INSTITUTE OF AERONAUTICAL ENGINEERING

(Autonomous)

Dundigal, Hyderabad - 500 043

Course : Foundation Engineering (A60126)

III Year II Semester

Course Instructor : **Y Ravi Kumar**

Assistant Professor,

Department of Civil Engineering.

COURSE OVERVIEW:

It is the branch of civil engineering concerned with the design and construction of foundations, slopes, retaining walls, embankments, tunnels, levees, wharves, landfills and similar facilities; and with the engineering characterization and behaviour of the ground and its constituent materials

1. Plays a key role in all civil engineering projects built on or in the ground.
2. Is vital for the assessment of natural hazards such as earthquakes, liquefaction, sinkholes, rock falls and landslides.

COURSE OBJECTIVES:

The course should enable the students to:

I. Identify the methods of soil exploration, different field tests, planning and preparation of soil investigation programme.
II. understand and analyze finite and infinite earth slopes to analyze the stability of slopes of earth dams under different conditions
III. Know earth pressure theory Rankine's theory of earth pressure Coulomb's earth pressure theory and Culmann's graphical method.
IV. Enhance the ability of students in understanding the types, choice of foundation, location of depth, and safe bearing capacity when considering shallow foundation.
V. Know the Indian standard methods for calculating safe bearing pressure based on N value, allowable bearing pressure and safe bearing capacity.
VI. Analysis of pile foundation, types of piles, load carrying capacity of piles based on static pile formulae and dynamic pile formulae
VII. Understand the systems of well foundations, components of wells, functions and design criteria

COURSE OUTCOMES:

After completing this course the student must demonstrate the knowledge and ability to:

1. Analyze the of need and methods of soil exploration
2. Ability to learn the field test and soil investigation
3. Apply knowledge for stability of slopes of earth dams under different conditions.
4. Students should be able to understanding earth pressure theories and design of retaining walls
5. Ability to learn the theory of shallow foundation
6. Students should be able to understanding the concept of allowable bearing pressure, safe bearing capacity
7. Ability to understanding the concept of pile foundation and of well foundation

SYLLABUS:

Unit –I:

SOIL EXPLORATION: Need and Methods of soil exploration- Boring and Sampling methods- Penetration tests - Plate load test- Pressure meter- planning of Programme and preparation of soil investigation report.

Unit –II:

SLOPE STABILITY: Infinite and finite earth slopes- types of failures- factor of safety of infinite slopes- stability analysis by Swedish arc method, standard method of slices, Bishop's Simplified method- Taylor's Stability Number- Stability of slopes of earth dams under different conditions.

Unit –III:

EARTH PRESSURE THEORIES: Rankine's theory of earth pressure- earth pressures in layered soils- Coulomb's earth pressure theory- Culmann's graphical method.

SYLLABUS:

RETAININGWALLS: Types of retaining walls- stability of retaining walls against overturning, sliding, bearing capacity and drainage from backfill.

Unit –IV:

SHALLOW FOUNDATIONS-Strength Criteria- Types, choice of foundation, Location of depth-, Safe Bearing Capacity, Terzaghi, Meyerhof, Skempton and IS Methods.

SHALLOW FOUNDATIONS-Settlement criteria: Safe bearing pressure based on N- value- allowable bearing pressure, safe bearing capacity, plate load test, allowable settlements of structures.

PILE FOUNDATION: Types of piles- Load carrying capacity of piles based on static pile formulae in Dynamic pile formulae, Pile load tests, Load carrying capacity of pile groups in sands and clays, Settlement of pile groups.

SYLLABUS:

Unit –V:

WELL FOUNDATIONS: Types- Different shapes of wells- Components of wells- Sinking of well-Tilts and shifts.

BOOKS :

Textbooks:

1. Principles of foundation Engineering- Cengage Learning by Das, B.M., (2012)
2. Basic and Applied Soil Mechanics by Gopal Ranjan & ASR Rao, New Age International

Pvt. Ltd, (2004).

3. Geotechnical Engineering: Principles and practices of soils mechanics and foundation

Engineering by VNS Murthy, Taylor & Francis Group.

Reference Books:

1. Analysis and Design of Substructures – Swami Saran, Oxford and IBH Publishing company Pvt Ltd 1998.
2. Geotechnical Engineering by S.K. Gulhati & Manoj Datta – Tata Mc. Graw Hill publishing company New Delhi. 2005.
3. Teng, W.C – Foundation Design, Prentice Hall, New Jersy.
4. Bowles, J.E., (1988) Foundation Analysis and Design- 4th Edition, McGraw-Hill

Publishing Company, Newyork.

EVALUATION SCHEME:

<u>Weightage:</u>

- MidTerm examination =20M
- Assignments and class notes = 5M
- End Semester Exams =75M

Unit-I : Soil Exploration

Need and methods of soil exploration, boring and sampling methods, penetration tests, plate load test, pressure meter, planning of programme and preparation of soil investigation report.

Site Investigation

- Site investigations and testing are required in choosing the method for the determination of the ultimate capacity or for the estimation of settlement.
- In determining the said factor of safety against failure, due consideration shall be given to the form and depth of the foundation, loading characteristics, the *general geological conditions of the ground and its surrounding including the presence of dissolution features, jointing conditions and any other relevant characteristics for rock.*

General Design

- Foundations of any building or structure shall be designed and constructed to withstand safely all the dead, imposed and wind loads without impairing the stability or inducing excessive movement to the building or of any other building, street, land, slope or services.

- The allowable capacity of the soil/rock under working loads where any foundation is founded shall be the lesser of :
    - the ultimate capacity for bearing, bond or friction with an adequate factor of safety against failure; or
    - the value in relation to bearing, bond or friction such that the maximum deformation or movement induced to the foundation under working loads can be tolerated by the building, any other building, structure, land, street and services.

- The allowable capacity may be increased by 25% when such increase is solely due to wind effects.

Rational Design method

- Rational design method for calculating the ultimate capacity should be based on sound engineering approach and should include :
    - the reasonable interpretation of the results of site investigation;
    - the assessment of test results obtained in situ or from samples in the laboratory and
    - an analysis based on the laws of physics and recognized engineering principles taking into account the ground conditions and foundation geometry, or an established empirical method proven with adequate correlation.

- Normally, the allowable capacity is estimated by applying a factor of safety of 3 to the calculated ultimate bearing capacity. However, other factors of safety may be adopted having regard to the nature of the soil or rock, its variability over the site and the reliability of the design method.

In situ testing method

The allowable capacity for soils and rocks may also be estimated by appropriate load testing of the foundation on site. The following should be considered when using this method :

- the variation at founding conditions between the location of the testing foundation and locations of the actual foundations
- the duration of load application in the test as compared to the working life of the foundation;and
- the scale effect of the test relative to the full size of the foundation.

Soils Investigation

- Required to evaluate an area for the construction of a project or evaluate local material as a construction material
- Soil Investigation
  - Field Sampling and Testing
  - Laboratory Analysis
  - Report preparation
- Planning and evaluation of field work are aided by knowledge of the mechanics of soil deposit's formation

Soils Investigation

- Soil grains are the result of weathering of bedrock
  - physical weathering
    - granular soil types (gravel, sand, silt)
  - chemical weathering
    - clays
- Soil deposits
  - residual- product of weathering the original bedrock
  - transported- moved from their place of origin

Soils Investigation

- Transportation agents
  - Rivers and streams
    - gravel sand silt deposited as a fn (water velocity)
  - Lakes
    - clays and silts settling out
  - Wind
    - sand dunes and loess deposits (silt particles)
  - Glaciers
    - movement in North America eroded, transported, and deposited soils in many types of formations

Soils Investigation

- Glacier soil deposits
  - tills (mixture of gravel sand silt clay)
    - material that has been shoved forward or picked up from an advancing glacier
    - this material is deposited when a glacier stops or retreats as it melts
  - fluvial deposits associated with glaciers
    - clays and varved clays from glacier lakes
    - marine clays deposited from salt water
    - sorted gravel, sand and silt from glacier streams

Soils Investigation

Table 2–1
MAIN TYPES OF SOIL DEPOSITS

Type	*Formation*	*Characteristic*	*Material*	*Significance*
Glacial				
End or lateral moraine	At front or sides of glacier	Rough, rolling ground	Till-mixed	Hard, fairly impervious
Ground moraine	Under glacier	Rolling	Till-mixed	Hard, fairly impervious
Drumlin	Under glacier	Mounds about 1 km × 1/2 km (1/2 mi × 1/4 mi)	Till-mixed	Hard, fairly impervious
Kame moraine	Water flowing over or out of glacier	Rough, rolling ground	Sand, gravel, silt	Aggregates source
Esker	Streams flowing under glacier	Long ridges	Sand, gravel	Aggregates source
Outwash or spillway	Streams flowing away from glaciers	Fan-shaped delta	Sand, gravel, silt	Aggregates source

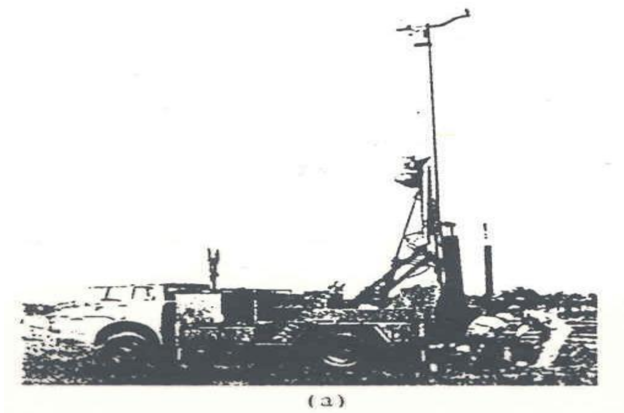
Lake bottom	Soil grains settling out	Flat	Clays, varved clays	Poor foundation soil, soft and compres- sible
Beach	Shores, bars, etc., of lakes	Beach-type, sorted de- posits	Sand, gravel	Aggregates source
Marine	Soil grains settling out in salt water	Flat	Marine days	Very soft, very sen- sitive and com- pressible
Post-glacial Alluvial	Stream	Existing stream . valley	Sand, gravel, silt	May be aggregates or pockets of soft, variable material
Dune,loess	Wind	Small hills	Sand, silt	Uniform particles
Beach	Shores of lakes, oceans	Beach, well- sorted	Sand, gravel, uniform	,Aggregates source
Organi c	Low.poorly drained areas	Marshes, muskegs	Peat, muskeg, muck	Very weak, ex- tremely high compressibilit y

Soils Investigation

- Field Investigation Techniques
  - determine bearing capacity for foundations
  - determine water resources
  - find aggregate deposits (road construction)
  - estimate infiltration and seepage rates
  - assess land use capabilities
- Information required
  - depth, thickness, properties of each soil layer
  - location of groundwater table
  - depth to bedrock

Soils Investigation-Drill Rig

(a)



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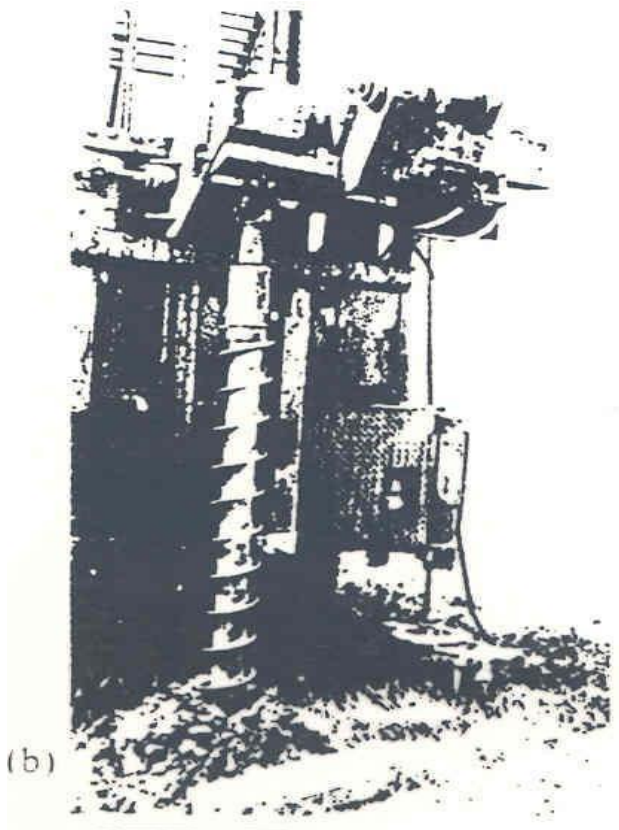
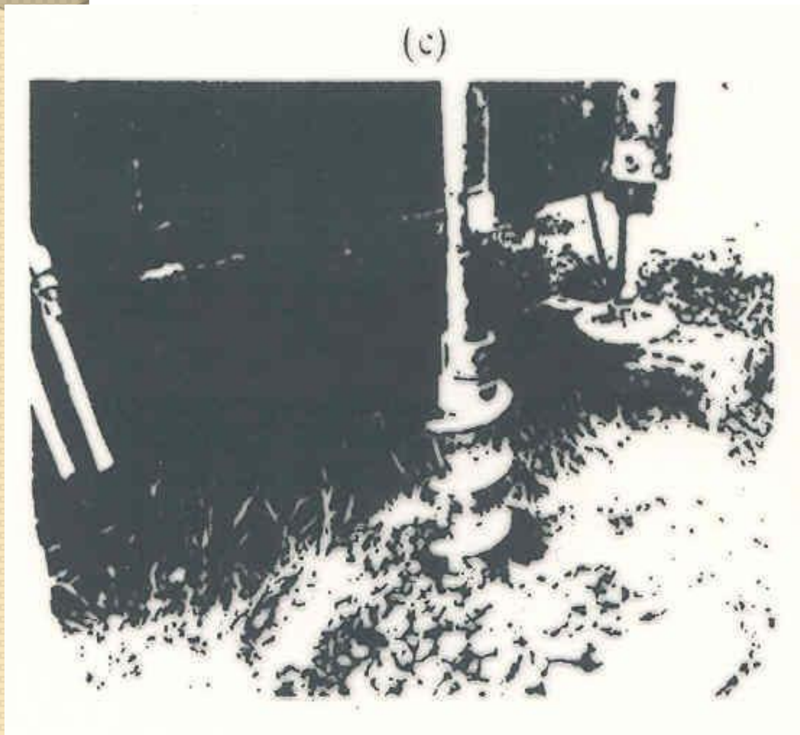


a) drill rig with the standard penetration hammer attached to the cable

b) augering to open a test hole

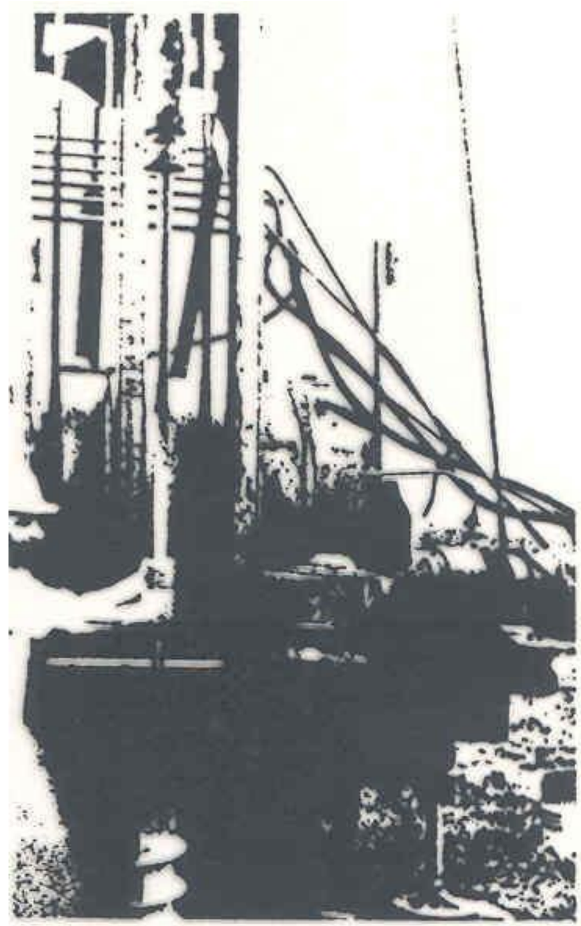
Soils Investigation-Drill Rig



c) split-spoon sampler attached to drill rods have been lowered to the bottom of the test hole through the hollow stem of this auger and chalk marks have been placed on the rod at 15 cm (6 in) intervals

Soils Investigation-Drill Rig

d) drill rod and spoon being driven into soil at the bottom of the test hole



Soils Investigation

- Preliminary Investigation
  - geological and agricultural maps
    - types of soils or geological formations
  - aerial photographs
    - drainage patterns and color and tone can indicate what type of soil
  - area reconnaissance
    - other structures performance in the area
    - wells can indicate groundwater levels

Soils Investigation

- Subsurface Investigation
  - Geophysical methods
    - seismic or electrical-variations in the speed of sound waves or electrical resistivity of soil formations
  - Test pits or trenches
    - shallow depths only
  - Hand Augers
    - shallow depths only
  - Boring test holes and sampling with drill rigs
    - principal method for detailed soil investigations

Soils Investigation

- Boring Test Holes
  - continuous flight auger
  - samples are taken by sampling tools inserted in the test hole when the auger is removed at certain depths
  - if the hole does not stay open, a pipe can be driven into the ground and the hole augured from inside the pipe or a hollow stem auger is used through which samples are taken

Soils Investigation

- Undisturbed Samples
  - soil structure of the sample is as close as possible to the structure of the soil in the field
  - Thin-wall sampler (Shelby Tube) clays/silts
    - sample taken by pushing the tube into soil and sealed to prevent moisture loss
- Disturbed Samples
  - auger samples- taken at surface depth unknown
  - split spoon sampler-depth known

Soils Investigation

- Field Testing
  - Standard penetration test
    - most common strength test conducted in the field
    - number of blows (N value) required to drive sampler into the soil layer 30 cm by a standard mass (63.5 Kg) dropped a specific distance (75 cm).
    - Used on all soils except gravel
    - a disturbed sample can also be taken from the spoon
  - Vane
    - measures cohesiveness in clays
    - shoved into the soil and a torque applied

Soils Investigation

Table 2–3
FIELD TERMS TO DESCRIBE SOIL CONDITIONS, BASED ON THE STANDARD PENETRATION TEST

N = Blows/30 cm (1 ft)	*Relative Condition of Sand and Silt Soils*
0–4	Very loose
5–10	Loose
11–30	Medium dense
31–50	Dense
more than 50	Very dense
	Consistency of Clays
0–1	Very soft
2–4	Soft
5–8	Firm
9–15	Stiff
16–30	Very stiff
more than 30	Hard

Soils Investigation

- Field testing
  - Cone
    - cone is driven through the soil and the number of blows for each foot recorded.
    - Indicates the depth of fill or the depth to layer changes
  - Pressuremeter test
    - probe consisting of three cell, upper and lower prevents middle cell from expanding vertically
    - pressure from a gas is applied to the probe and any volume change is measured by a volumeter
    - settlement prediction and shear strength

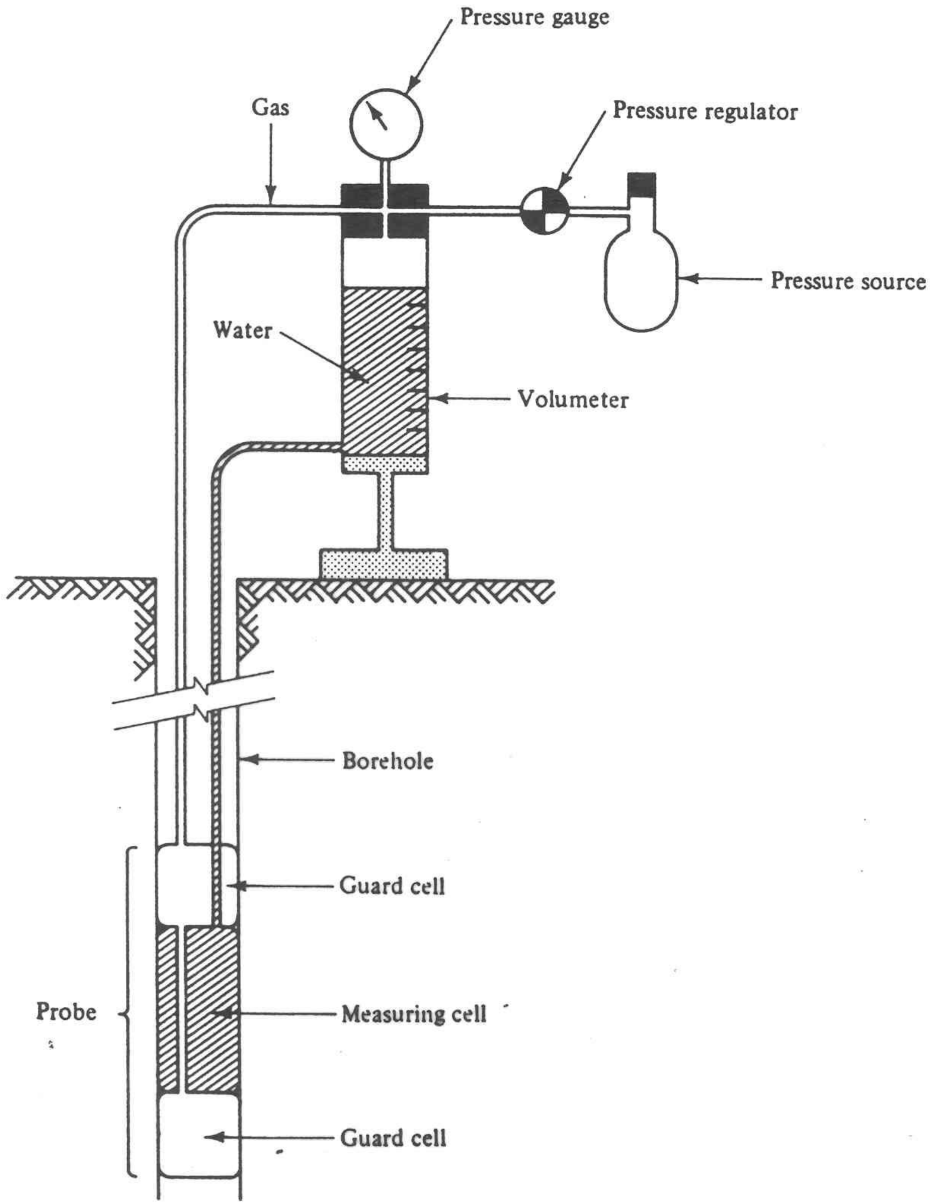


Figure 2–5 The pressuremeter.

Soils Investigation

- Water table elevation determination
  - measure down the hole to the water table as water fills the hole
    - easily done for granular soils
  - piezometer for fine grained clay soils

Soils Investigation

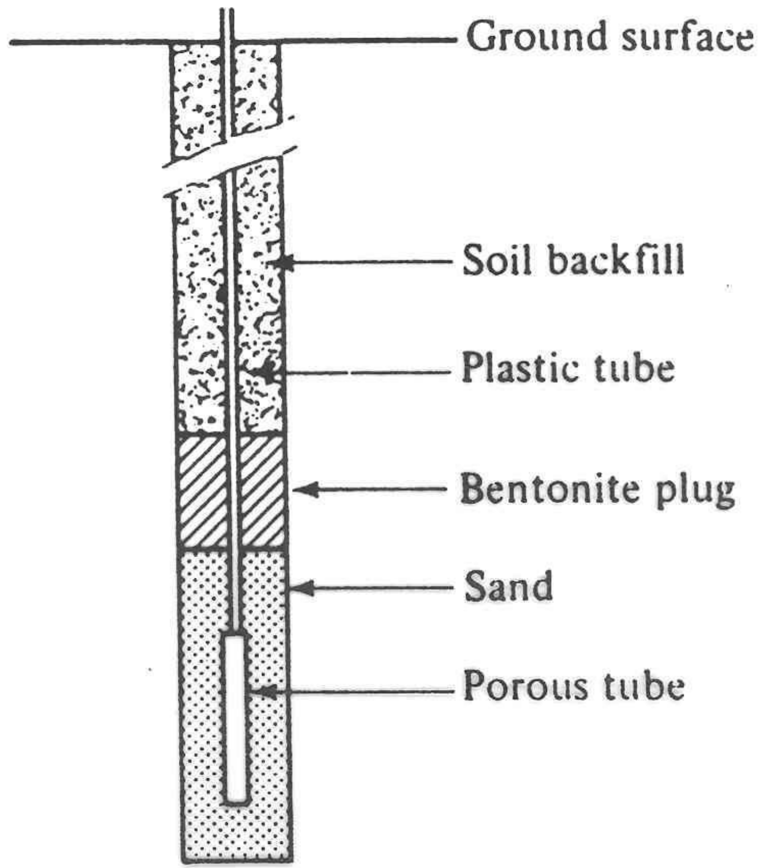


Figure 2–6 Piezometer installation.

