2.1 halite
Anhydrite	4000-5500	2200-3100	2.9-3.0	-
Dolomite	3500-6500	1900-3600	2.5-2.9	(Ca, Mg) CO_3 2.8-2.9
Granite	4500-6000	2500-3300	2.5-2.7	-
Basalt	5000-6000	2800-3400	2.7-3.1	-
Gneiss	4400-5200	2700-3200	2.5-2.7	-
Coal	2200-2700	1000-1400	1.3-1.8	-
Water	1450-1500	-	1.0	-
Ice	3400-3800	1700-1900	0.9	-
Oil	1200-1250	-	0.6-0.9	-

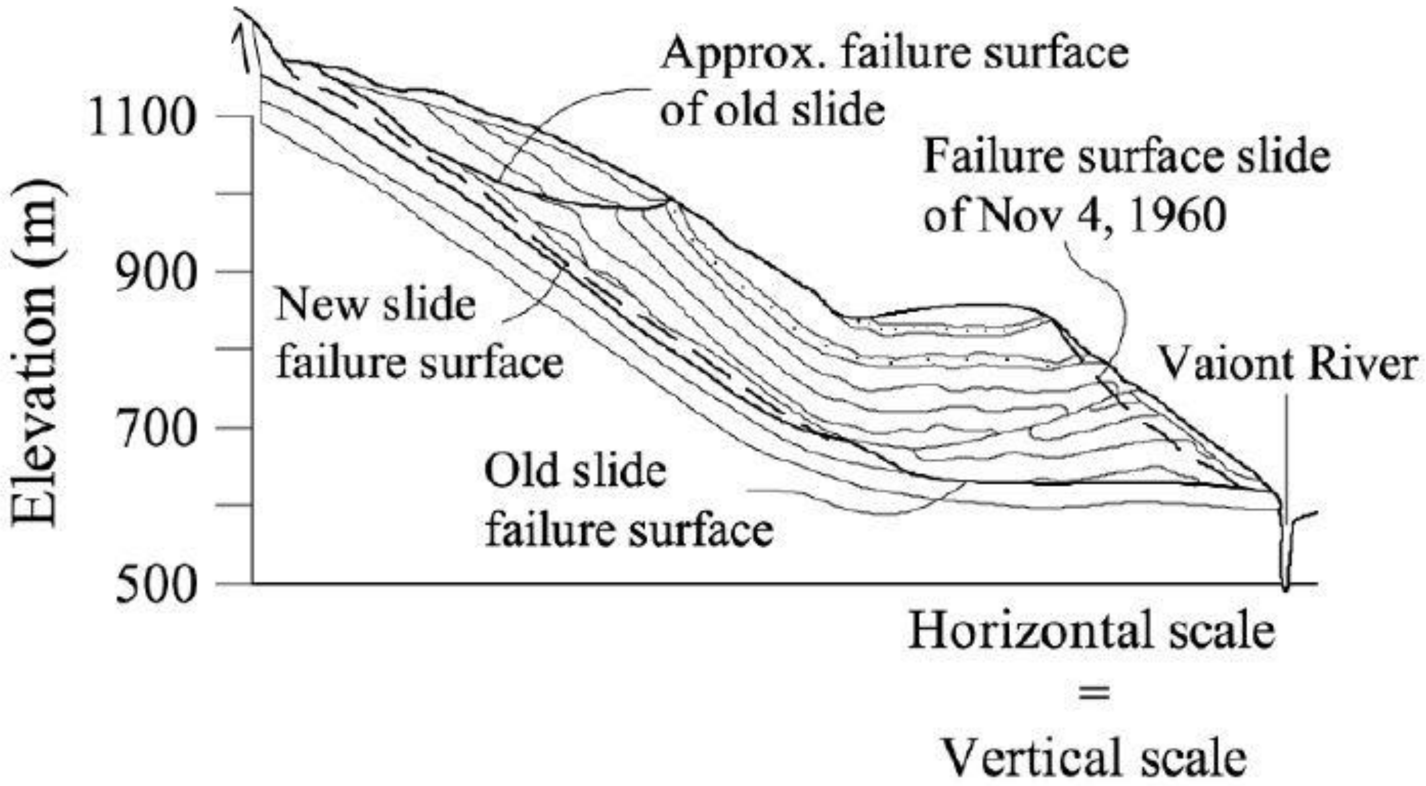


Total length of core run = 200 cms

$$RQD = \frac{\sum \text{Length of core pieces} > 10 \text{ cm length}}{\text{Total length of core run}} \times 100$$

$$RQD = \frac{38 + 17 + 20 +}{200} \times 100 = 55\ \%$$

Figure 1: Procedure for measurement and calculation of *RQD* (After Deere, 1989)


Approx. failure surface
of old slide
Failure surface slide
of Nov 4, 1960
1100
900
700
500
Elevation (m)
New slide
failure surface
Vaiont River
Old slide
failure surface
Horizontal scale
=
Vertical scale

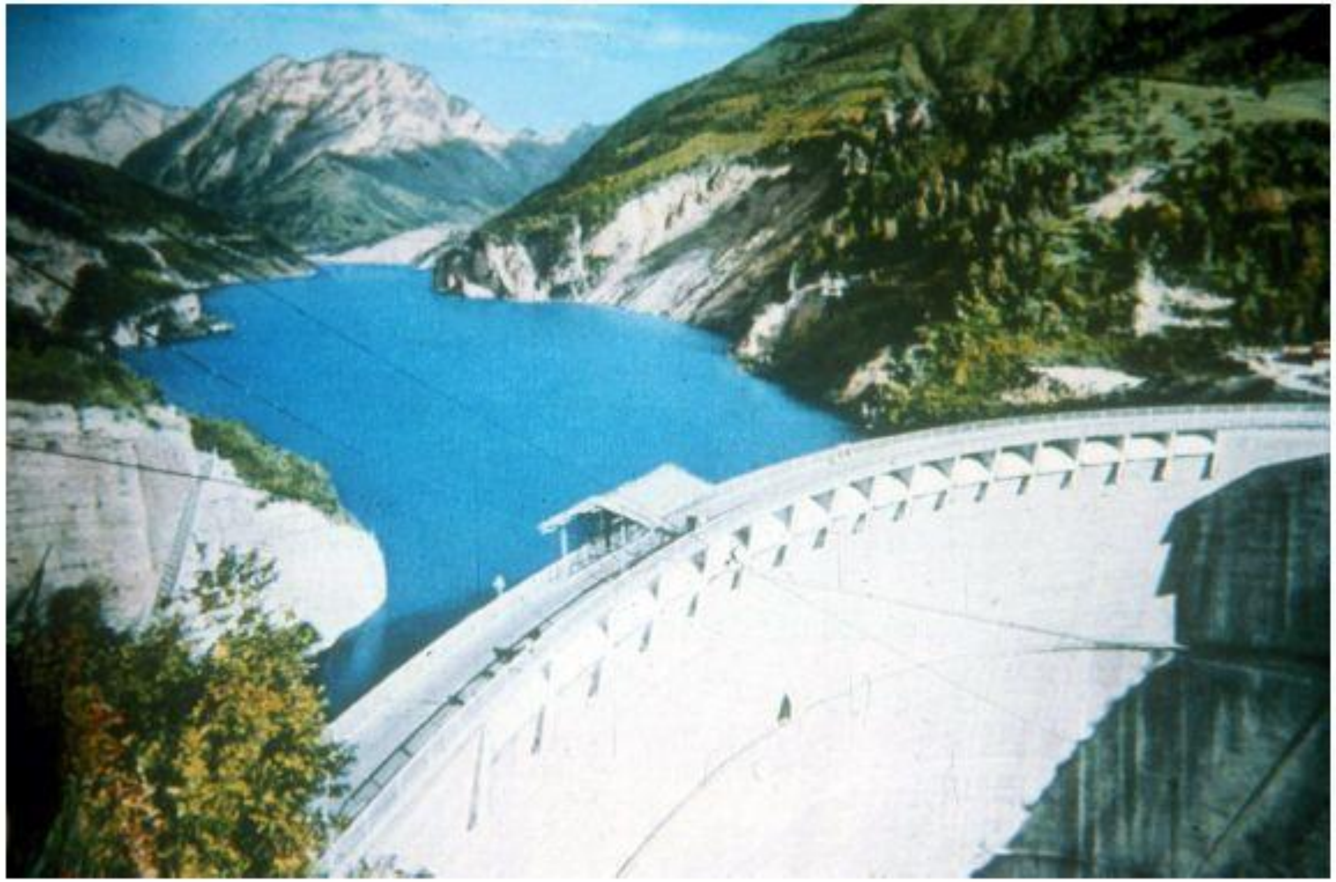


Figure 2a: The Vajont dam during impounding of the reservoir. In the middle distance, in the centre of the picture, is Mount Toc with the unstable slope visible as a white scar on the mountain side above the waterline.

Figure 2c: The town of Longarone, located downstream of the Vajont dam, before the Mount Toc failure in October 1963.

Figure 2b: During the filling of the Vajont reservoir the toe of the slope on Mount Toc was submerged and this precipitated a slide. The mound of debris from the slide is visible in the central part of the photograph. The very rapid descent of the slide material displaced the water in the reservoir causing a 100 m high wave to overtop the dam wall. The dam itself, visible in the foreground, was largely undamaged.