Soils Investigation

- Field log of the test hole
  - sample number, depth and type
  - field tests, depth and results
  - depth to layer changes
  - field soil description
    - type of soil grains
    - moisture conditions
    - consistency or density
    - seams and stratification
    - other distinguishing features

Test Hole No. __1 __

Date ___'f6 ? 161_1 __ ?

Drilling Method A_u_.3,_(,_r_

Depth to water:

immediate 1.'L l'k

24 hours 1.0 l'k

Depth (m)	Soil Description	Sample No.	Sample Type	Sample Depth (m)	SPT Value "N"
0	\ opsoil, s0- 1 1.oly < 0-0.3 l'k				
0.3 - 0.5	l.ees« brow lkOisf 11. s0- (O 3/11) /11.(,l",<) 11.ol	1	Au1(,r	O g	
1.0 1.'L	Y(.11.S(. siliy s0- 11.ol 1.2--1g ,.,,_	L	$plii SOO OH.	1.'l-16	U1'L 'LO
1.5 1g - 2.0	-rur, cl0-1(,1, wii/... /...Q-ro(<1.i-'L ,.,,_ g SOlk(, sili, tkoisi),	3	Sp/if spoo1 1.	'L.1-'L.5	15"I 1g 33
2.5 'L.i - 3.0	Cl0-y, so!i W(,i ('Li-3.6 l'k)	"	r.5/...(,l by	3.0-3.6	
3.5 3.6	E o! i(.5"! /...ol e,				

Figure 2-7 Typical field notes.

Soils Investigation

- Laboratory Testing
  - representative samples of each soil type found at the site
  - types of testing done depends on soil type, cohesive or granular, and if the sample is disturbed or undisturbed

Soils Investigation

Table 2–4
LABORATORY TESTS RELATED TO A SOILS INVESTIGATION

Test	*Sample Required*		*Soils*	
	Disturbed or Undisturbed	*Undisturbed*	*Cohesive*	*Granular*
Moisture content	X		X	X
Grain size	X		X	X
Atterberg Limits	X		X	
Relative density (specific gravity)	X		X	X
Density (unit weight)		X	X	X
Unconfined compression		X	X	
Triaxial compression		X	X	X
Direct shear		X	X	X
Consolidation		X	X	
Vane shear		X	X	
Permeability		X		X

2–3.2 Approximate values for soil strength may be obtained from simple field tests, as indicated in Table 2–5.

Soils Investigation

- Soils Report
  - includes a summary of the test program
  - general description of soil conditions
  - detailed analysis of each soil type found
  - recommendations for the design
  - copies of test hole logs and a soil profile

Gravel	Sand	Silt	Clay	Organic	Till
Sandy top soil	Gravelly sand	Silty sand	Sandy silt	Silty clay	Sandy till

S

Z

Water Table

Bed Rock

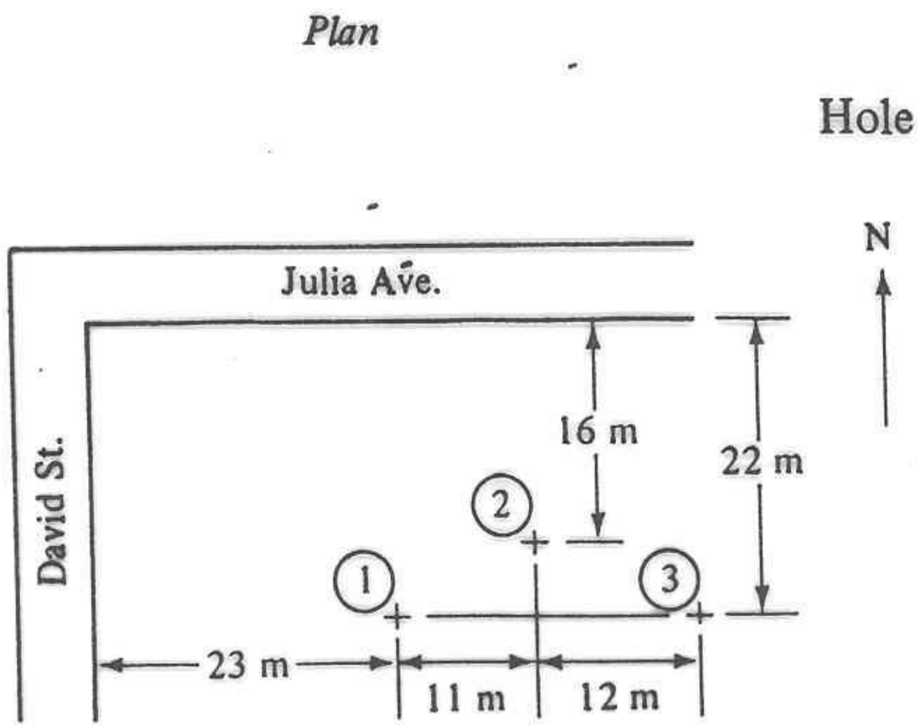
Type o] Sample		*Soil Tests*	
S.S.	- Split Spoon	F.V. -	Field Vane
S.T.	- Shelby Tube	L.V. -	Lab Vane
A.S.	- Auger Sample	Qu -	Unconfined Compression
W.S.	- Washed Sample	-	Undrained (quick) Triaxial
R.C.	- Rock Core	QqC	- Dynamic Cone {Blows per foot)
V.S.	- Vane Shear Sample		

Figure 2-9 Typical test-hole symbols and abbreviations.

Soils Investigation

Example 2–1 Following are results of a soils investigation:

1. TEST-HOLE LOCATIONS



Elevations

Hole No.	1	575.5 m
	2	574.7 m
	3	576.2 m

Vane shear tests were conducted in holes 1 and 3:

Hole	*Depth (m)*	*Cohesion*
1	1.5	30 kPa (600 lb/ft")
3	1.7	33 kPa (660 lb/fr')

Water levels one day after the holes are drilled:

Hole No. 1	0.8 m
Hole No. 2	0.3 m
Hole No. 3	1.5 m

3. LABORATORY TESTS

Sample No.	w	w_L	w_p	*ShearStr�h kPa (lb!) (Unconfined Compression Test)*
1	21			
2	47	53	21	
3	11			
4	9			
5	26			
6	58	55	20	
7	40	51	26	42 (850)
8	15			
9	11			
10	16			
11	41	58	29	65 (1300)
12	18			
13	12			

4. TEST-HOLE LOG (Figure 2-10)
5. SOIL PROFILE (Figure 2-11)

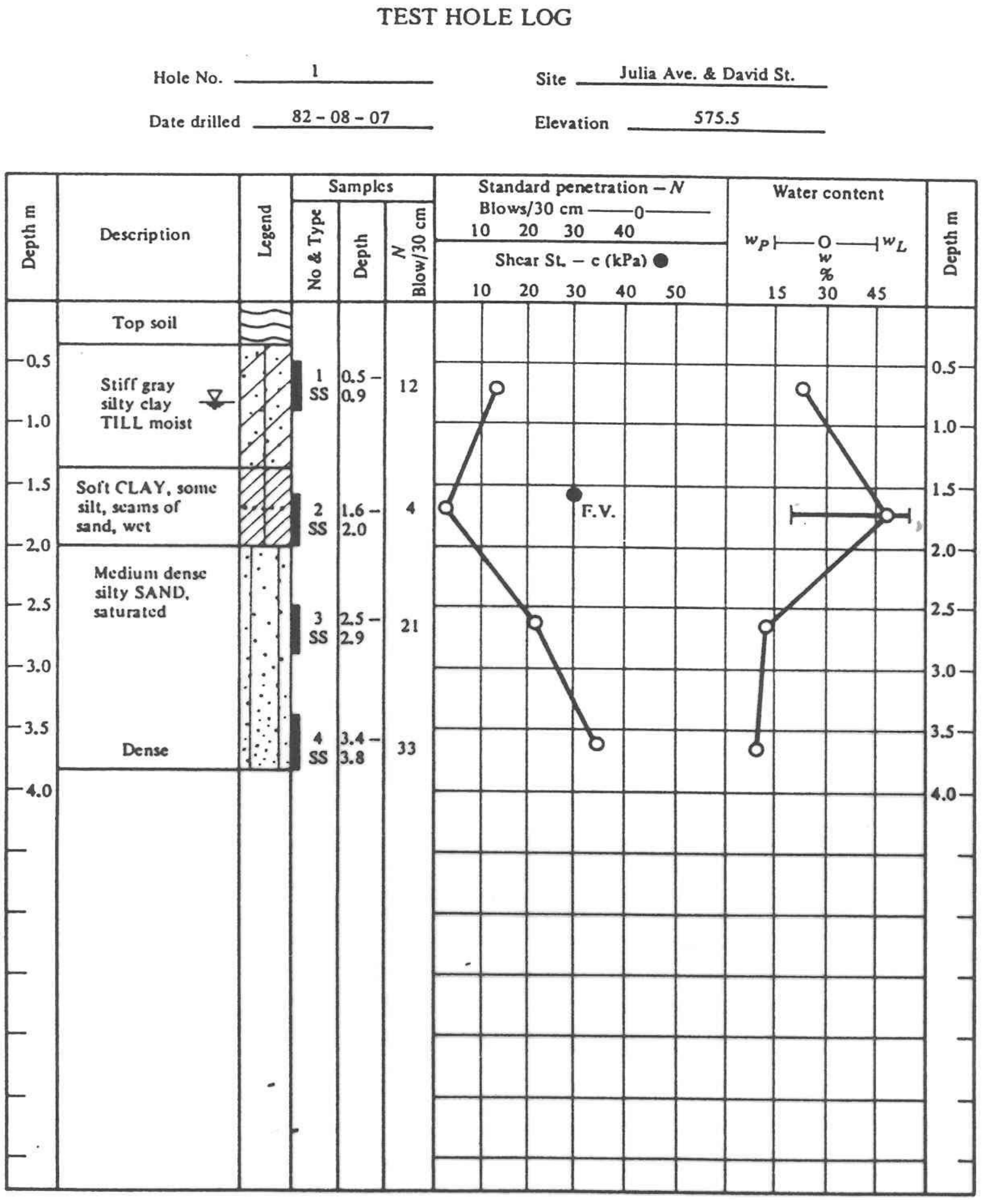


Figure 2–10 Test-hole log.

576

575
12

Gray, silty clay till

41

574
4

18

573

21 11

40 12

572

Silty sand

33

571

60 11

570

569

Figure 2-11 $oil **profile.**

Estimation of Settlement

Prediction of settlement is an important part of foundation design to ensure the future stability and serviceability of the structure supported by the foundation. The prediction of settlement should be :

- based on the results of a proper site investigation and appropriate laboratory or field tests identifying the conditions of the groundwater and the ground that contribute to the settlement of the foundation;
- based on the principles of mechanics or established empirical methods proven with adequate correlation;and

Site Investigation

- **"A comprehensive accumulation of information on the ground and its characteristics so as to facilitate an appropriate foundation design and enable a practical, safe and economic construction process to be planned".**

- **It must include the detailed survey of the land, structures and services within and adjacent to the site. The survey can be**
  - **Topographical, geological, structural, survey of disused tunnel, culvert, nullah or stream course, survey of underground structures.**

Site Investigation

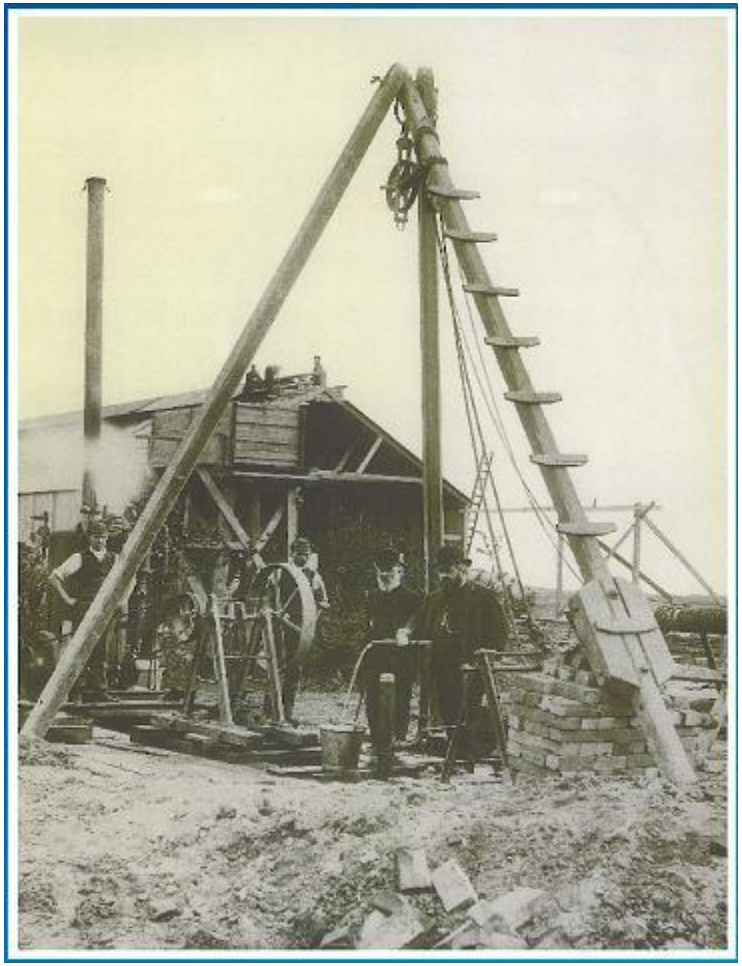
Requirement

- To determine the type of foundation required for the proposed project at the site, i.e. shallow foundation or deep foundation.
- To make recommendations regarding the safe bearing capacity or pile load capacity.
- Ultimately, it is the subsoil that provides the ultimate support for the structures.

Inspite of such importance of SI

- What hasn't changed?
  - Clients understanding / perception of the industry
  - Status of geotechnics generally
  - Funding levels for innovation
  - Time taken to get test results from laboratories
  - Length of time to complete a borehole
  - Availability of geotechnical engineers
  - Number of'can tackle any problem' SI contractors

Quality of site investigation report



"PUT ANOTHER SHOVEL IN, FRED — REMEMBER THEY WANT FULL CORES."

Boring and sampling – tradition

- Could we do without them
- They a waste of money

Mining Subsidence

Double-decker bus trapped by mining subsidence, in Norwich (UK) (Courtesy Eastern Counties News Papers Limited)

Double-decker bus trapped by mining subsidence, in Norwich (UK) (Courtesy Eastern Counties News Papers Limited)

The Exploration Program

- The program should be planned so that the maximum amount of information can be obtained at minimum cost.

- Steps:
  1. Assembly of available information
  2. Reconnaissance of the area
  3. Preliminary site investigation
  4. Detailed site investigation

Less is more?

For straight forward ground conditions:

Use-

- Maps & historical data
- Desk studies
- Previous SI's
- Published data for soil and rock type
- Limited intrusive SI

For difficult ground conditions need detailed desk study and more extensive SI

BUT…

- SI MUST be tailored to the structures
- SI need not be proportional to size of site (scatter gun)

Execution

- Collection of disturbed and/or undisturbed samples of subsurface strata from field.
- Conducting in-situ tests of subsurface material and obtaining properties directly or indirectly.
- Study of ground water conditions and collection of sample for chemical analysis.
- Geophysical exploration, if necessary.
- Laboratory testing on samples

A complete site investigation will consist of:

- Preliminary work
  - Collecting general information and already existing data such as study of geologic , seismic maps, etc. at or near site.
  - Study site history – if previously used as quarry, agricultural land, industrial unit, etc.
- Site Reconnaissance: Actual site inspection.
  - To judge general suitability
  - Decide exploration techniques

- Exploration
  - Preliminary Investigations: Exploratory borings or shallow test pits, representative sampling, geophysical investigations, etc
  - Detailed Investigations: Deep boreholes, extensive sampling, in-situ testing, lab testing, etc.
  - Depth and spacing: In general, depth of investigation should be such that any/all strata that are likely to experience settlement or failure due to loading. Spacing depends upon degree of variation of surface topography and subsurface strata in horizontal direction. Refer to Alam Singh.

Methods of Investigation

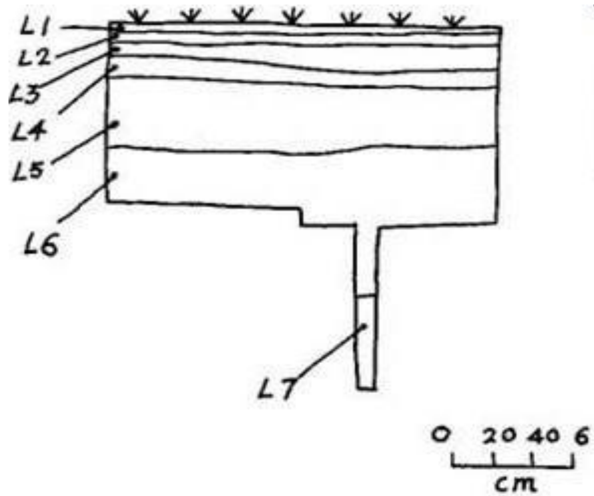
- Test pits:
  - Permits visual inspection of subsurface conditions in natural state.
  - Max. depth limited to 18 - 20 feet.
  - Especially useful for gravelly soil where boreholes may be difficult.
  - Sampling/testing done on exposed surfaces.

Stratigraphy and Finds

Layer	Soil	Soil Colour	Finds	Chronology
L1	Sandy soil	Gray 7.5YR 5/1	Modern Rubbish (filled soil)	1980s
L2	Sandy soil	Pinkish white 7.5YR 8/2	Modern rubbish (filled soil)	1980s
L3	Sandy soil	Reddish yellow 7.5YR 7/6	Modern rubbish (filled soil)	1980s
L4	Sandy soil	Gray 7.5YR 6/1	Modern rubbish (filled soil)	1980s
L5	Loamy soil	Reddish yellow 5YR 6/6	Nil (original decomposed soil)	
L6	Loamy soil	Reddish yellow 5YR 6/8	Nil (original decomposed soil)	
L7	Loamy soil, with some decomposed bed rock texture	Light red 2.5YR 6/8	Nil (original decomposed soil)	

Test Pit Wall Photograph
Western Wall Section

Test Pit Wall Drawing
Western Wall Section Drawing



GROUND WATER TABLE LEVEL

Groundwater conditions and the potential for groundwater seepage are fundamental factors in virtually all geotechnical analyses and design studies. Accordingly, the evaluation of groundwater conditions is a basic element of almost all geotechnical investigation programs. Groundwater investigations are of two types as follows:

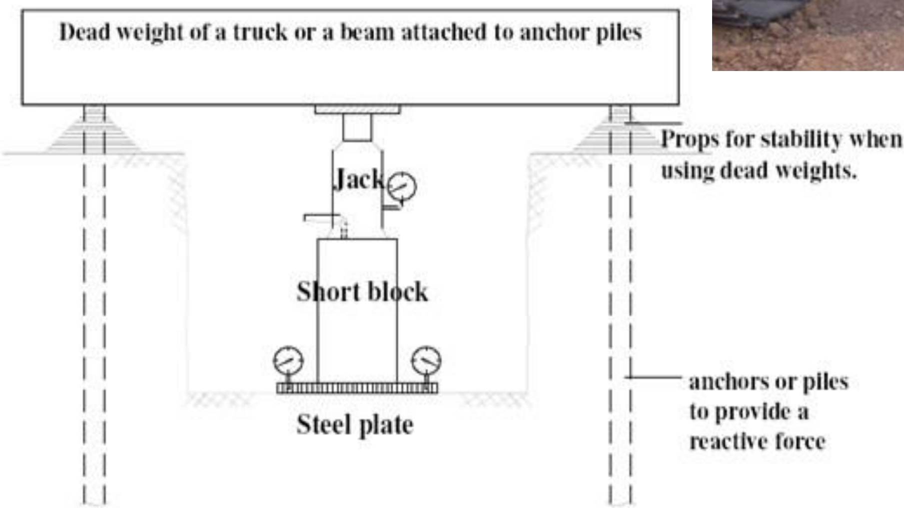
- Determination of groundwater levels and pressures.
- Measurement of the permeability of the subsurface materials.

FIELD STRENGTH TESTS

The following are the major field tests for determining the soil strength:

1. Vane shear test (VST).
2. Standard Penetration Test (SPT).
3. Cone Penetration Test (CPT).
4. The Borehole Shear Test (BST).
5. The Flat Dilatometer Test (DMT).
6. The Pressure-meter Test (PMT).
7. The Plate Load Test (PLT).

The Plate Load Test (PLT)



Several dial gauges attached to an independent suspension system to record plate settlements with each increment of the jack load.

The Plate Load Test (PLT)

Scalle Effect irn Foundation Desi1gn

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Compacted Layer

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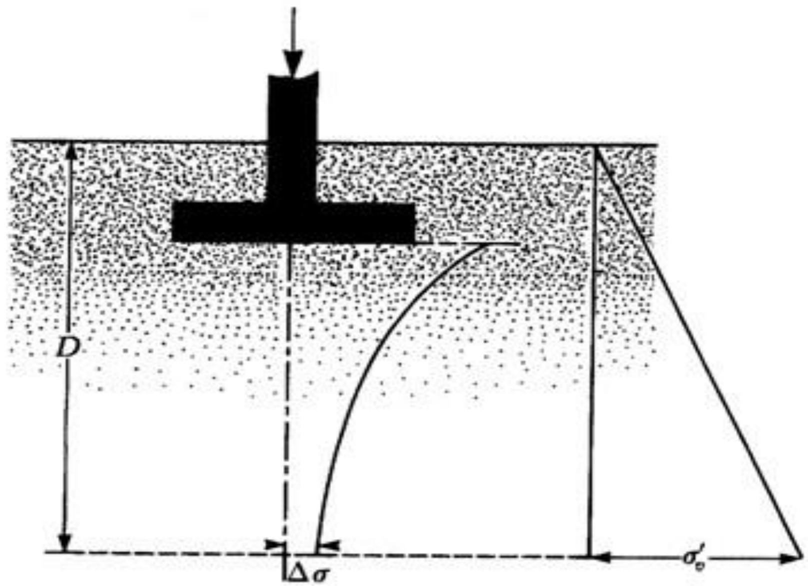
For Foundations..

- Foundation Design Preparation:
  - Calculate proposed loads to be transmitted by foundation to underlying rocks & soils.
  - Incorporate all requirements into design of foundations.
  - Obtain data on soils & rock properties.
  - Soil investigations-
    - **Check** for **natural exposures** in road cuts or gullies, or auger drill for samples;
    - **Laboratory test soils** for shear strength, compressibility, swelling characteristics
    - Disturbed soils have lost soil structure- limited use
    - **Undisturbed soils** used for wet/dry density, triaxial shear, & compression, permeability, consolidation tests
    - Soil samples change on exposure to air- extended exposure reduces their suitability for testing.
    - **In-situ measurements** give superior physical determinations.

Depth of Boring

The approximate required minimum depth of the borings should be predetermined. The estimated depths can be changed during the drilling operation, depending on the subsoil encoun•tered.



Determination of the minimum depth of boring

Depth of Boring

Building width (m)	Number of Stories				
	1	2	4	8	16
	Boring Depth (m)				
30.5	3.4	6.1	10.1	16.2	24.1
61.0	3.7	6.7	12.5	20.7	32.9
122.0	3.7	7.0	13.7	24.7	41.5

Depth of Boring

For hospitals and office buildings, the following rule could be use to determine boring depth

$D_b = 3S^{0.7}$ (for light steel or narrow concrete buildings)

$D_b = 6S^{0.7}$ (for heavy steel or wide concrete buildings)

where:

D_b = depth of boring, in meters

S = number of stories

Spacing Boring

There are no hard and fast rules for the spacing of the boreholes. The following table gives some general guidelines for borehole spacing. These spacing can be increased or decreased, depending on the subsoil condition. If various soil strata are more or less uniform and predictable, the number of boreholes can be reduced.

Type of project	Spacing (m)
Multistory building	10-30
One story industrial plants	20-60
Highways	250-500
Residential subdivision	250-500
Dams and dikes	40-80

Investigations in Rock sites

- RockVariability:
  - Rocks & lithologies highly variable in all three dimensions.
  - Must assess this variability/ anisotropy.
  - Not all rocks outcrop equally- some more resistant to weathering.
  - Surface outcrops can yield biased data, if considered solely.
  - Soft & less resistant rocks may not outcrop at all.
  - All rocks can be obscured by thin sheets of younger sediments.
  - Deformation features such as folds, faults, fractures, shear zones must be identified.
  - These are frequently preferentially weathered & infilled by secondary materials.
  - Folds alter orientations of planes of weakness.
  - Weathering depths may vary considerably over different rock types - <u>affect rock strengths</u>.

Site Investigations

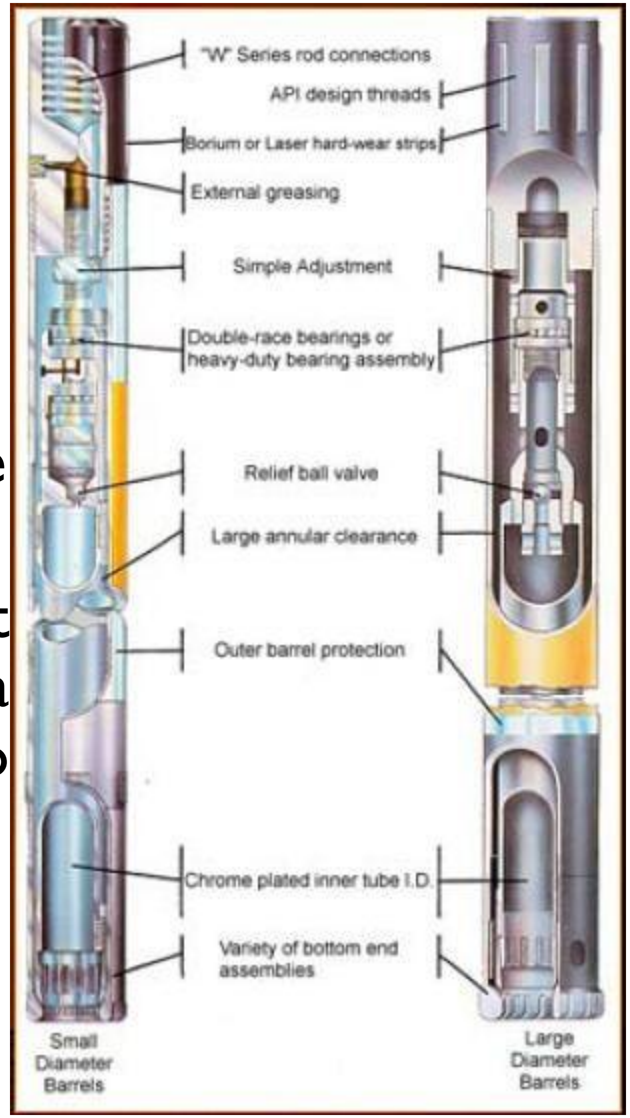
- Planes of Weakness:
  - Discontinuities in rocks have > effect on rock properties than lithology.
  - Include - bedding planes, joints, foliations, cleavage, faults, etc.
  - All influence Foundation design.
  - Remember- compressive strength is > perpendicular to discontinuity than // to it.
  - All discontinuities evaluated for character, orientation, frequency, etc.
  - Data best determined on rock exposures- more difficult on core.
  - Trenches & costeans very useful.

Site Investigations for Foundations

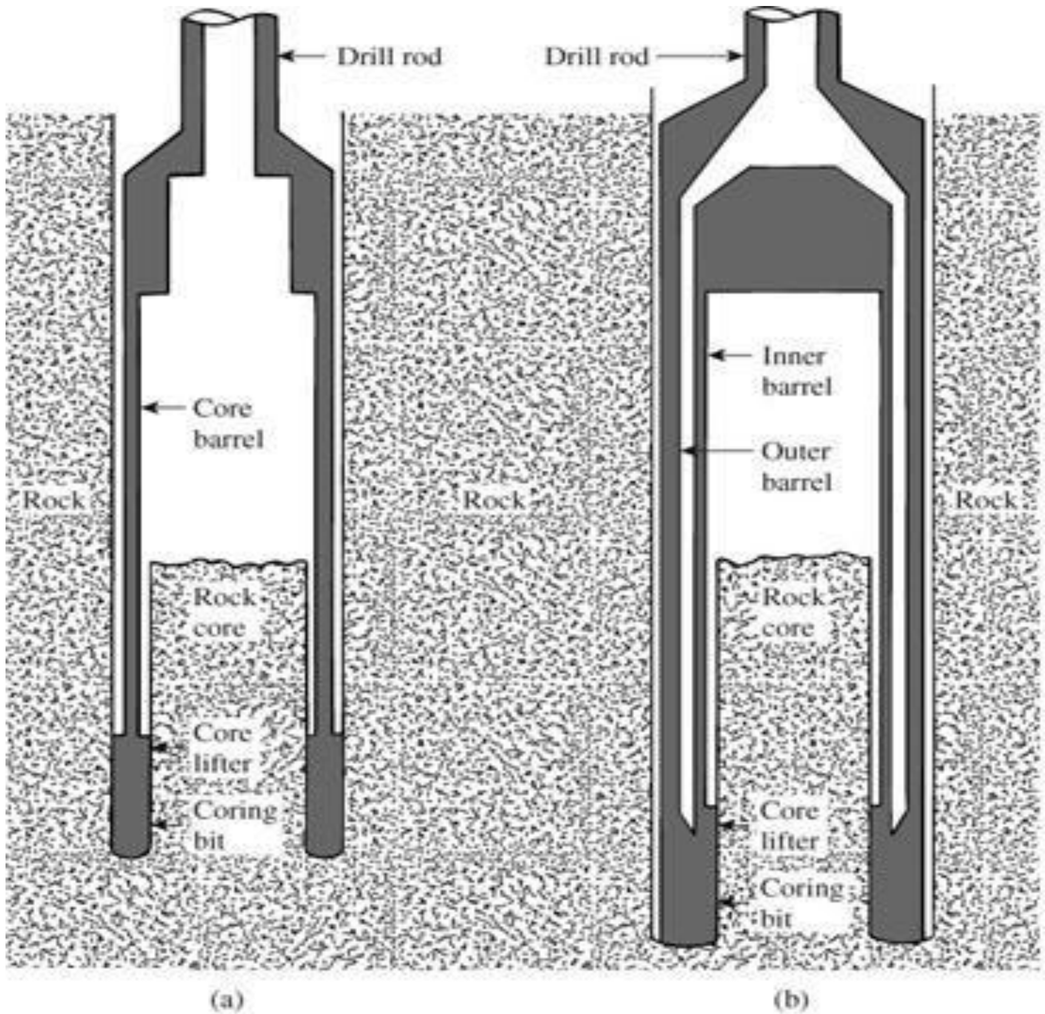
- Three main types-
  - **Solid rock** - rock strength & discontinuities identified.
  - **Soil & solid rock** at accessible depths - establish depth to Bedrock & as above.
  - **No Solid Rock** - must place foundations in unconsolidated materials .Must also allow for rate of anticipated settling.

ROCK SAMPLING

- Rock cores are necessary if the soundness of the rock is to be established.
- Small cores tend to break up inside the drill barrel.
- Larger cores also have a tendency t break up (rotate inside the barrel a degrade), especially if the rock is so or fissured.



Rock coring



Rock coring: (a) single-tube core barrel; (b) double-tube core barrel

ROCK SAMPLING - Definition

$$\text{Recovery Ratio} = \frac{\sum \text{Lengths of intact pieces of core}}{\text{Length of core advance}}$$

$$\text{RQD} = \frac{\sum \text{Lengths of intact pieces of core} \geq 10.16\ \text{cm}}{\text{Length of core advance}}$$

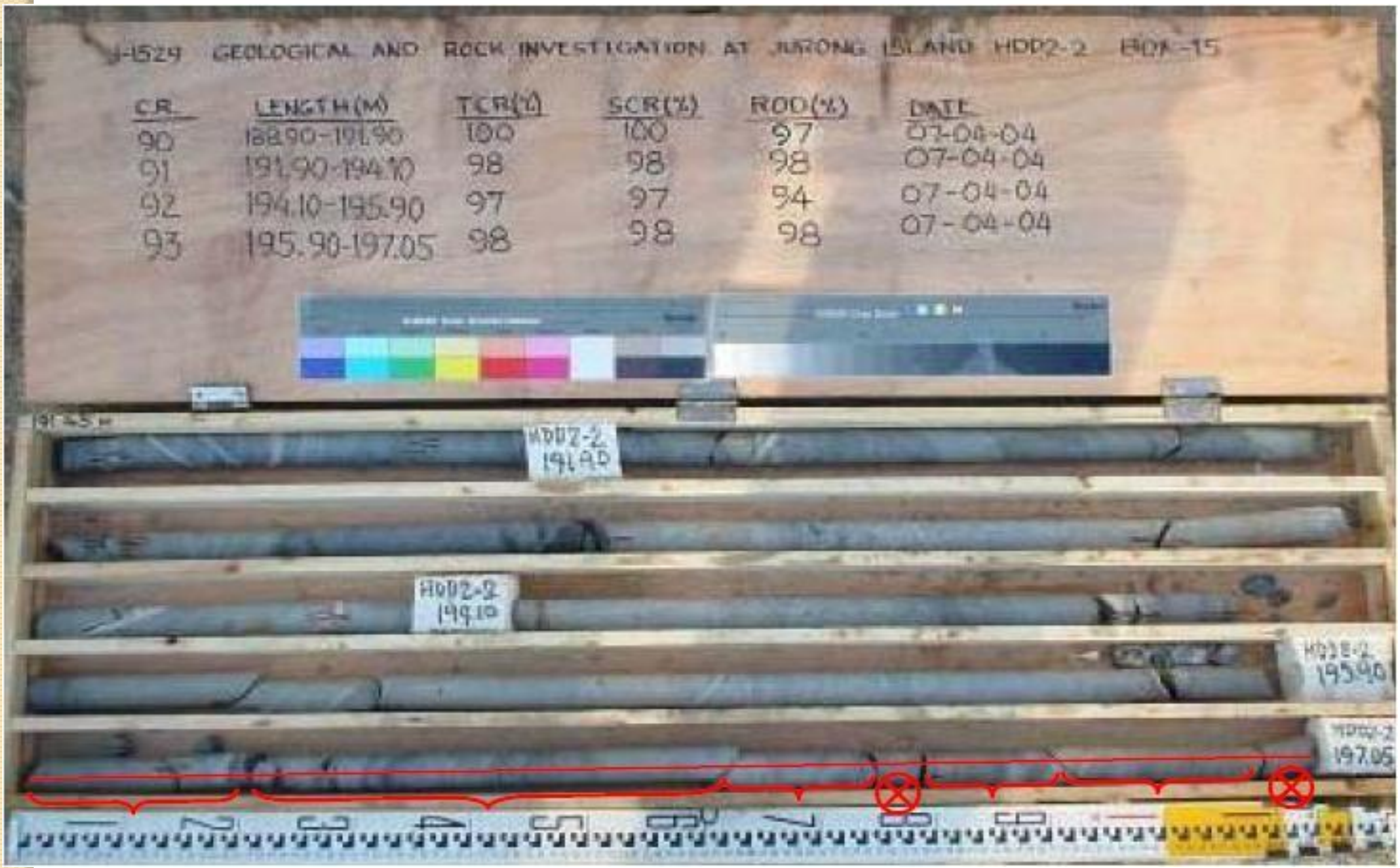
Rock Core Drilling

- Done with either tungsten carbide or diamond core bits
- Use a double or triple tube core barrel when sampling weathered or fractured rock
- Used to determine Rock Quality Designation

Diamond coring bit

core barrel

Rock Quality Designation RQD

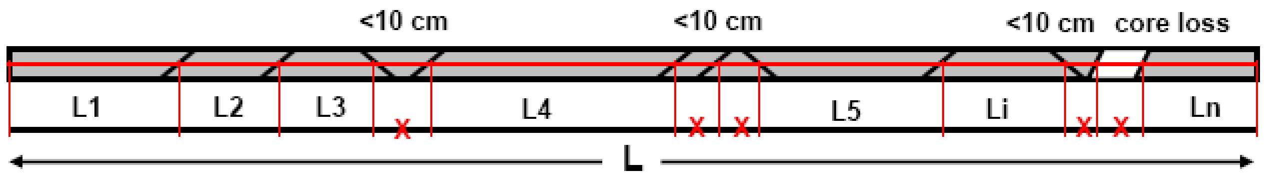


Rock Quality Designation

RQD

Rock Quality Designation (RQD) is defined as the percentage of rock cores that have length equal or greater than 10 cm over the total drill length.

$$RQD = \Sigma Li / L \times 100\%, \quad Li > 10\ cm$$



$$RQD = (L1 + L2 + \ldots + Ln) / L \times 100\%$$

RQD	Rock Mass Quality
< 25	Very poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
99 – 100	Excellent

Example on Core Recovery & RQD

- Core run of 150 cm
- Total core recovery = 125 cm
- Core recovery ratio = 125/150 = 83%
- On modified basis, 95 cm are counted

RQD = 95/150=63 %

Core Recovery cm	Modified Core Recovery, cm
25	25
5	0
5	0
7.5	0
10	10
12.5	12.5
7.5	0
10	10
15	15
10	10
5	0
12.5	12.5
125	95

Site Investigations

- General Procedures:
  - Collect & assess all published accessible data on soils & rocks.
  - Conduct detailed geological study of project areas.
  - Use combination of drilling & geophysical surveys to complete geology, and confirm interpreted geology from surface outcrops.
  - Field test sub surface materials to determine engineering properties.
  - Detailed laboratory tests on sub surface materials to determine physical properties.
  - All data reviewed, assessed, & recommendations made on site suitability for project.
  - Continued investigations of sub surface materials while project constructed -confirms earlier interpretations or leads to modifications of plans/ construction methods.

- Dam Foundations:
  - Small dams for rural purposes- based on soil mechanics & sited in gully
  - Large Dams- must investigate underlying rocks of dam area:
    - Check **rock strength** is adequate to support water load;
    - Check for **weakness planes** & potential slippage;
    - Establish **orientation** of any weakness planes;
    - **Depth of weathering-** removal prior to dam construction;
    - Determine **Durability** of rock to water exposure;
    - Measure rock **permeability**;
    - Identify any seismic record of **earthquake activity**;
    - Dam type chosen based on availability of materials;
    - Establish risk of **siltation** reducing dam capacity, before construction.

Geophysical techniques:

- Advantages
  - Relatively **low cost**
  - Obtain results quickly
  - Can be undertaken in rough , inhospitable terrains by small teams and
  - Can assist planning of expensive drilling programs.
- Limitations
  - Techniques all identify boundaries between two layers with appreciably different properties. Little contrast - poor definition of layers.
  - Requires confirmation by independent means.

- Techniques
  - Seismic reflection & refraction
  - Electrical resistivity (ER)
  - Ground Penetrating Radar (GPR)
  - Gravity & Magnetic Surveys, and
  - Downhole techniques.

Seismic Methods

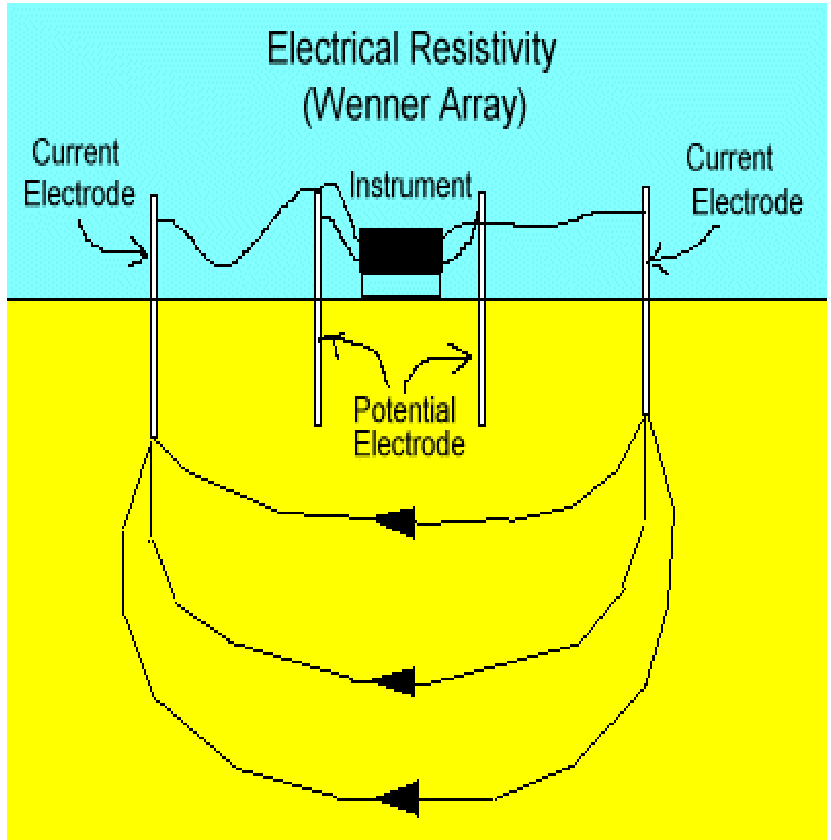
- Seismic Methods involve propagation of waves through earth materials. The methods can be *seismic refraction* and *seismic reflection methods*.
- ***Seismic Refraction:***
  - Theory of refraction derived from behaviour of rays that bend on entering a different velocity medium. The larger the velocity difference between two media, the larger the refraction.
  - Used for depths of 30-60 metres. Soils below ground water table can be distinguished from unsaturated soils above water table
- ***Seismic Reflection:***
  - Depths determined by observing travel times of P waves generated near surface & reflected back from deep formations. Comparable to echo sounding of water depths.
  - Advantages- permits mapping of many horizons for each shot. Can determine depths to dipping interfaces, as well as angle of dip.

Electric Methods

- Electrical methods involve measurement of electrical properties of earth materials either-
  - measurement of natural earth currents, or
  - the resistance to induced electrical flow.
- Natural earth current flow generated under geological conditions in which anode & cathode develop naturally.
  Measurement of strength & extent of current helps establish geologic conditions.
- Electrical resistivity is resistance to electrical flow through earth materials. Current induced & resistivity measured- identifies basic property of earth material.

Advantages

- Non-Destructive
- Cost Effective
- Provides Preliminary or Supplemental Information



Summary :

- Refraction seismic & electrical resistivity are two most applicable techniques to engineering geology.
- Best methods practicable for exploring shallow depths of the sub-surface to 30 m.
- If there is a low velocity layer below a high velocity layer, refraction seismic will not work properly.
- This occurs where sand & gravel layer lies below a clay layer
- Electrical resistivity method would work well here;
- Refraction seismic works well where soil overlies bedrock.

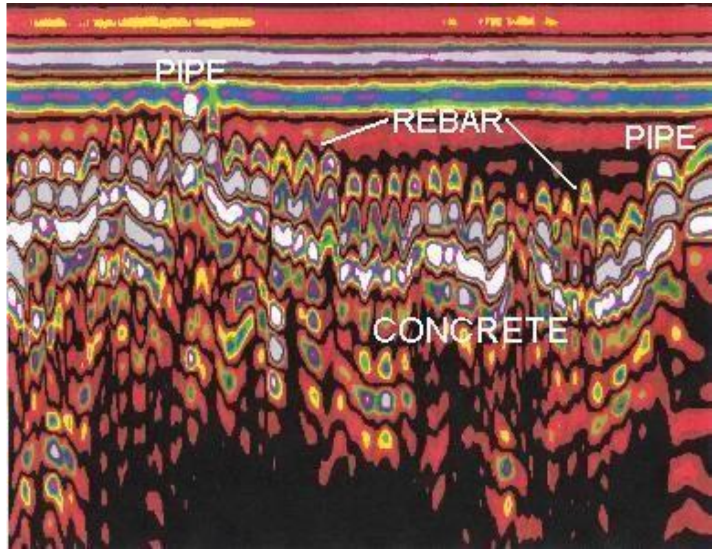
Ground-penetrating Radar

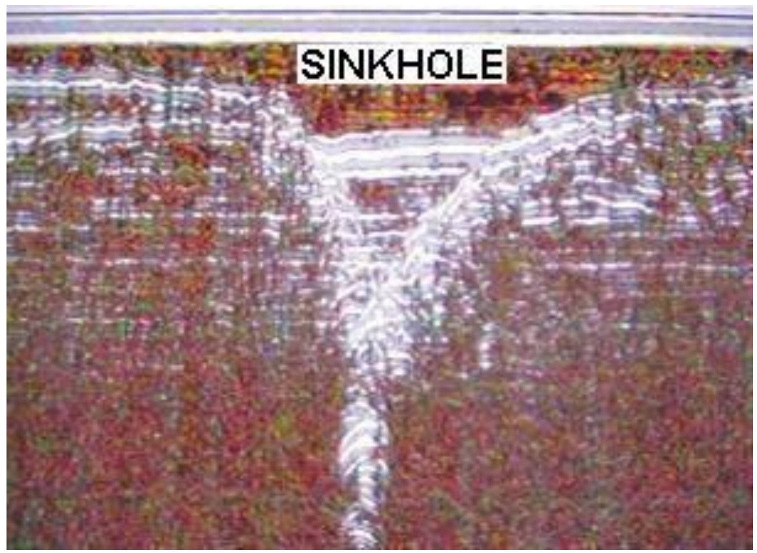
- Essentially the same as Reflection Seismology
- Radar impulse is energy source & receiver used to detect reflections
- Strength of reflections depends on the electromagnetic properties.
- GPR :
  - Can identify sinkholes at depths of 25 m
  - Subsurface anomalies were identified as voids in old earth-filled dam in Michigan -locations aided in a grouting program to fill voids
  - Gaining rapid acceptance in environmental engineering applications
  - Can be used for non-destructive testing of highways

Application of GPR for different Antenna

Application	Antenna Frequency (MHz)	Approximate Depth (m)
Structural Concrete, Roadways, Bridge Decks	2600	0-0.3
	1600	0-0.45
	1000	0-0.6
Concrete, Shallow Soils, Archaeology	900	0-1
Shallow Geology, Utilities, UST's, Archaeology	400	0-4
Geology, Environmental, Utility	270	0-5.5
	200	0-9
Geologic Profiling	100	0-30

- Greater surface difference = Stronger signal
- Strong signal has large amplitude
- Weak signal has small amplitude
- Amplitude wavelength and time are used to create image of what is underground



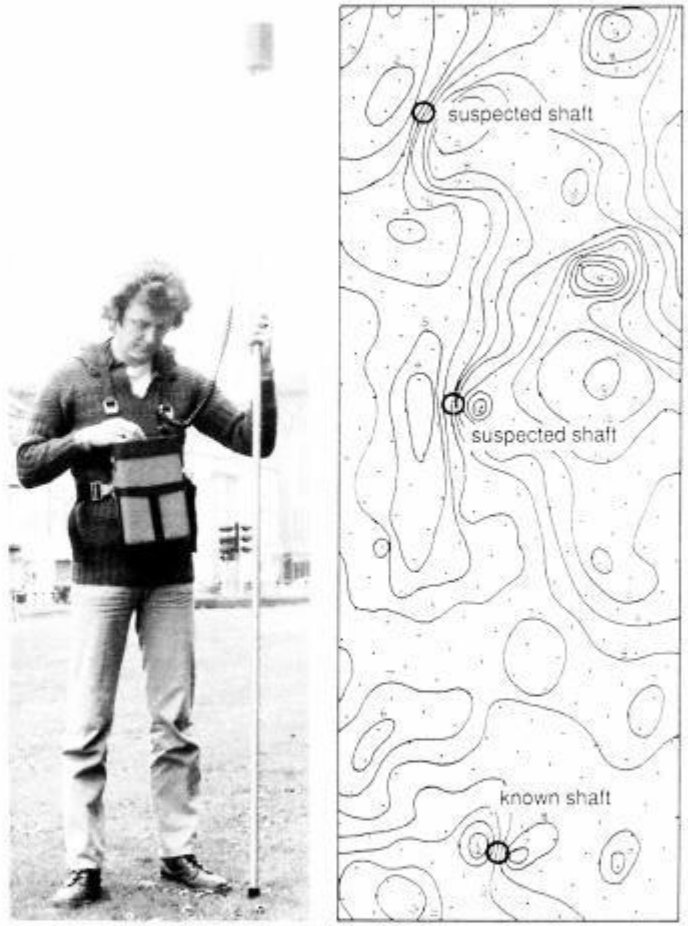


Gravity and Magnetic Methods

- Deal with strength of the fields of gravity & magnetism generated between a mass of rock and the Earth
- Measure Earth's surface gravity or magnetic field & compare with that of an adjacent area. Changes known as anomalies that imply the size, nature & location of a high or low gravity/magnetic source
- Used for specialised engineering applications only.