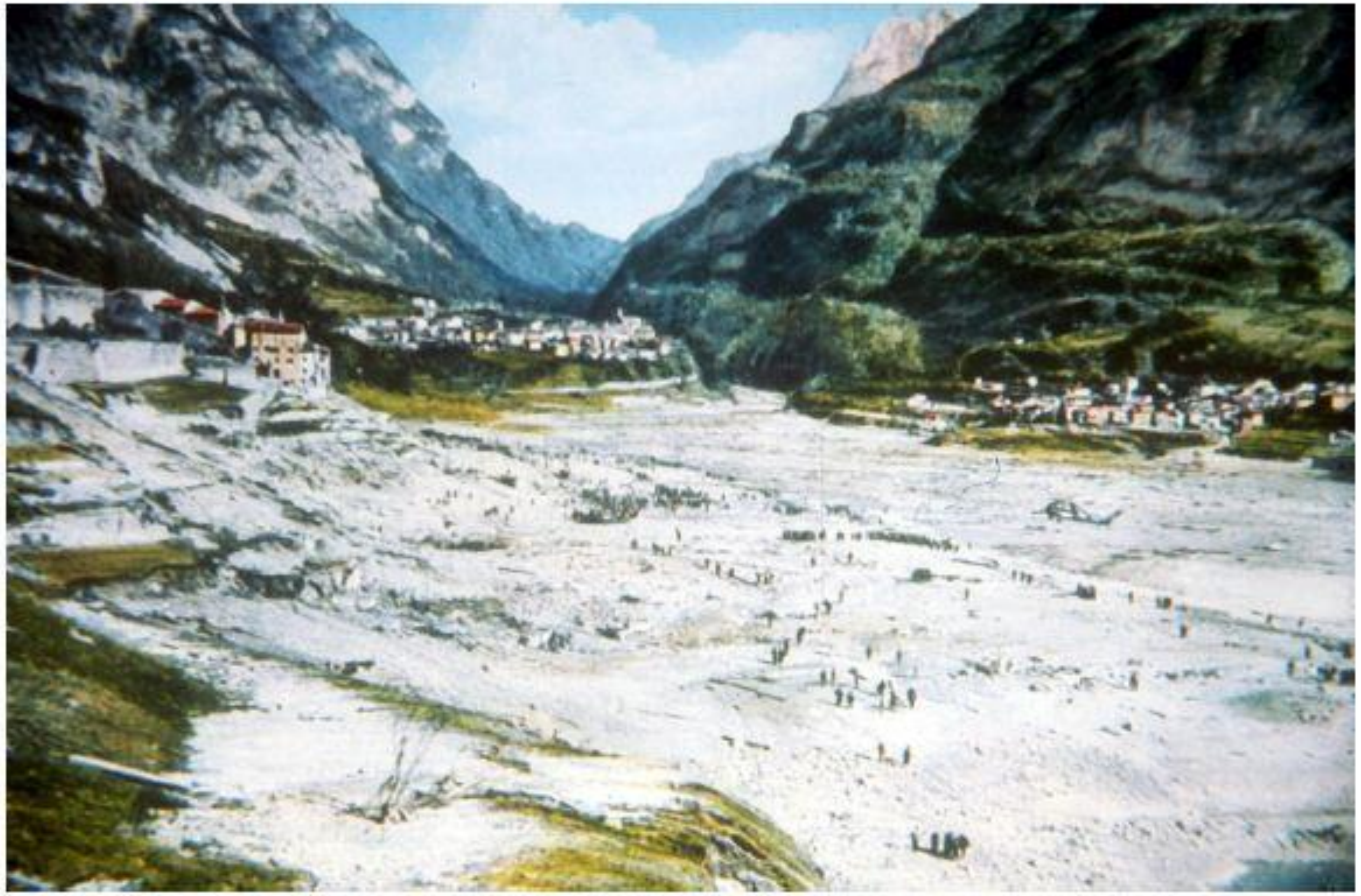
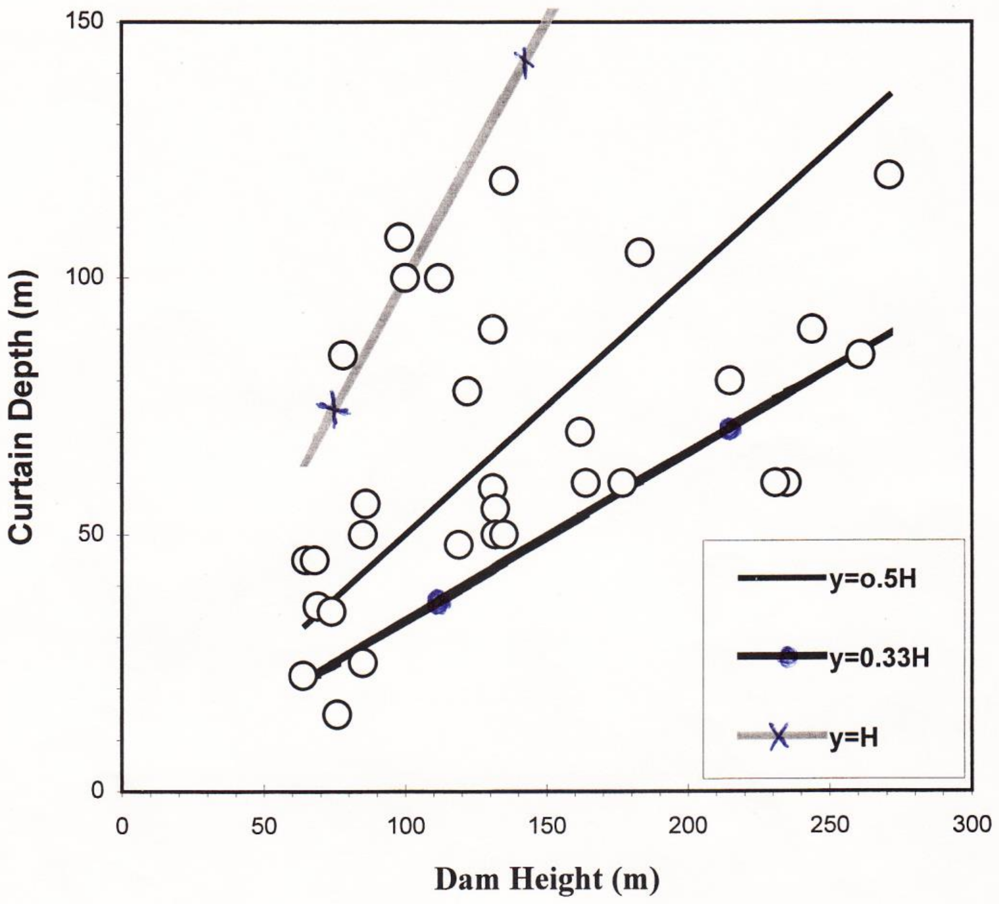
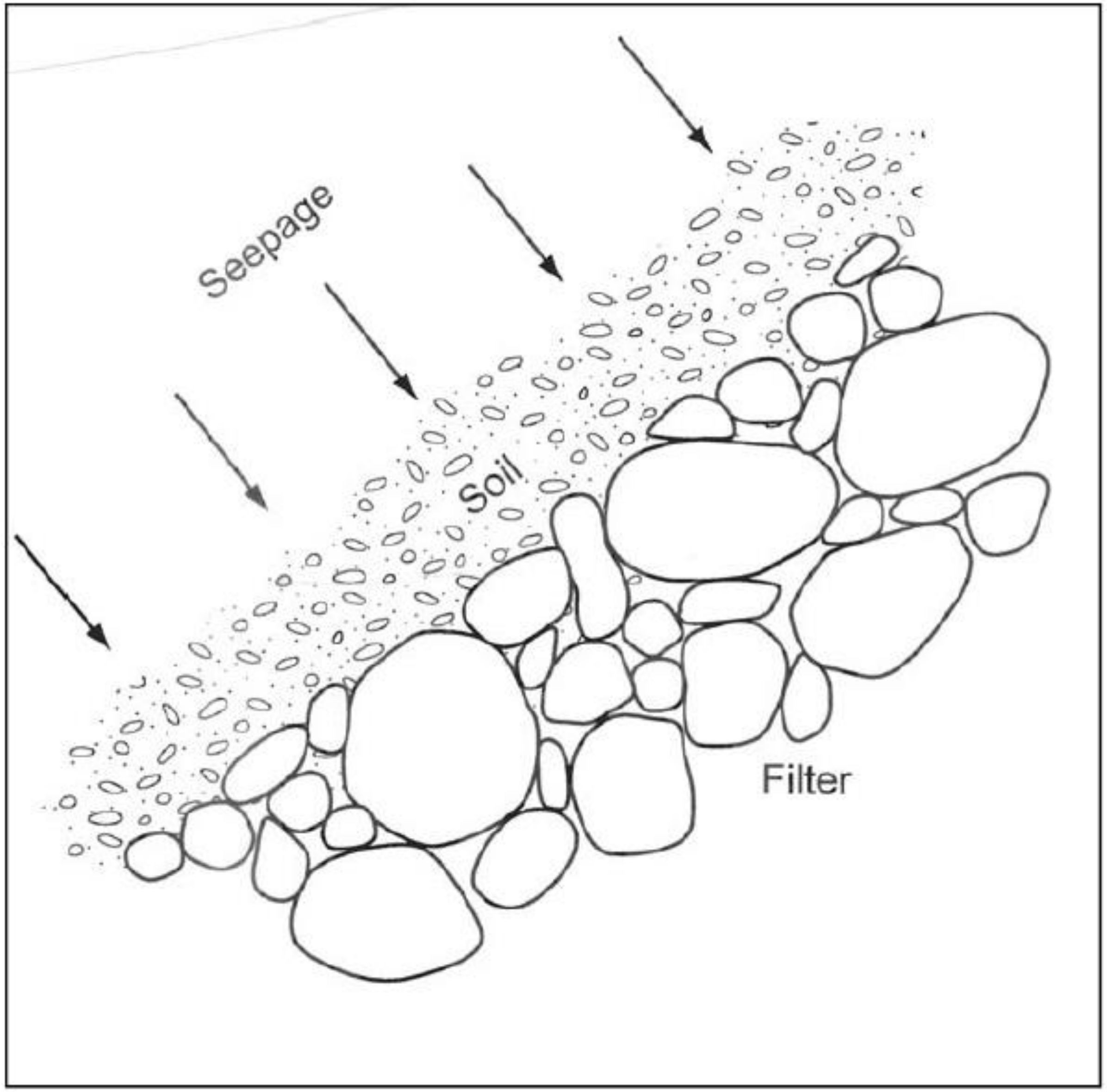