Magnetic surveys

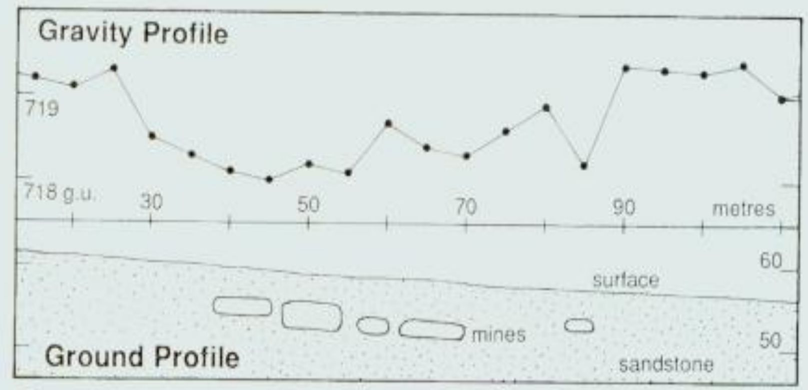
- Utilizes dipole anomalies arising from vertical linear features
- Particularly useful for identifying buried mine shafts



Proton magnetometer and map of a magnetic survey on a site in the Derbyshire coalfield. Stations are on 3 m grid; contours are at 100 nT intervals above a base of 48 000 nT. Dipole anomalies lie over one known shaft and indicate the location of two more shafts

Gravity surveys

- Underground voids or low density rock/soil show up as negative anomalies
- Usual to drill all negative anomalies



Gravity surveys over sand mines in Nottingham

- Magnetic & Gravity Methods:
  - Rarely used in engineering geology;
  - Many new geophysical devices that utilise these properties of rocks;
  - Most commonly used in delineation of buried valleys or basin fills.

Well logging methods

A variety of techniques that involve lowering instruments down a drill hole & generating data on sub- surface rock types as the instrument traverses the hole.

- Well logging techniques:
  - Record generated by lowering probe into an uncased drill hole;
  - Spontaneous potential (SP), & resistivity logging are most common;
  - Also Gamma ray- all rocks & soils are naturally radioactive;

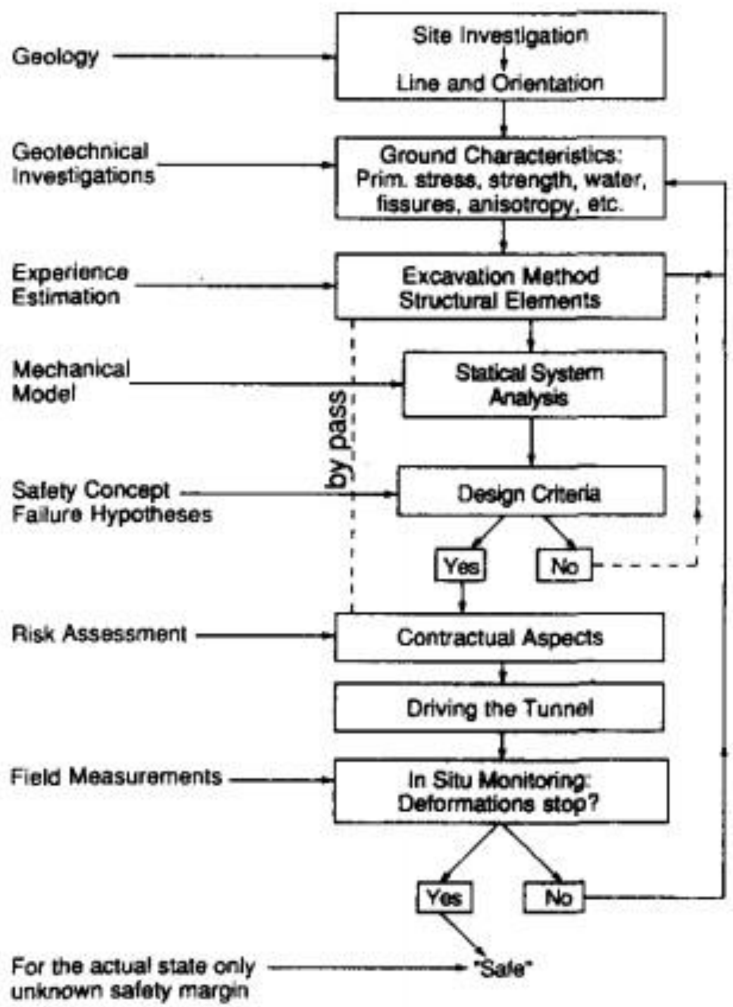
Conclusions

- The code requirements for site investigation, if any, usually stipulate the minimum amount of site work required.
- Site investigation is a specialized operation, requires specialized organizations and specialized personnel.
- Site investigation is the combined product from ground investigation contractor and geotechnical consultant. The contractor is responsible for obtaining reliable data.
- The geotechnical consultant is responsible for planning and execution of the site investigation work, interpretation and analyses of data, recommendations of design and assumed professional responsibility.

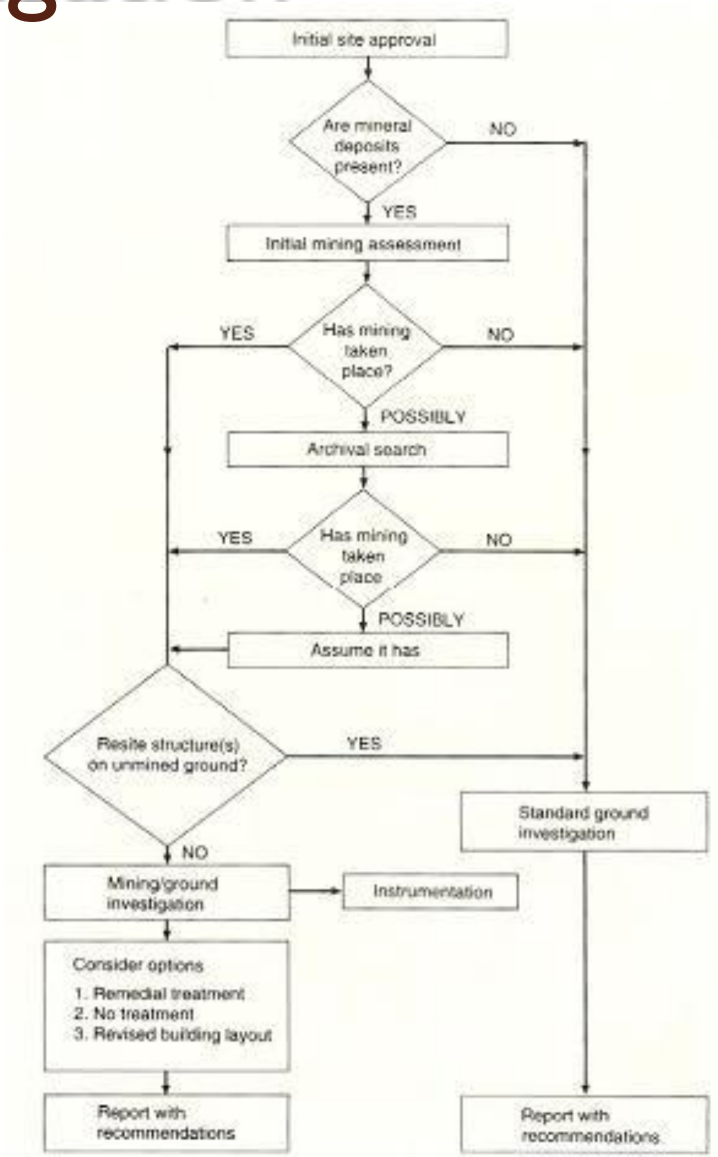
Conclusions

- The extent and cost of site investigation should be such that the risk is at an established acceptable level to the designer and also comply to the accepted code of practice.
- The practice of recommending lowest tender as the main criteria for site investigation should not be preferred but be discouraged.
- Selection should be made on the basis of the geotechnical consultant’ s competency and investigation contractor’ s ability to provide reliable factual data.

Tunneling Investigation



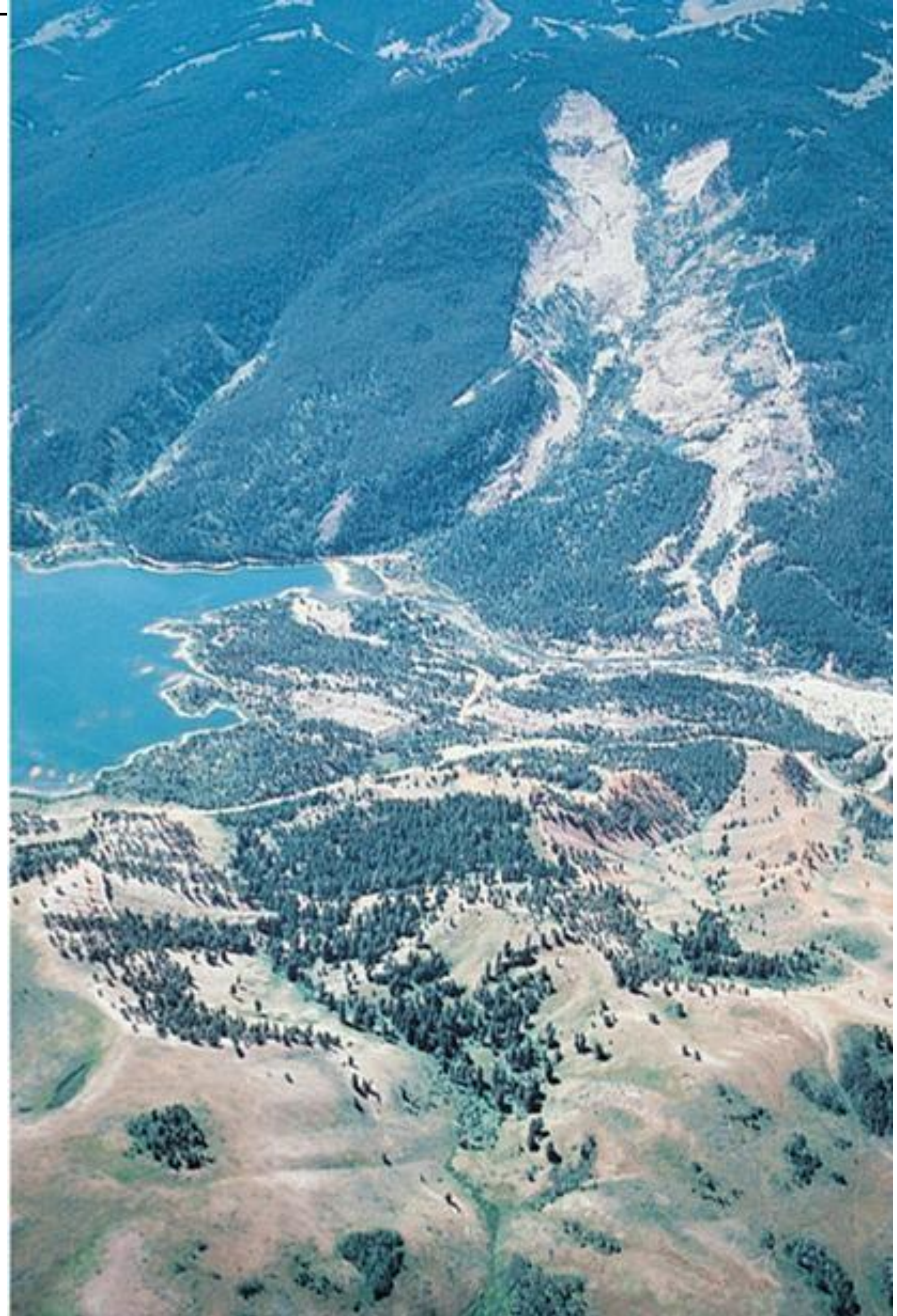
Mining Investigation



Infinite and finite earth slopes, types of failures, factor of safety of infinite slopes, stability analysis by Swedish arc method, standard method of slices, Bishop's Simplified method, Taylor's Stability Number, Stability of slopes of earth dams under different conditions.

Gros Ventre Slide, WY, 1925
(pronounced —gro vahnt‖)
w

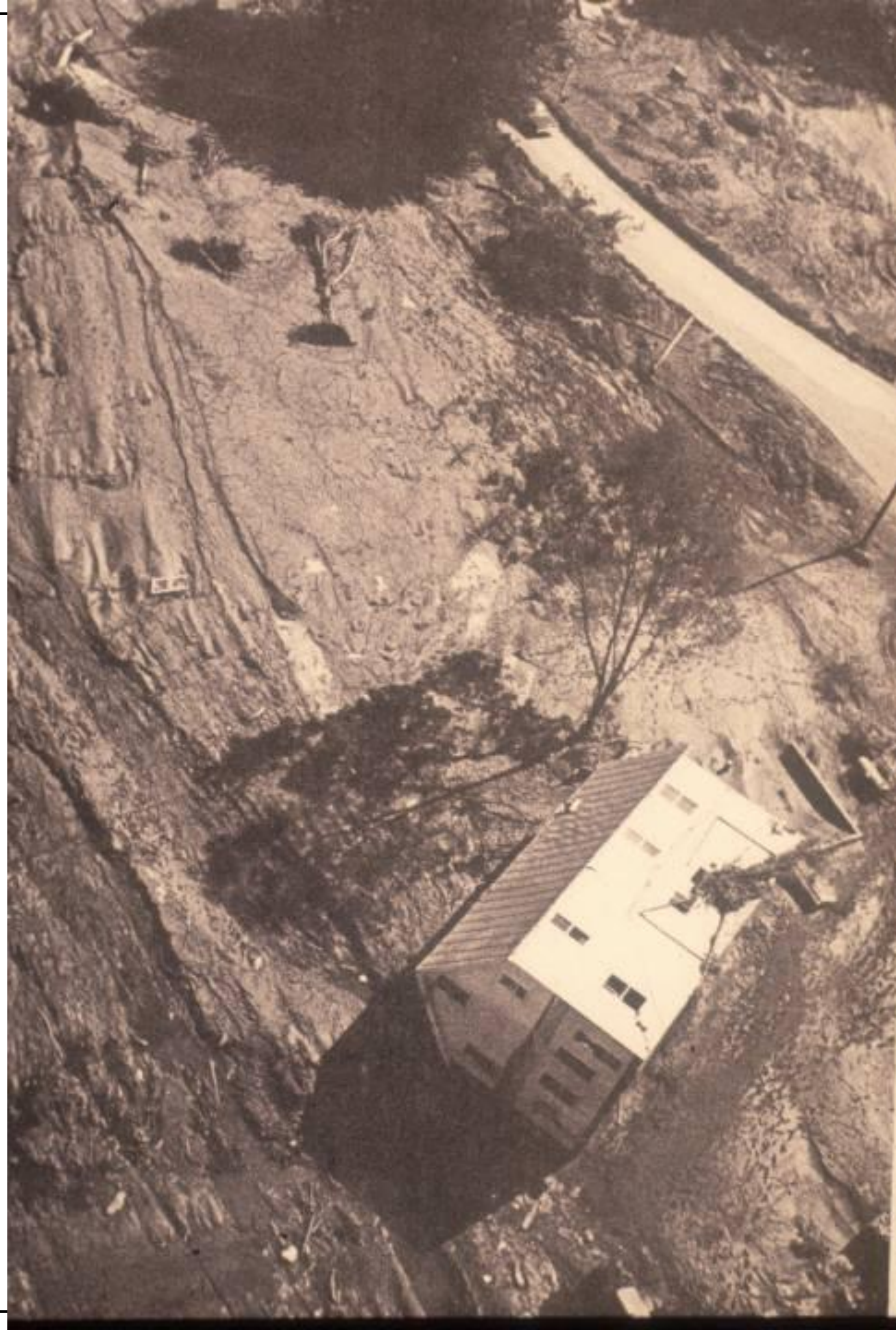
50 million cubic yards



Earthquake Lake, MT, 1959

29 fatalities

Nelson County, VA



Slope Stability

I. Stresses and Strength

A. Applies to all sloping surfaces

- Balancing of driving and resisting forces
- If Resisting forces > Driving Forces: *stability*

Slope Stability

I. Stresses and Strength

A. Applies to all sloping surfaces
- Balancing of driving and resisting forces
- If Resisting forces > Driving Forces: *stability*

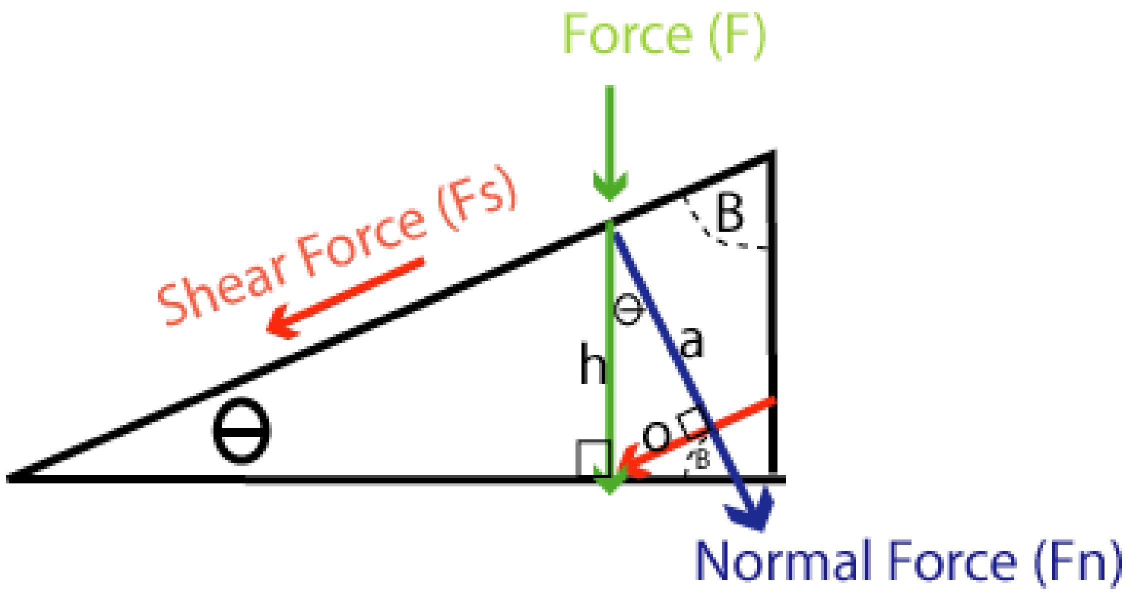
B. Engineering Approach
- Delineate the surface that is most at risk
- Calculate the stresses
- Calculate the Shear Strength

A Friendly Review From Last Month……

Stress on an inclined plane to Force

σ = Force / Area

Where is Normal Force and Shear Force = ??



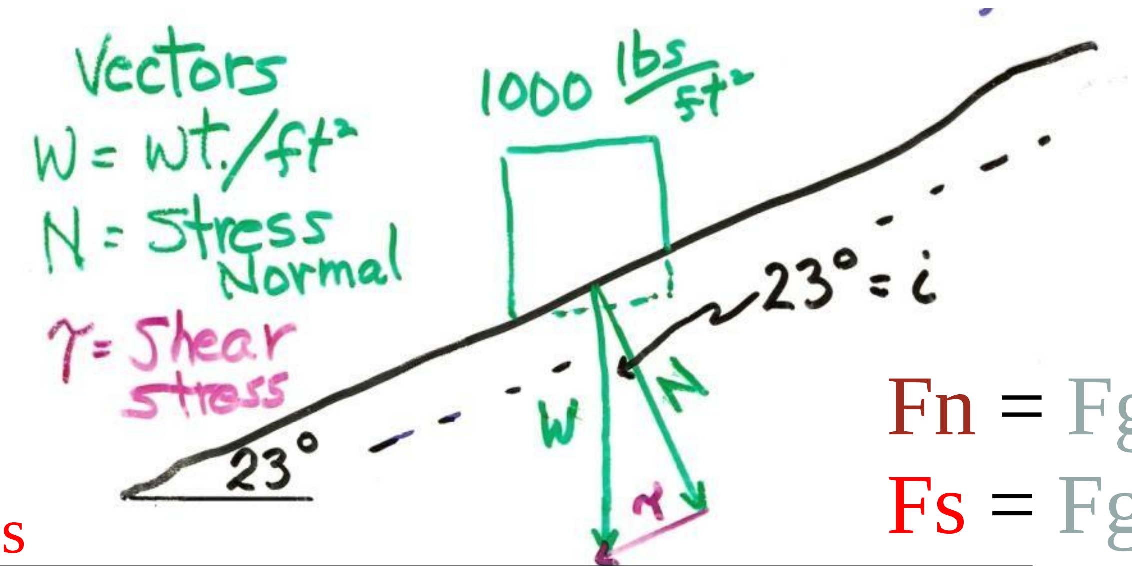
$$\cos \Theta = \frac{a}{h} = \frac{Fn}{Fg}$$

$$\sin \Theta = \frac{o}{h} = \frac{Fs}{Fg}$$

$$Fn = Fg \cos \Theta$$

$$Fs = Fg \sin \Theta$$

Shear Stress Analysis



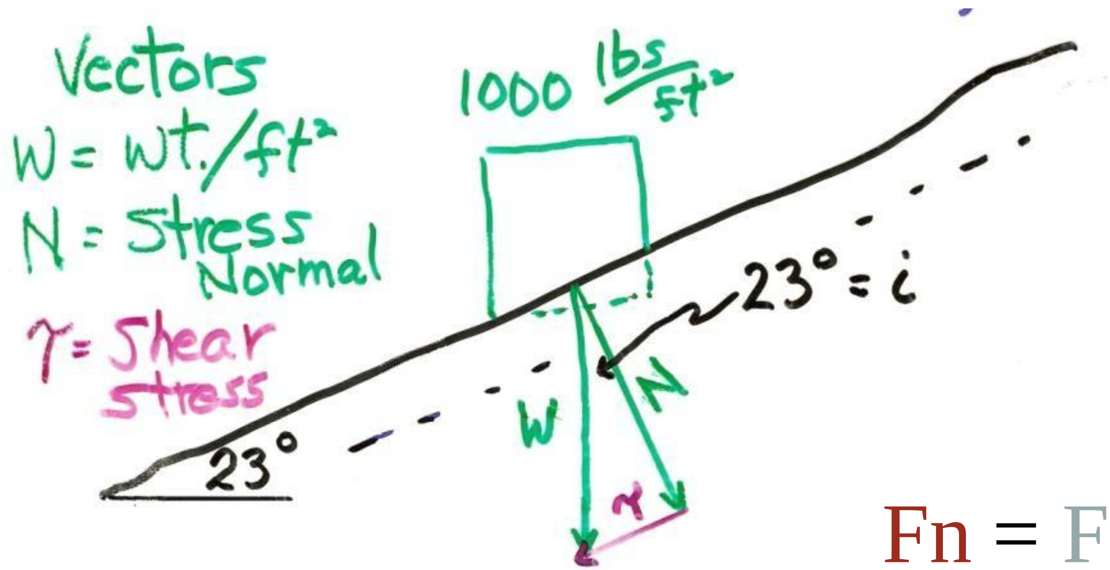
Find the
Shear stress

Fn = Fg cos Θ

Fs = Fg sin Θ

What is behind this pretty little box???

Shear Stress Analysis



$$Fn = Fg \cos \Theta$$

$$Fs = Fg \sin \Theta$$

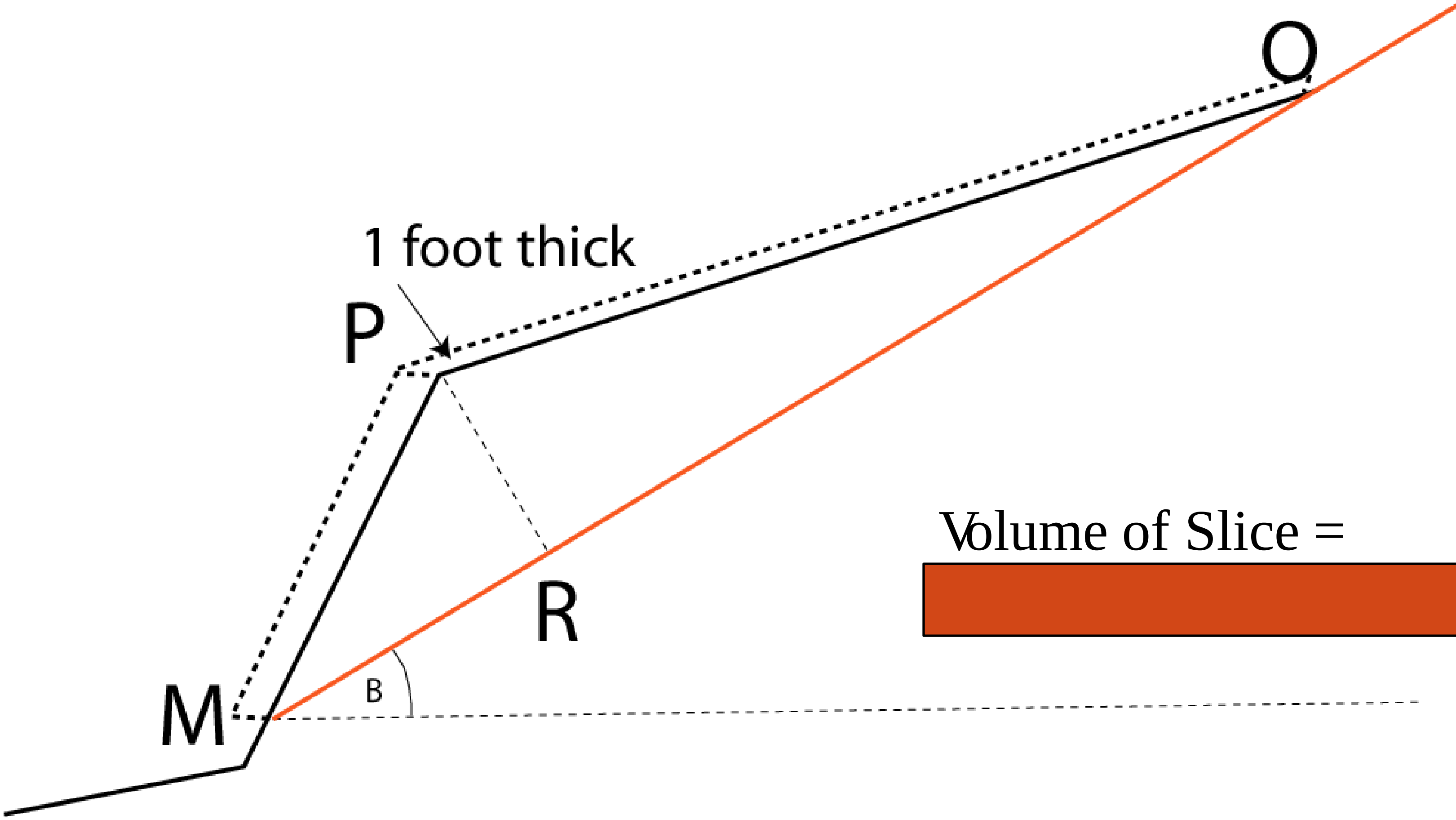
$$\sin i = \frac{\tau}{W}$$

$$\therefore \tau = W \sin i$$

$$\tau = 1000 \text{ lbs/ft}^2 \times 0.39$$

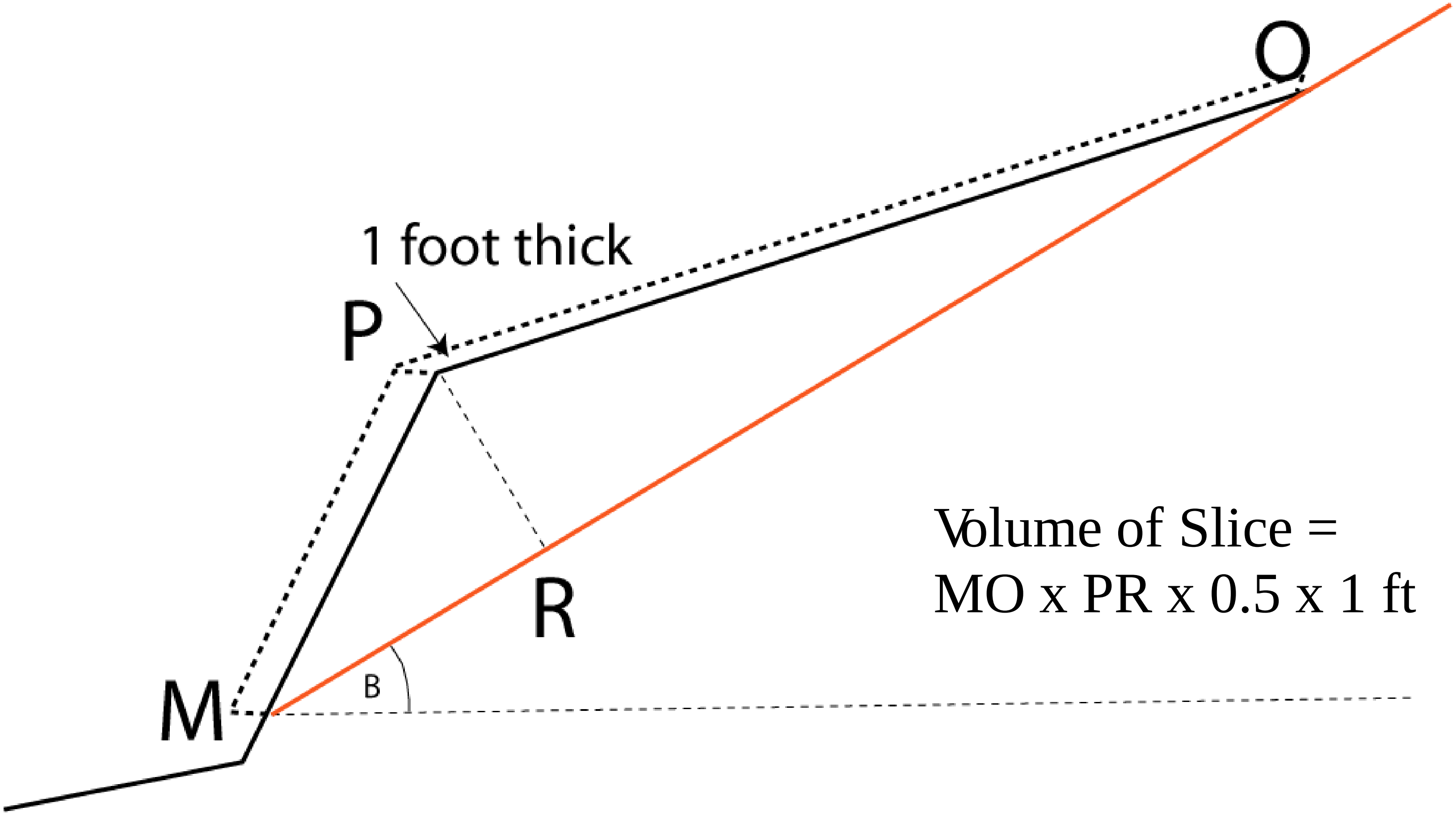
$$= 390.73 \text{ lbs/ft}^2$$

II. Planar Slide—case 1



Volume of Slice =

II. Planar Slide—case 1



Volume of Slice =
MO x PR x 0.5 x 1 ft

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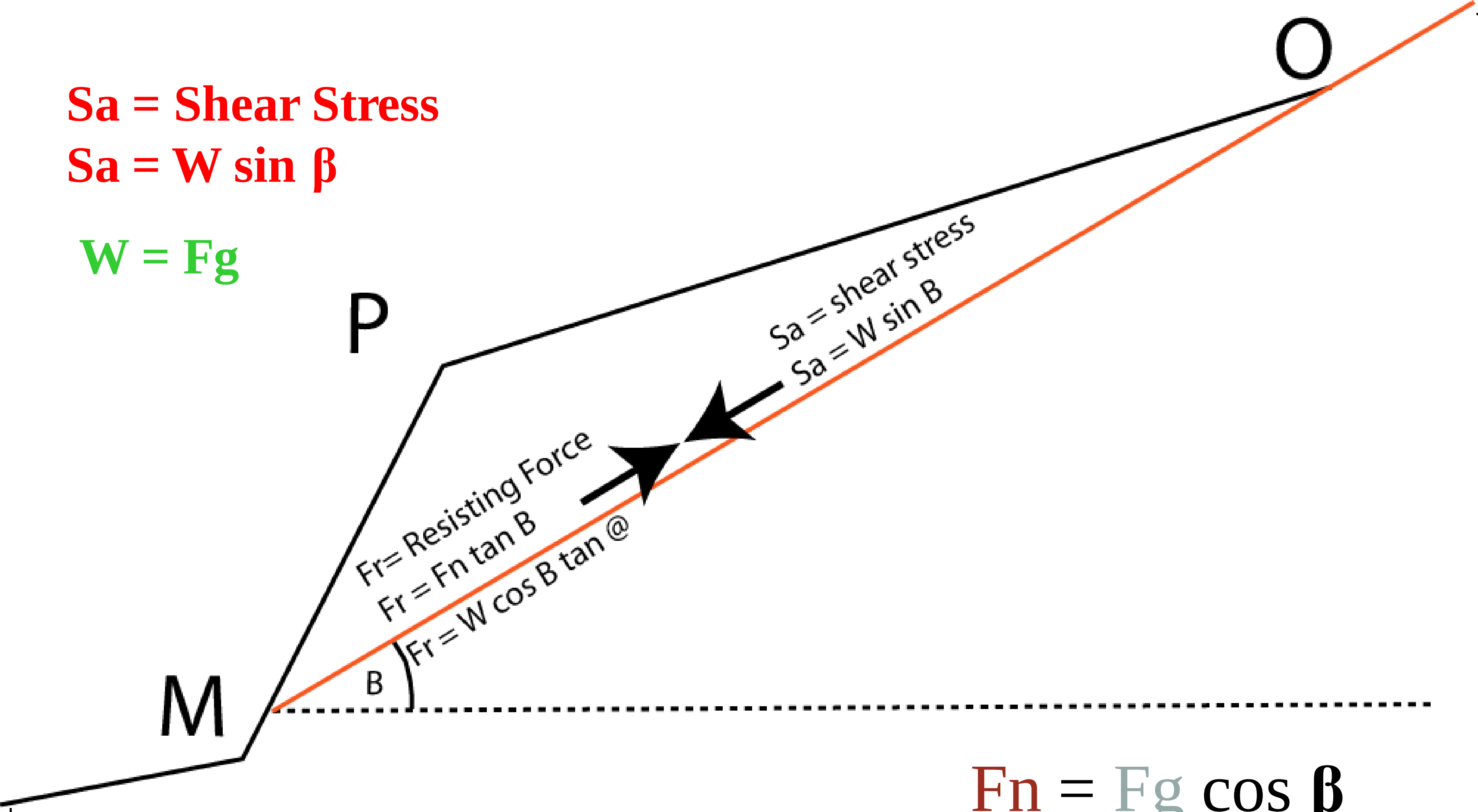
Sa = Shear Stress

Sa = W sin β

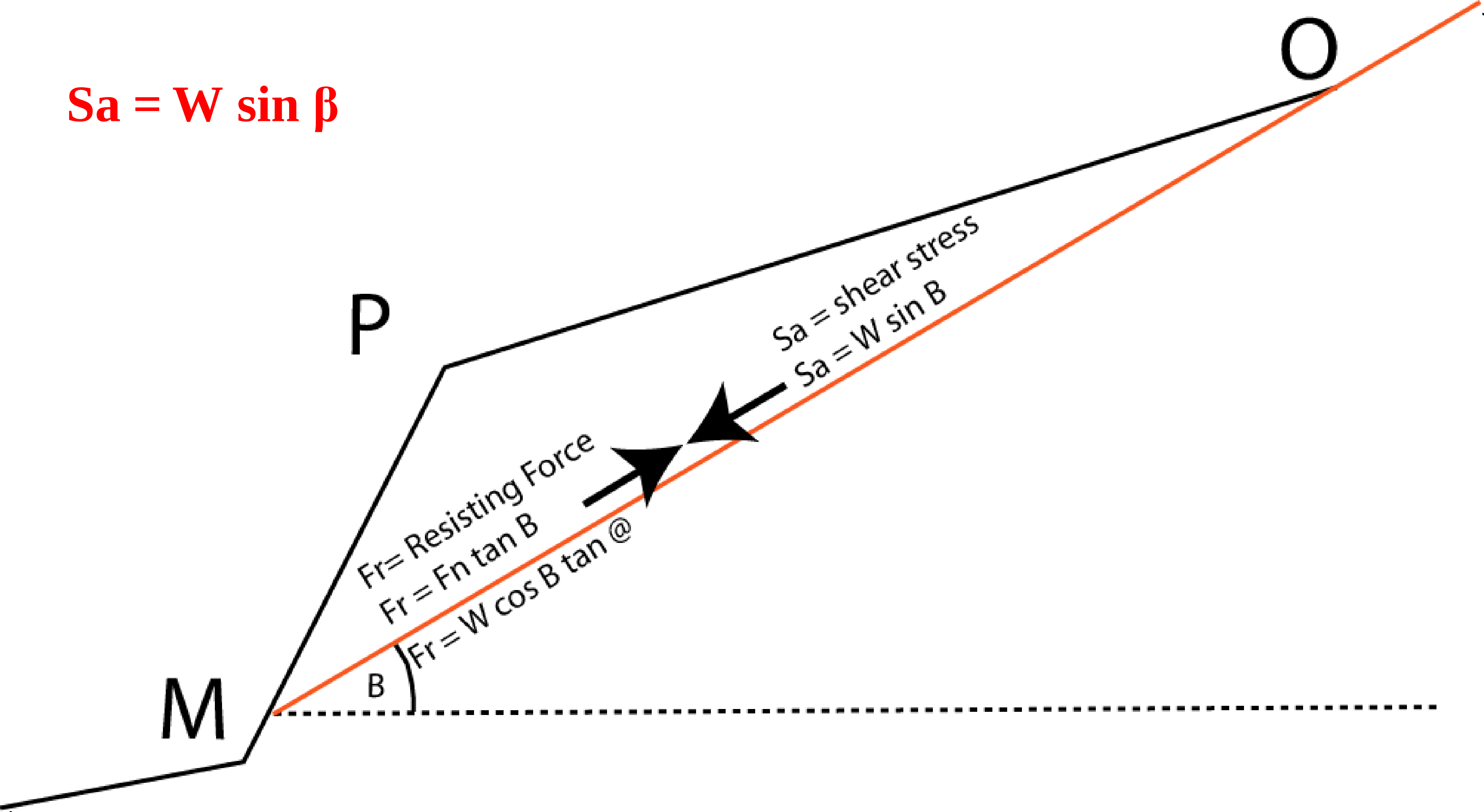
W = Fg



Fn = Fg cos **β**

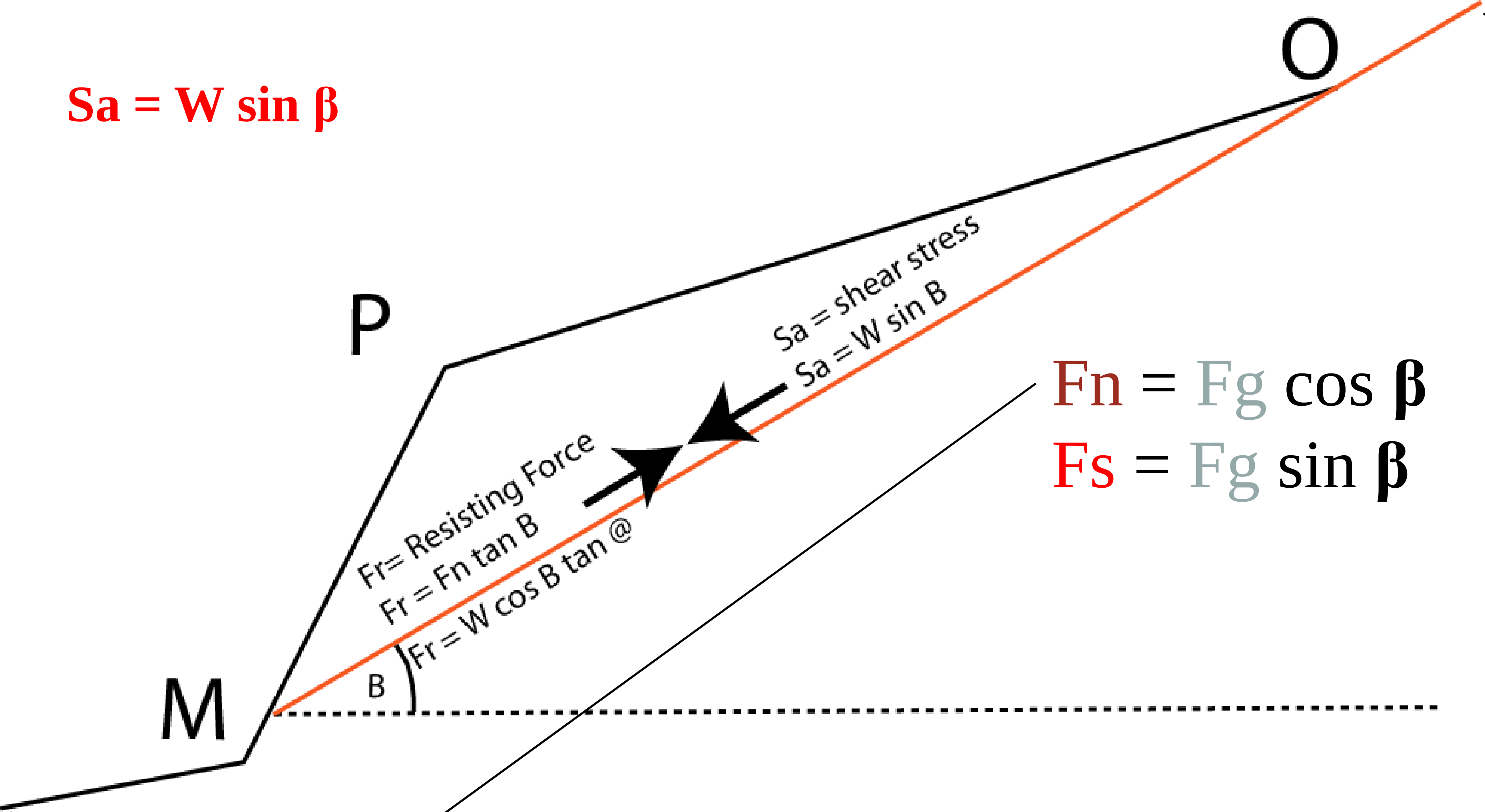
Fs = Fg sin **β**

Sa = W sin β



Sr = Shear Resistance = (Friction + Cohesion)

$$Sa = W \sin \beta$$



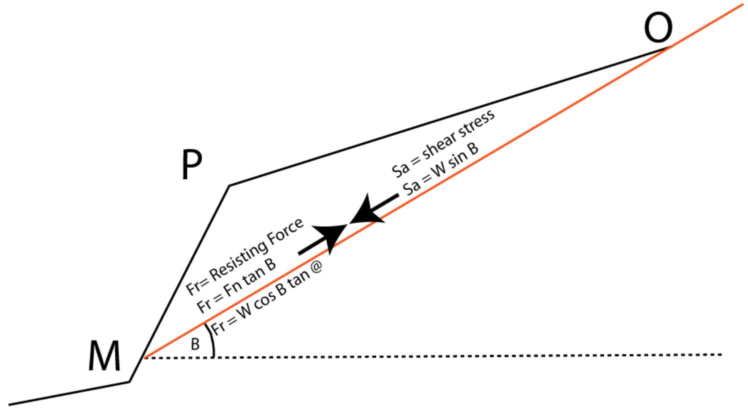
$$Fn = Fg \cos \beta$$

$$Fs = Fg \sin \beta$$

$$Sr = \text{Friction} + \text{Cohesion}$$

$$= W \cos \beta \tan \phi + c * (\text{segment } MO)$$

$$Sr = W \cos \beta \tan \phi + cL$$



Factor of Safety

$$Fs = \frac{Sr}{Sa} = \frac{W \cos \beta \tan \phi + cL}{W \sin \beta}$$

An Example…..