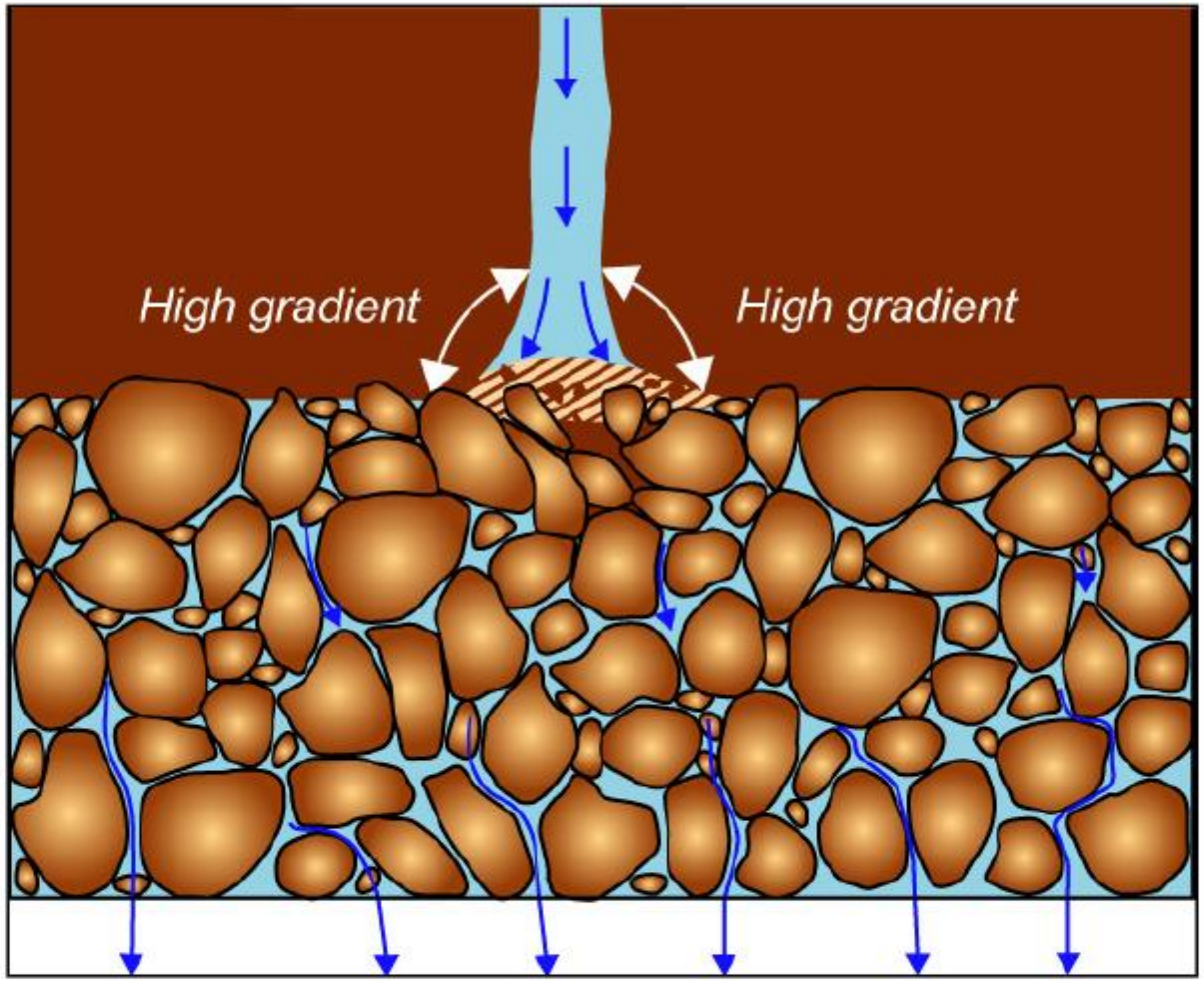


Figure 2d: The remains of the town of Longarone after the flood caused by the overtopping of the Vajont dam as a result of the Mount Toc failure. More than 2000 persons were killed in this flood.


150
100
50
0
Curtain Depth (m)
0
50
100
150
200
250
300
Dam Height (m)
y=o.5H
y=0.33H
y=H


Seepage
Soil
Filter


High gradient
High gradient