Factor of Safety

- Slope of 23 degrees
- Angle of internal friction of 30 degrees
- Cohesion of 90 lbs/ft2
- Soil is 100lbs/ft3
- MO has a distance of 100 ft
- PR has a distance of 22 ft

$$Fs = \frac{Sr}{Sa} = \frac{W \cos \beta \tan \phi + cL}{W \sin \beta}$$

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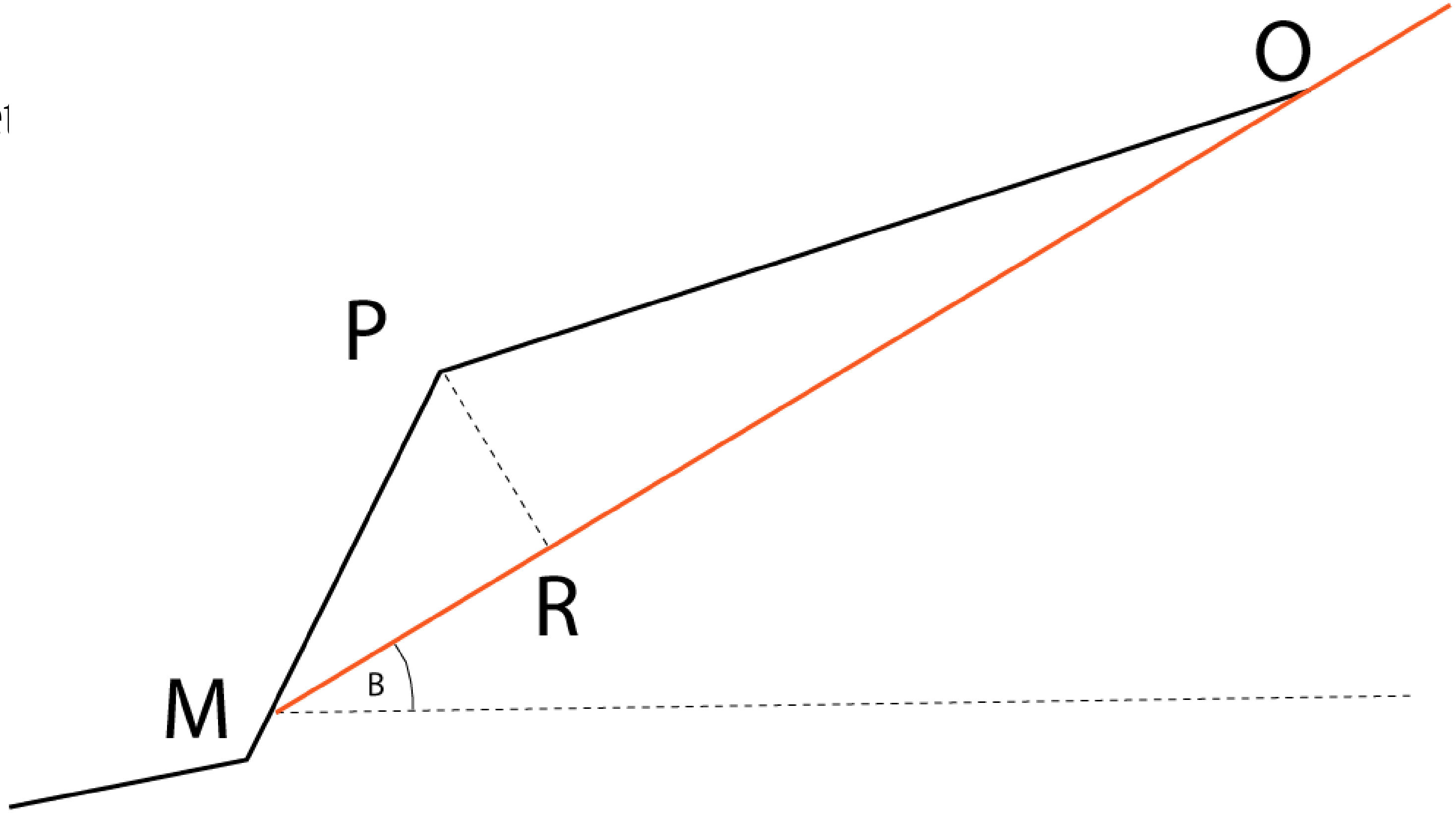

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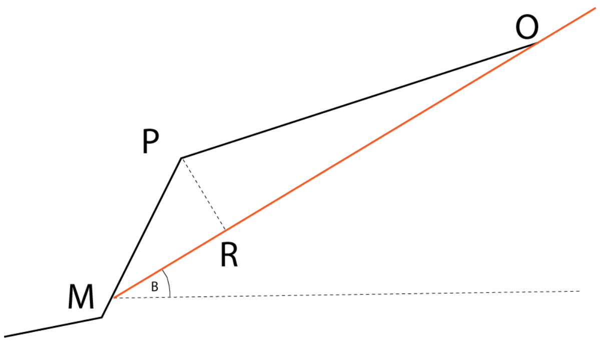

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Wt. of Slice

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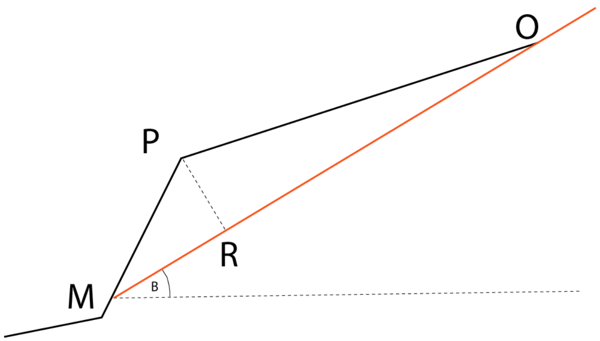

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Wt. of Slice

Vol = 100 ft × 22 ft × .5 × 1 ft = 1100 ft^3

Wt = 1100 ft^3 × 100 lbs/ft^3

= 110,000 lbs

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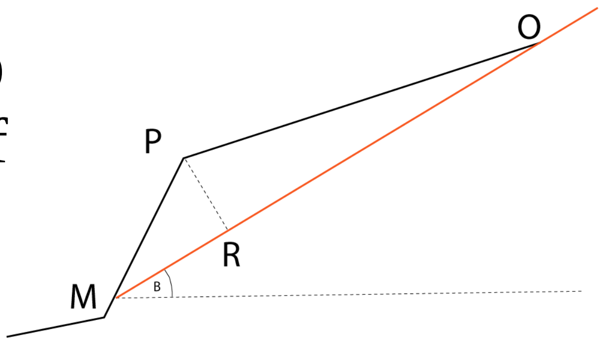

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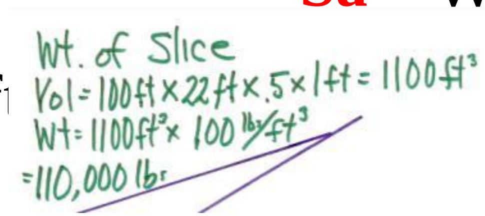

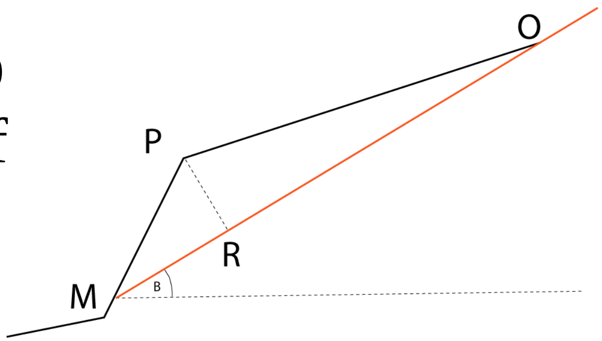

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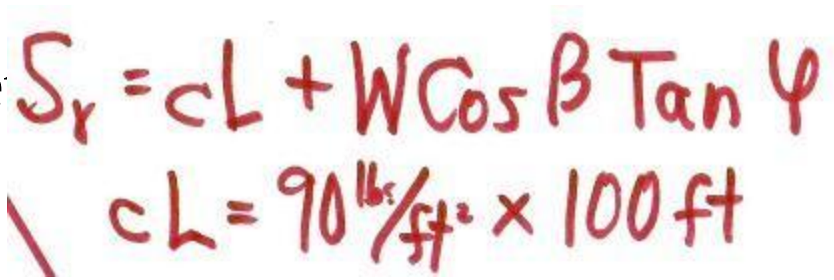

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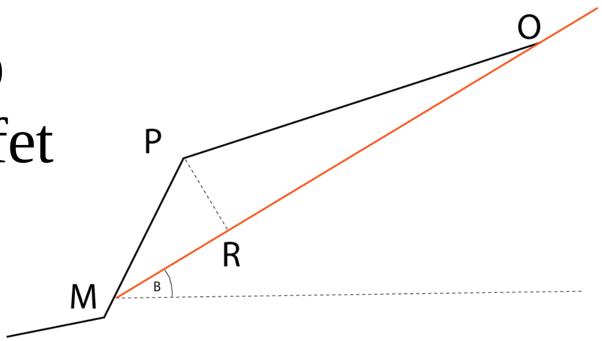


$S_r = cL + W \cos \beta \tan \varphi$

$cL = 90\ \text{lbs}/\text{ft}^2 \times 100\ \text{ft}$

$W \cos \beta \tan \varphi =$

$110{,}000\ \text{lbs} \times .92 \times .58$

$= 67{,}696\ \text{lbs}$

An Example…..

•Slope of 23 degrees

•Angle of internal friction of 30 degrees

•Cohesion of 90 lbs/ft2

•Soil is 100lbs/ft3

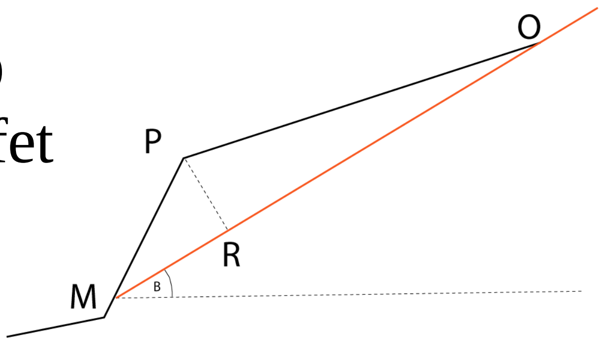
•MO has a distance of 100 ft

•PR has a distance of 22 ft

Factor of Safety

Fs = Sr = W cos βtan ϕ + cL

Sa = W sin β



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Wt. of Slice

$Vol = 100 ft \times 22 ft \times .5 \times 1 ft = 1100 ft^3$

$Wt = 1100 ft^3 \times 100 \text{ lbs}/ft^3$

$= 110,000 \text{ lbs}$

$S_r = cL + W \cos\beta \tan\varphi$

$cL = 90 \text{ lbs}/ft^2 \times 100 ft$

$W \cos\beta \tan\varphi =$

$110,000 \text{ lbs} \times .92 \times .58$

$= 67,696 \text{ lbs}$

$S_a = W \sin\beta$

An Example…..

- Slope of 23 degrees
- Angle of internal friction of 30 degrees
- Cohesion of 90 lbs/ft2
- Soil is 100lbs/ft3
- MO has a distance of 100 ft
- PR has a distance of 22 ft

Factor of Safety

$$Fs = \frac{Sr}{Sa} = \frac{W \cos \beta \tan \phi + cL}{W \sin \beta}$$

Wt. of Slice

$Vol = 100\,ft \times 22\,ft \times .5 \times 1\,ft = 1100\,ft^3$

$Wt = 1100\,ft^3 \times 100\,lbs/ft^3$

$= 110{,}000\,lbs$

$S_r = cL + W \cos \beta \tan \varphi$

$cL = 90\,lb/ft^2 \times 100\,ft$

$W \cos \beta \tan \varphi =$

$110{,}000\,lbs \times .92 \times .58$

$= 67{,}696\,lbs$

$S_a = W \sin \beta$

$= 110{,}000\,lbs \times .39$

$= 42{,}900\,lbs$

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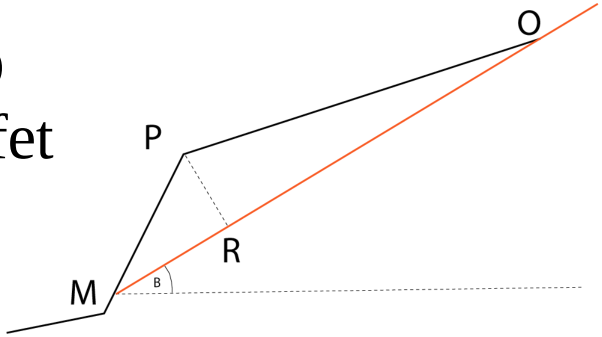

An Example…..

•Slope of 23 degrees

•Angle of internal friction of 30 degrees

•Cohesion of 90 lbs/ft2

•Soil is 100lbs/ft3

•MO has a distance of 100 ft

•PR has a distance of 22 ft

Factor of Safety

$$Fs = \frac{\color{green}{Sr}}{\color{red}{Sa}} = \frac{W \cos \beta \tan \phi + cL}{W \sin \beta}$$

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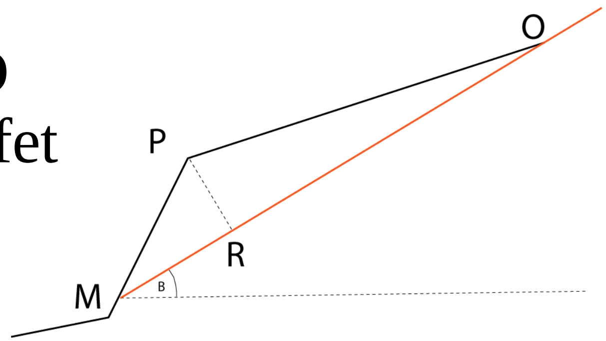

Wt. of Slice

Vol = 100ft × 22ft × .5 × 1ft = 1100ft^3

Wt = 1100ft^3 × 100 lbs/ft^3

= 110,000 lbs

$S_r = cL + W \cos \beta \tan \varphi$

cL = 90 lbs/ft^2 × 100 ft

$W \cos \beta \tan \varphi$ =

110,000 lbs × .92 × .58

= 67,696 lbs

$S_a = W \sin \beta$

= 110,000 lbs × .39

= 42,900 lbs

Factor of Safty

$\frac{S_r}{S_a} =$

An Example…..

- Slope of 23 degrees
- Angle of internal friction of 30 degrees
- Cohesion of 90 lbs/ft2
- Soil is 100lbs/ft3
- MO has a distance of 100 ft
- PR has a distance of 22 ft

Factor of Safety

$$Fs = \frac{Sr}{Sa} = \frac{W \cos \beta \tan \phi + cL}{W \sin \beta}$$

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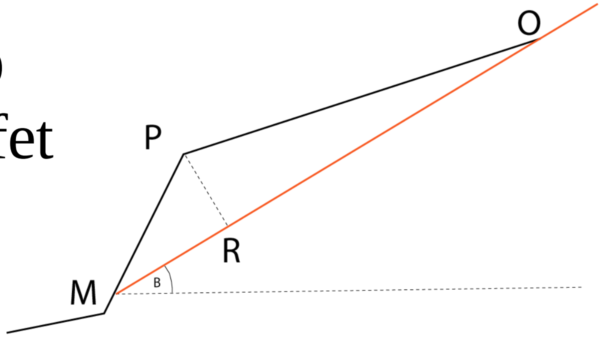
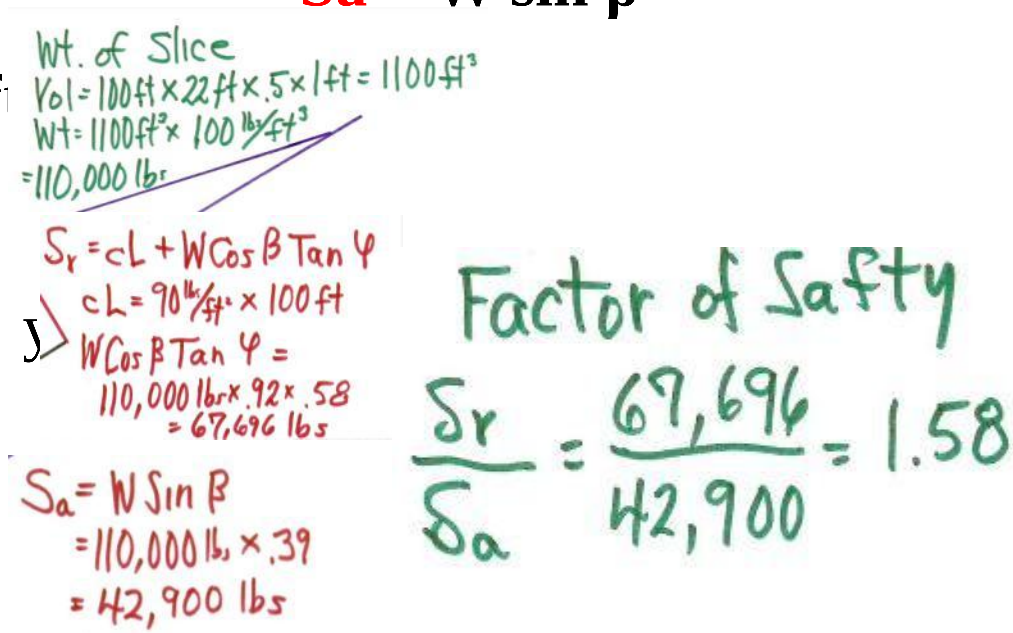
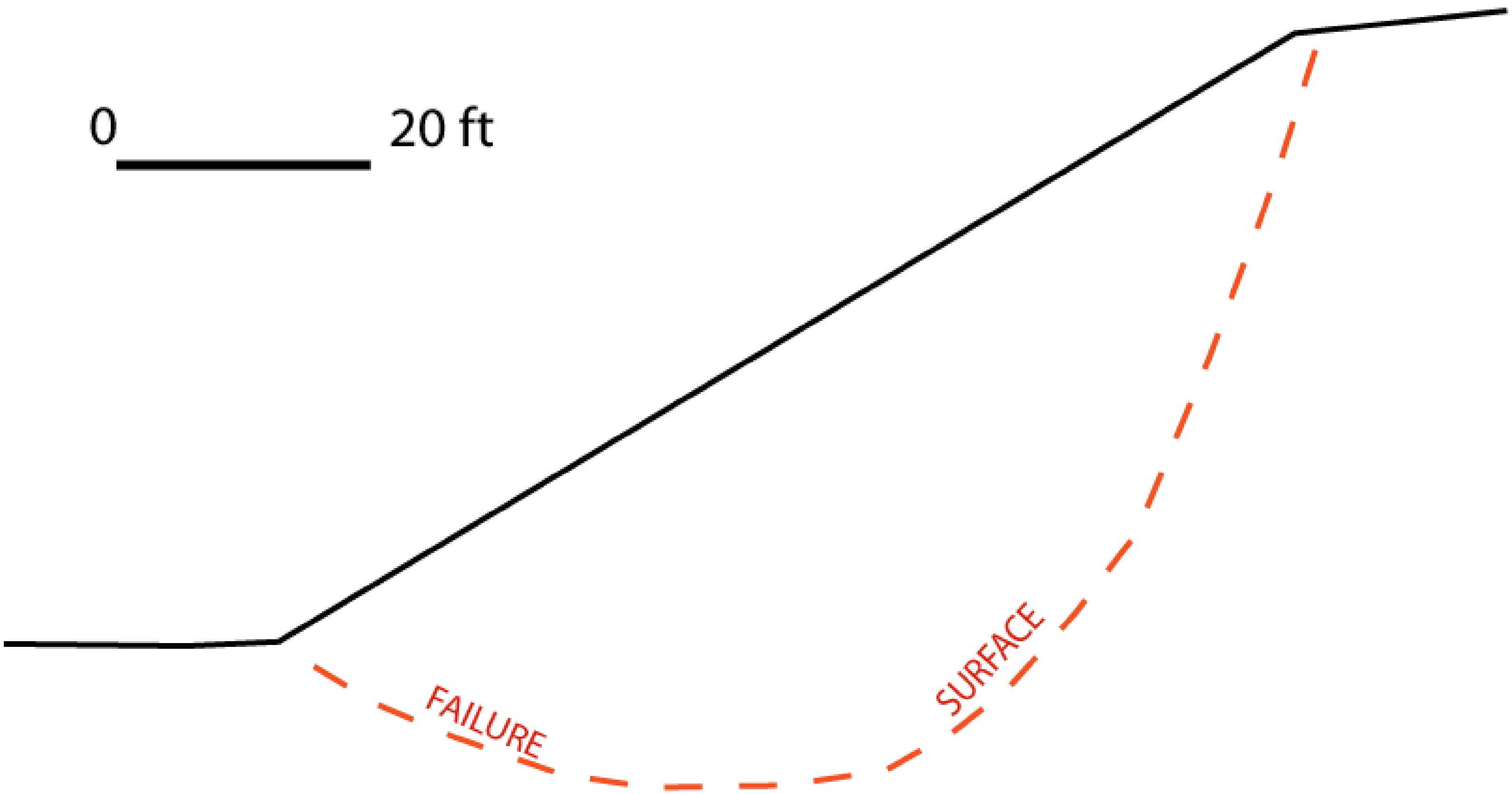




III. Rotational Slide—case 1



EARTH PRESSURETHEORIES: Rankine's theory of earth pressure, earth pressures in layered soils, Coulomb's earth pressure theory, Culmann's graphical method.

Introduction

- Earth Pressure
  - The force which is on the retaining wall when the soil is retained at a slope steeper than it can sustain by virtue of its shearing strength.
  - The magnitude of earth pressure is a function of the magnitude and nature of the absolute and relative movements of the soil and the structure.

LATERAL EARTH PRESSURES

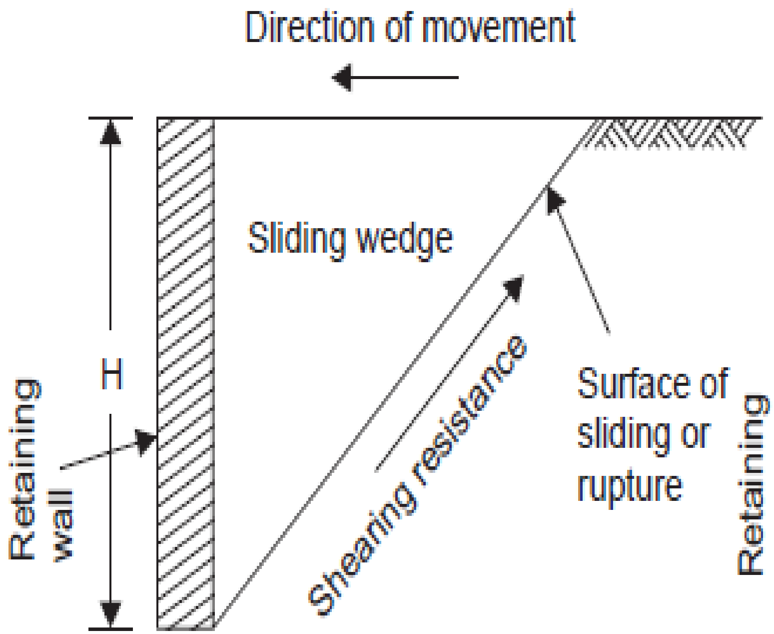


Fig. Conditions in the case of active earth pressure

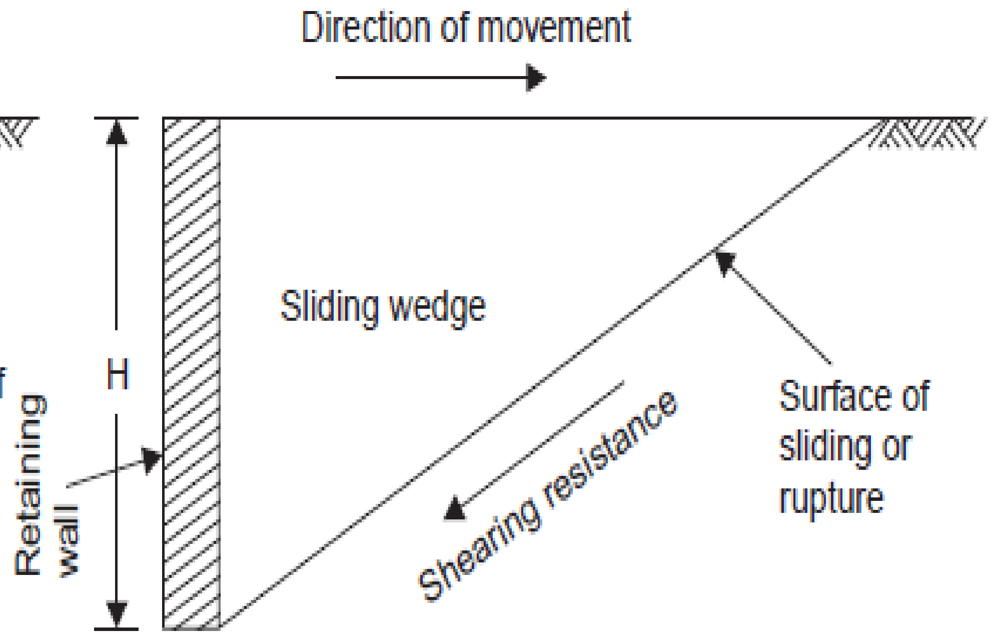
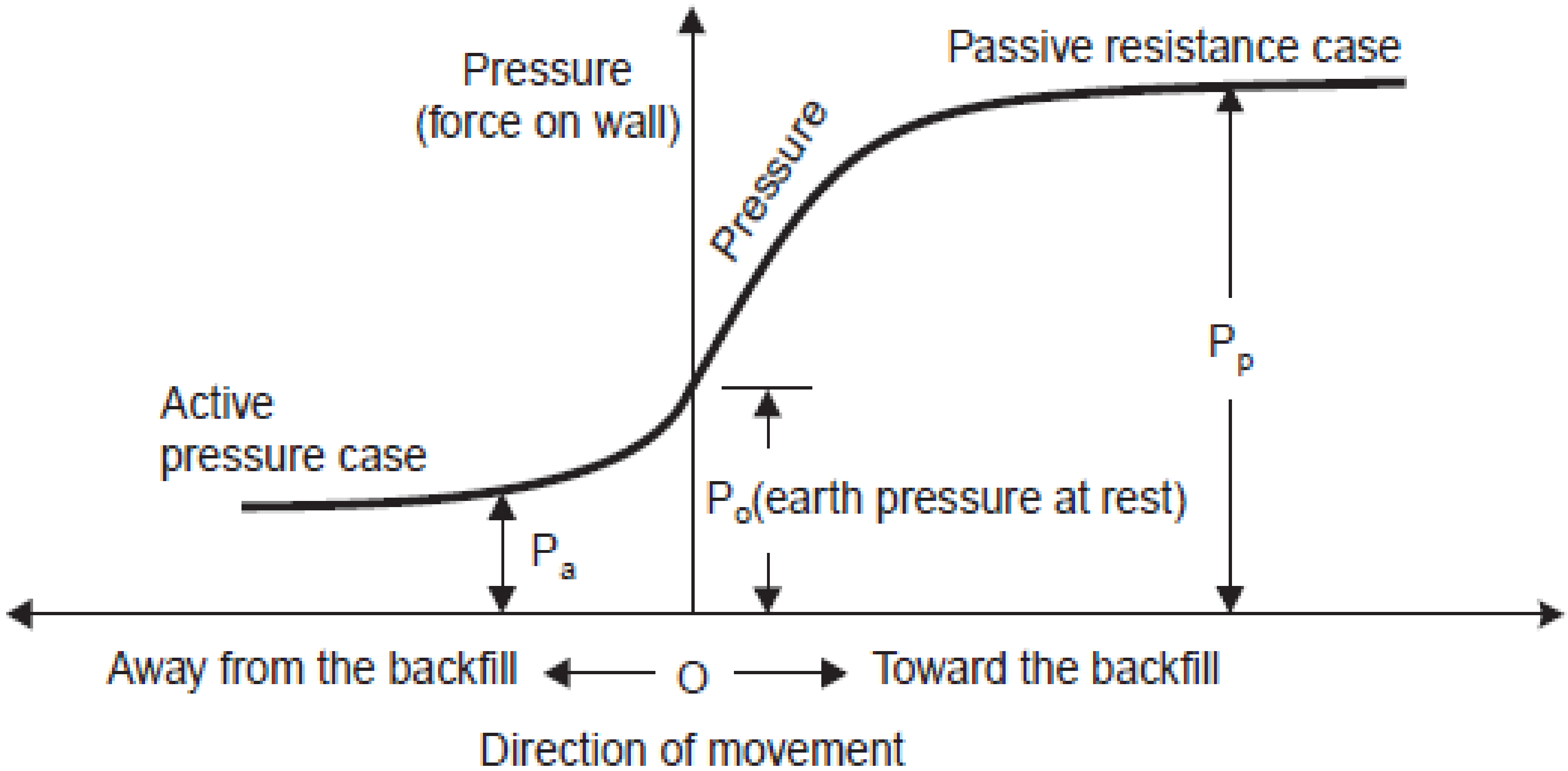


Fig. 13.3 Conditions in the case of passive earth resistance

Effect of Wall Movement on Earth Pressure



Effect of Wall Movement on Earth Pressure

- The Earth Pressure At Rest
  - The earth pressure that the soil mass is in a state of rest and there are no deformations and displacements.

Earth Pressure At Rest

Table 12.1 Typical Values of K_o

Soil	K_o
Dense sand	0.40-0.45
Loose sand	0.45 - 0.SO
Mechanically compacted sand	0.8 - 1.5
Normally consolidated clay	0.5-0.6
Overconsolidated clay	1.0-4.0

Rankine's Theory of Earth Pressure

- Assumptions:
  - The backfill soil is isotropic, homogeneous and is cohesionless.
  - The soil is in a state of plastic equilibrium during active and passive earth pressure conditions.
  - The rupture surface is a planar surface which is obtained by considering the plastic equilibrium of the soil.
  - The backfill surface is horizontal.
  - The back of the wall is vertical.
  - The back of the wall is smooth.

Active Earth Pressure of Cohesion less Soil

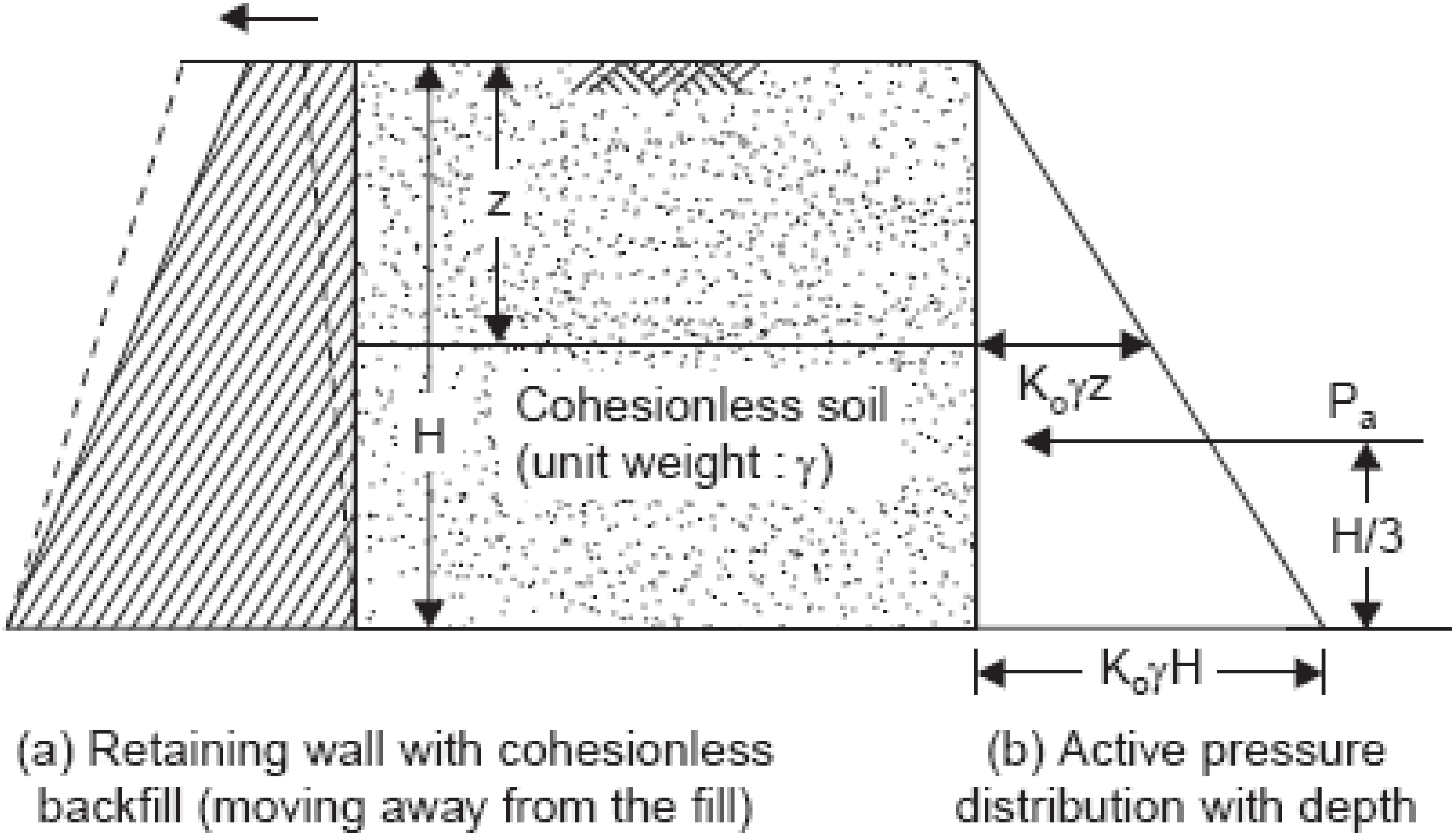


Fig. Active earth pressure distribution – Rankine's theory

σ_0 at a depth z belcw the surface = $\gamma.z$

Assuming that the wall yields sufficiently for the active conditions to develop,

$\sigma_a = K_a \gamma.z$

where $K_a = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2(45° - \phi/2)$

For a total height of H of the wall, the total thrust P_a on the wall per unit length of the wall, is given by

$$P_a = \frac{1}{2} K_a \gamma H^2 \quad \text{(Eq 13.10)}$$

This may be taken to act at a height of (1/3)H abeve the base as shewn, through the rentroid of the pressure distribution diagram

Effect of Submergence

(i) Lateral earth pressure due to submerged unit weight of the backfill soil; and
(ii) Lateral pressure due to pore water.

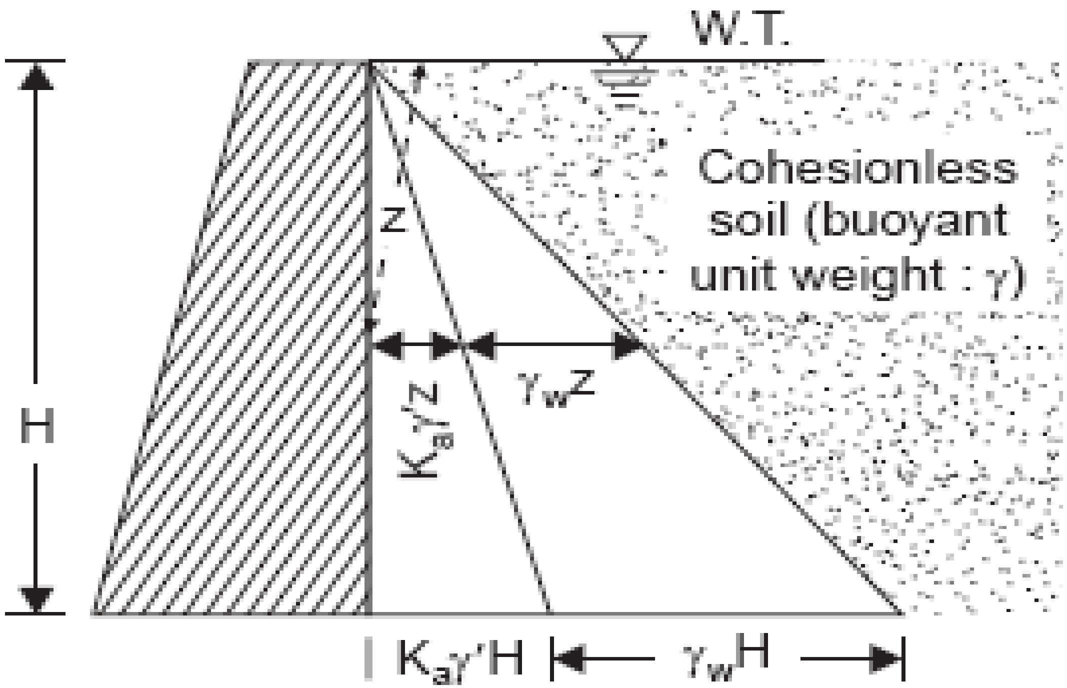
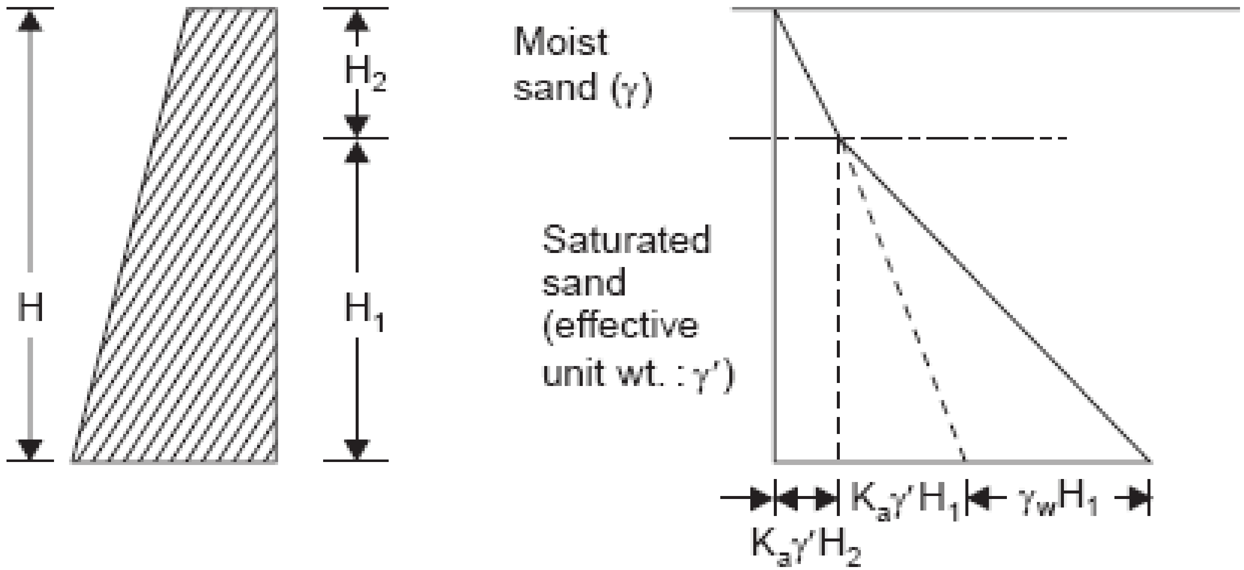


Fig. Effect of submergence on lateral earth pressure

At a depth H below the surface, the lateral pressure, σh, is given by : $\sigma h = Ka.\ \gamma' H + \gamma w.\ H$

Effect of partial submergence



(a) Partly submerged backfill

(b) Lateral pressure for partly submerged backfill

Fig. Effect of partial submergence on lateral earth pressure

The lateral pressure above the water table is due to the most unit weight of soil, and that below the water table is the sum of that due to the submerged unit weight of the soil and the water pressure.

- where H1 = depth of submerged fill,
- Ka = active earth pressure coefficient,
- H2 = depth of fill above water table (taken to be moist),
- γ = moist unit weight, and
- γ ′ = submerged or effective unit weight.

Lateral pressure at the base of wall,

$= K_a \gamma H_2 + K_a \gamma' H_1 + \gamma_w H_1$

Effect of Uniform Surcharge

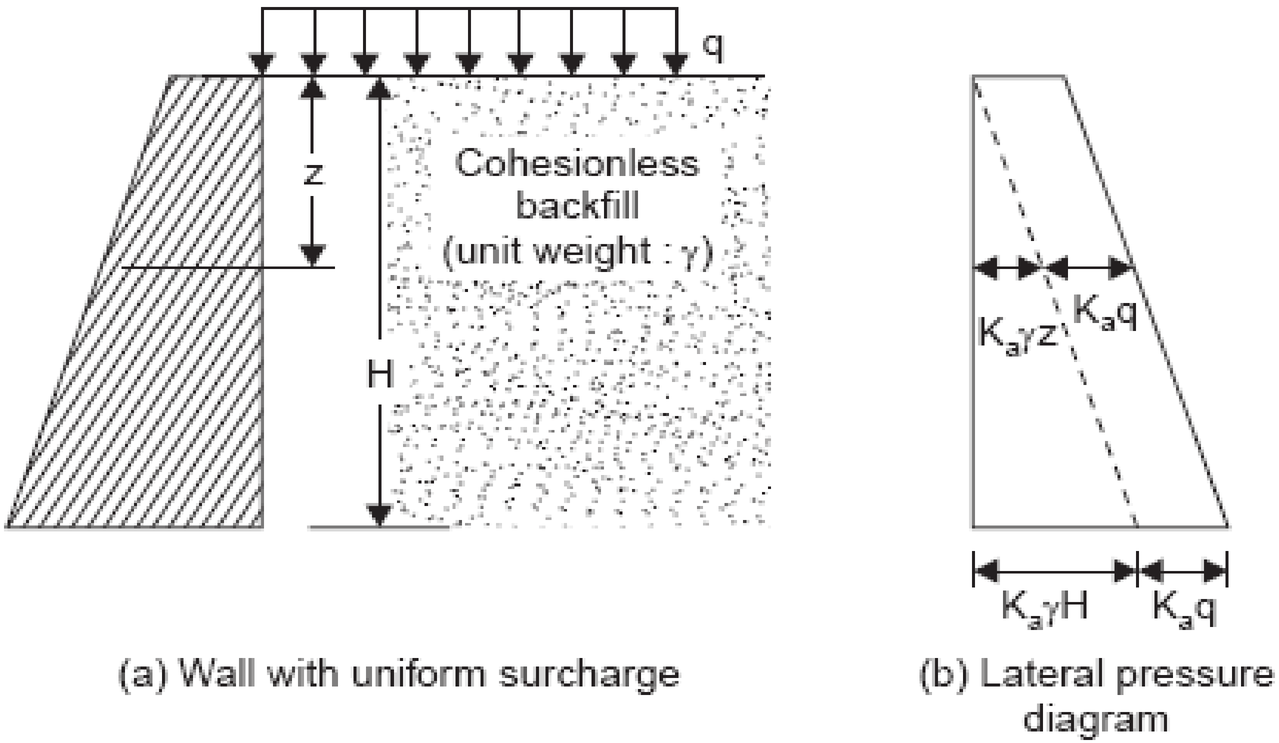


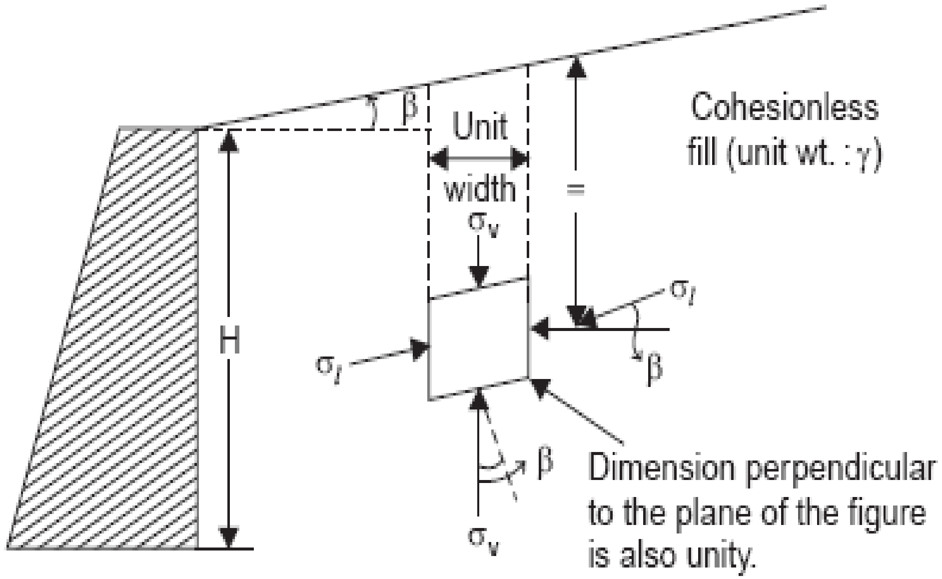
Fig. Effect of uniform surcharge on lateral pressure

- The extra loading carried by a retaining structure is known as 'surcharge'. It may be a uniform load (from roadway, from stacked goods, etc.), a line load (trains running parallel to the structure), or an isolated load (say, a column footing).

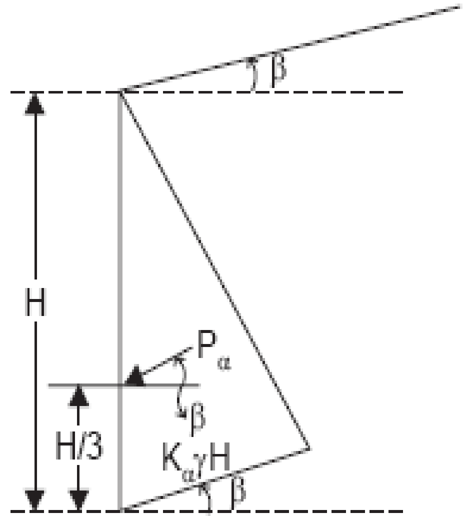
- In the case of a wall retaining a backfill with horizontal surface level with the top of the wall and carrying a uniform surcharge of intensity q per unit area, the vertical stress at every elevation in the backfill is considered to increase by q. As such, the lateral pressure has to increase by Ka.q.

- Thus, at any depth z, $\sigma h = Ka\gamma.z + Kaq$

Effect of Inclined Surcharge—Sloping Backfill



(a) Conjugate stresses on an element



(c) Active pressure distribution

$$K_a = \cos\beta \left(\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right)$$

The total active thrust *Pa per unit* length of the wall acts at (1/3)*H above the base of the wall and is equal to 1/2 Kaɣ.H^2; it acts* parallel to the surface of the fill.

Active Earth Pressure of Cohesive Soil

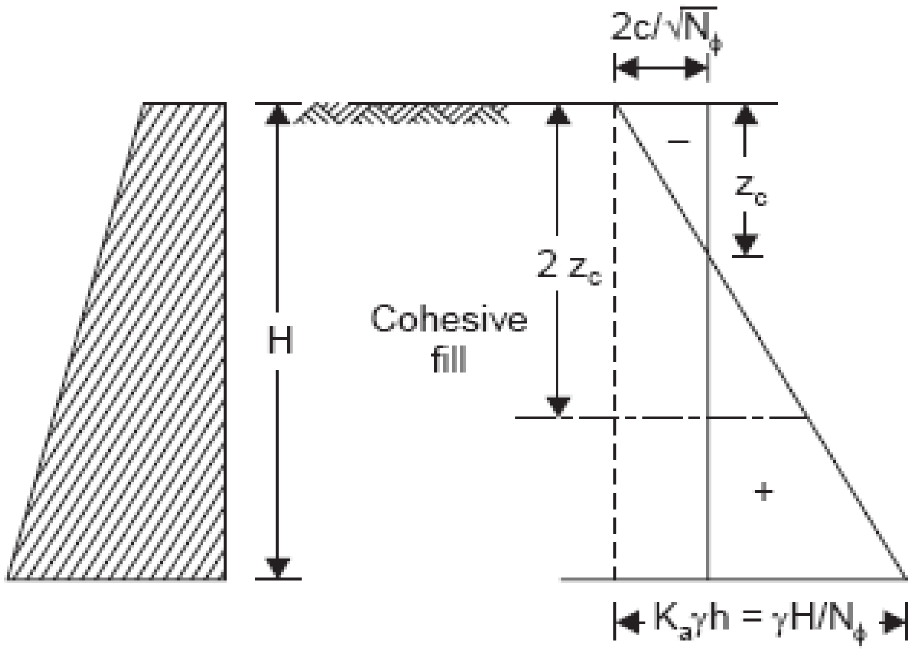


Fig. Active pressure distribution for a cohesive soil

For c- ϕ soil

$$P_a = \frac{\gamma H^2}{2N_\phi} - \frac{2cH}{\sqrt{N_\phi}} + \frac{2c^2}{\gamma}$$

For pure clay, ϕ = 0

$$P_a = \frac{1}{2}\gamma H^2 - 2cH + \frac{2c^2}{\gamma}$$

Passive Earth Pressure of Cohesive Soil

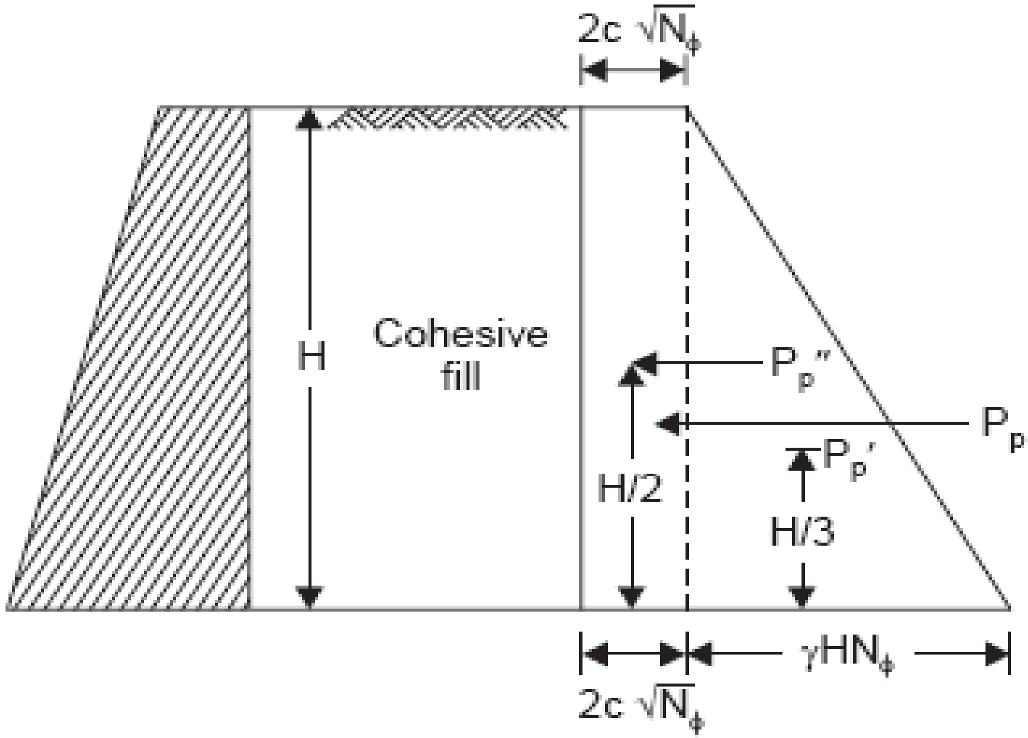


Fig. Passive pressure distribution for the cohesive soil

$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

$$\sigma_3 = \gamma z \quad \text{and} \quad \sigma_1 = \sigma_{h_c}$$

$$\sigma_{1_c} = \sigma_{h_c} = \gamma z N_\phi + 2c\sqrt{N_\phi}$$

(Here, $K_P = N_\phi$ in the usual notation).

The pressure distribution with depth is shown in Fig.

The total passive resistance per unit length of wall is $P_p = P_{p_1} + P_{p_2} = \frac{1}{2}\gamma H^2 N_\phi + 2cH\sqrt{N_\phi}$.

P_{p_1} acts at $H/3$ and P_{p_2} acts at $H/2$ above the base. The location of P_p may be found by taking moments about the base.

Coulomb's Theory of Earth Pressure

- Assumptions;
  - The backfill is a dry, cohesionless, homogeneous, isotropic soil.
  - The backfill surface is planar and can be inclined.
  - The back of the wall can be inclined to the vertical.
  - The failure surface is a plane surface which passes through the heel of the wall.
  - The position and the line of action of the earth pressure are known.
  - The sliding wedge is considered to be a rigid body and the earth pressure is obtained by considering the limiting equilibrium of the sliding wedge as a whole.

Coulomb's Theory of Earth Pressure

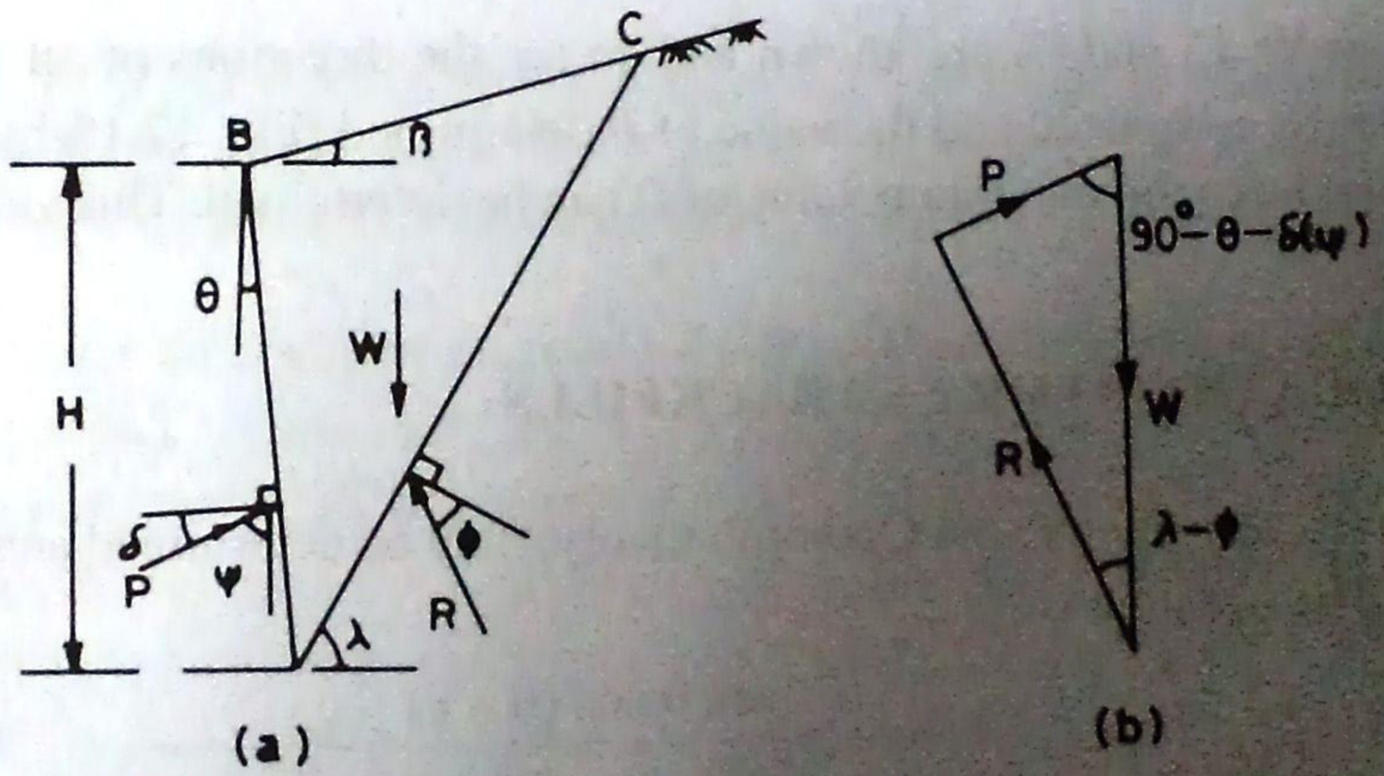


Fig. 12.17 Coulomb's theory – Active case; $c = 0$ backfill

Coulomb’s Theory of Earth Pressure

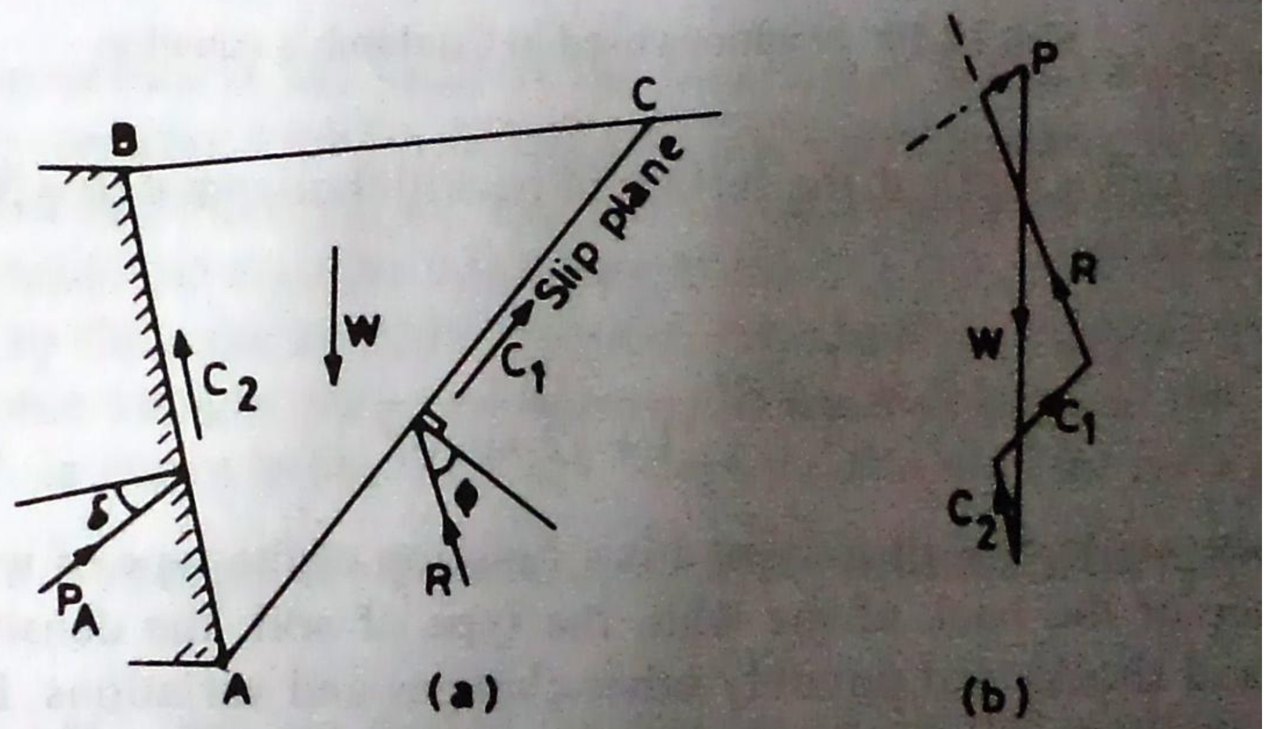


Fig. 12.18 Coulomb’s theory – Active case: $c - \phi$ backfill

Coulomb Equations for c=0 Backfills

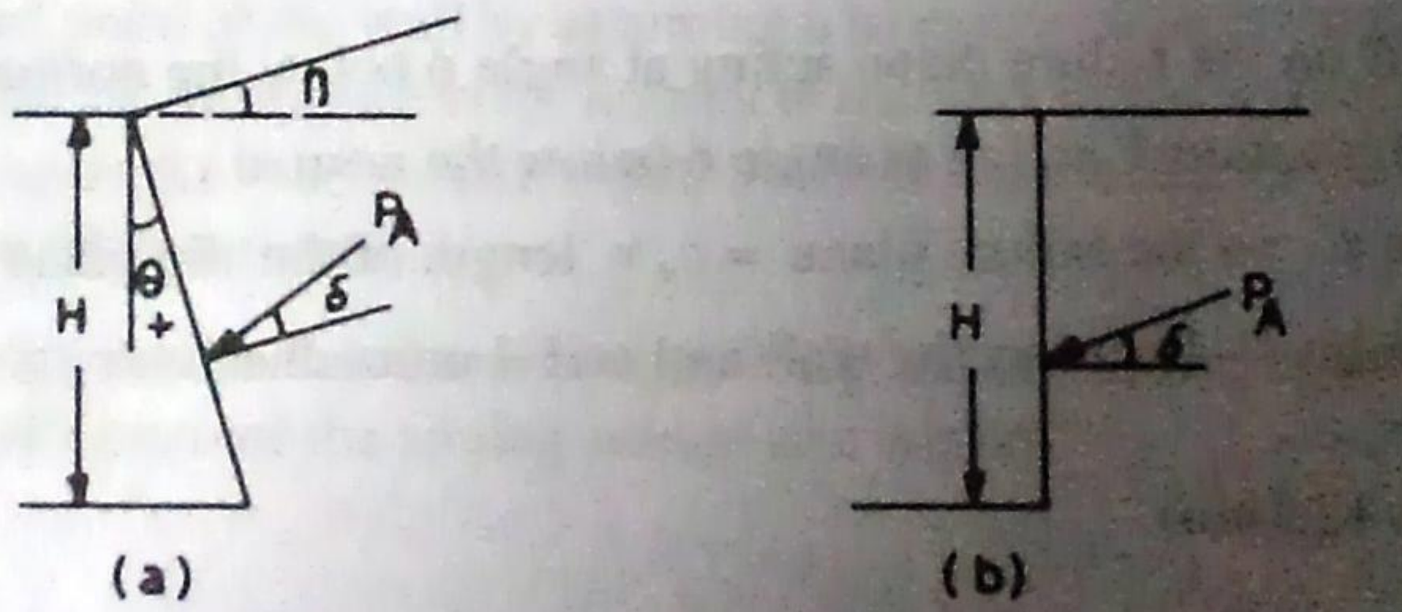


Fig. 12.19 Notations used in Coulomb's equation

TABLE 12.1 - Rankine Active Earth Pressure coefficient, K_a

α (deg)	ϕ (deg) 28	30	32	34	36	38	40
0	0.361	0.333	0.307	0.283	0.260	0.238	0.217
5	0.366	0.337	0.311	0.286	0.262	0.240	0.219
10	0.380	0.350	0.321	0.294	0.270	0.246	0.225
15	0.409	0.373	0.341	0.311	0.283	0.258	0.235
20	0.461	0.414	0.374	0.338	0.306	0.277	0.250
25	0.573	0.494	0.434	0.385	0.343	0.307	0.275

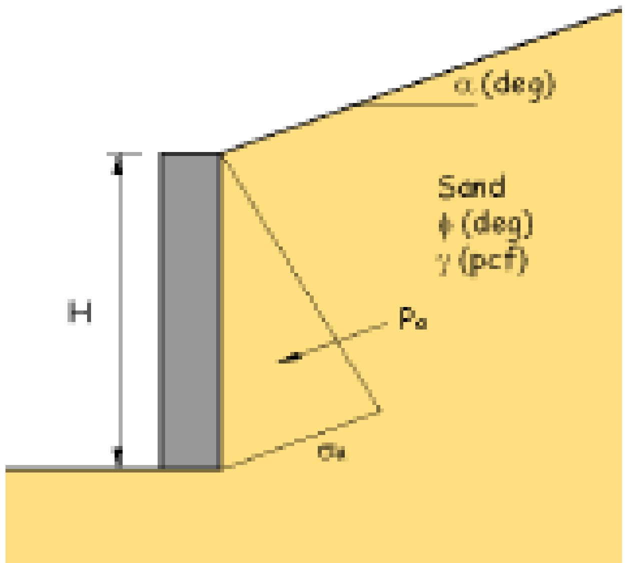


TABLE 12.2 - Coulomb's Active Earth Pressure coefficient, K_a

Note: $\delta = 2/3\ \phi$ (assumed design value of wall friction angle, δ)

α (deg)	φ (deg)	β (deg) 90	85	80	75	70	65
0	28	0.3213	0.3588	0.4007	0.4481	0.5026	0.5662
	29	0.3091	0.3467	0.3886	0.4362	0.4908	0.5547
	30	0.2973	0.3349	0.3769	0.4245	0.4794	0.5435
	31	0.2860	0.3235	0.3655	0.4133	0.4682	0.5326
	32	0.2750	0.3125	0.3545	0.4023	0.4574	0.5220
	33	0.2645	0.3019	0.3439	0.3917	0.4469	0.5117
	34	0.2543	0.2916	0.3335	0.3813	0.4367	0.5017
	35	0.2444	0.2816	0.3235	0.3713	0.4267	0.4919
	36	0.2349	0.2719	0.3137	0.3615	0.4170	0.4824
	37	0.2257	0.2626	0.3042	0.3520	0.4075	0.4732
	38	0.2168	0.2535	0.2950	0.3427	0.3983	0.4641
	39	0.2082	0.2447	0.2861	0.3337	0.3894	0.4553
	40	0.1998	0.2361	0.2774	0.3249	0.3806	0.4468
	41	0.1918	0.2278	0.2689	0.3164	0.3721	0.4384
	42	0.1840	0.2197	0.2606	0.3080	0.3637	0.4302

α (deg)	φ (deg)	β (deg) 90	85	80	75	70	65
10	28	0.3702	0.4164	0.4686	0.5287	0.5992	0.6834
	29	0.3548	0.4007	0.4528	0.5128	0.5831	0.6672
	30	0.3400	0.3857	0.4376	0.4974	0.5676	0.6516
	31	0.3259	0.3713	0.4230	0.4826	0.5526	0.6365
	32	0.3123	0.3575	0.4089	0.4683	0.5382	0.6219
	33	0.2993	0.3442	0.3953	0.4545	0.5242	0.6078
	34	0.2868	0.3314	0.3822	0.4412	0.5107	0.5942
	35	0.2748	0.3190	0.3696	0.4283	0.4976	0.5810
	36	0.2633	0.3072	0.3574	0.4158	0.4849	0.5682
	37	0.2522	0.2957	0.3456	0.4037	0.4726	0.5558
	38	0.2415	0.2846	0.3342	0.3920	0.4607	0.5437
	39	0.2313	0.2740	0.3231	0.3807	0.4491	0.5321
	40	0.2214	0.2636	0.3125	0.3697	0.4379	0.5207
	41	0.2119	0.2537	0.3021	0.3590	0.4270	0.5097
	42	0.2027	0.2441	0.2921	0.3487	0.4164	0.4990

α (deg)	φ (deg)	β (deg) 90	85	80	75	70	65
20	28	0.4602	0.5205	0.5900	0.6714	0.7689	0.8880
	29	0.4364	0.4958	0.5642	0.6445	0.7406	0.8581
	30	0.4142	0.4728	0.5403	0.6195	0.7144	0.8303
	31	0.3935	0.4513	0.5179	0.5961	0.6898	0.8043
	32	0.3742	0.4311	0.4968	0.5741	0.6666	0.7799
	33	0.3559	0.4121	0.4769	0.5532	0.6448	0.7569
	34	0.3388	0.3941	0.4581	0.5335	0.6241	0.7351
	35	0.3225	0.3771	0.4402	0.5148	0.6044	0.7144
	36	0.3071	0.3609	0.4233	0.4969	0.5856	0.6947
	37	0.2925	0.3455	0.4071	0.4799	0.5677	0.6759
	38	0.2787	0.3308	0.3916	0.4636	0.5506	0.6579
	39	0.2654	0.3168	0.3768	0.4480	0.5342	0.6407
	40	0.2529	0.3034	0.3626	0.4331	0.5185	0.6242
	41	0.2408	0.2906	0.3490	0.4187	0.5033	0.6083
	42	0.2294	0.2784	0.3360	0.4049	0.4888	0.5930

α (deg)	φ (deg)	β (deg) 90	85	80	75	70	65
5	28	0.3431	0.3845	0.4311	0.4843	0.5461	0.6190
	29	0.3295	0.3709	0.4175	0.4707	0.5325	0.6056
	30	0.3165	0.3578	0.4043	0.4575	0.5194	0.5926
	31	0.3039	0.3451	0.3916	0.4447	0.5067	0.5800
	32	0.2919	0.3329	0.3792	0.4324	0.4943	0.5677
	33	0.2803	0.3211	0.3673	0.4204	0.4823	0.5558
	34	0.2691	0.3097	0.3558	0.4088	0.4707	0.5443
	35	0.2583	0.2987	0.3446	0.3975	0.4594	0.5330
	36	0.2479	0.2881	0.3338	0.3866	0.4484	0.5221
	37	0.2379	0.2778	0.3233	0.3759	0.4377	0.5115
	38	0.2282	0.2679	0.3131	0.3656	0.4273	0.5012
	39	0.2188	0.2582	0.3033	0.3556	0.4172	0.4911
	40	0.2098	0.2489	0.2937	0.3458	0.4074	0.4813
	41	0.2011	0.2398	0.2844	0.3363	0.3978	0.4718
	42	0.1927	0.2311	0.2753	0.3271	0.3884	0.4625

α (deg)	φ (deg)	β (deg) 90	85	80	75	70	65
15	28	0.4065	0.4585	0.5179	0.5868	0.6685	0.7670
	29	0.3881	0.4397	0.4987	0.5672	0.6483	0.7463
	30	0.3707	0.4219	0.4804	0.5484	0.6291	0.7265
	31	0.3541	0.4049	0.4629	0.5305	0.6106	0.7076
	32	0.3384	0.3887	0.4462	0.5133	0.5930	0.6895
	33	0.3234	0.3732	0.4303	0.4968	0.5761	0.6721
	34	0.3091	0.3583	0.4150	0.4811	0.5598	0.6554
	35	0.2954	0.3442	0.4003	0.4659	0.5442	0.6393
	36	0.2823	0.3306	0.3862	0.4513	0.5291	0.6238
	37	0.2698	0.3175	0.3726	0.4373	0.5146	0.6089
	38	0.2578	0.3050	0.3595	0.4237	0.5006	0.5945
	39	0.2463	0.2929	0.3470	0.4106	0.4871	0.5805
	40	0.2353	0.2813	0.3348	0.3980	0.4740	0.5671
	41	0.2247	0.2702	0.3231	0.3858	0.4613	0.5541
	42	0.2146	0.2594	0.3118	0.3740	0.4491	0.5415

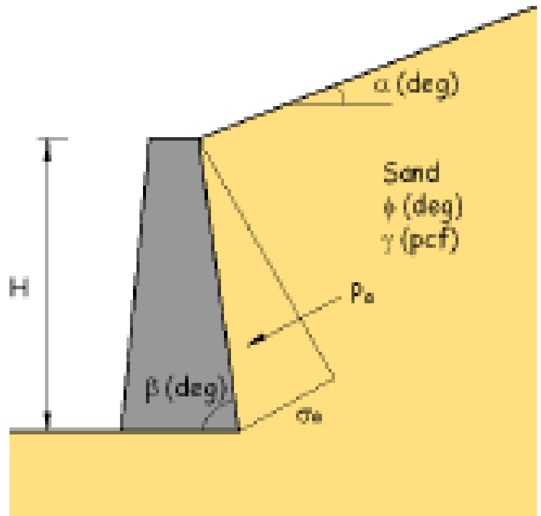


TABLE 12.3 - Rankine Passive Earth Pressure coefficient, K_p

α (deg)	ϕ (deg)						
	28	30	32	34	36	38	40
0	2.770	3.000	3.255	3.537	3.852	4.204	4.599
5	2.715	2.943	3.196	3.476	3.788	4.136	4.527
10	2.551	2.775	3.022	3.295	3.598	3.937	4.316
15	2.284	2.502	2.740	3.003	3.293	3.615	3.977
20	1.918	2.132	2.362	2.612	2.886	3.189	3.526
25	1.434	1.664	1.894	2.135	2.394	2.676	2.987

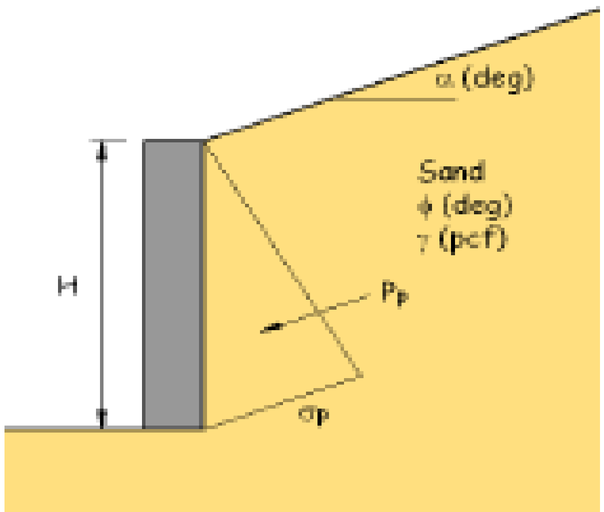
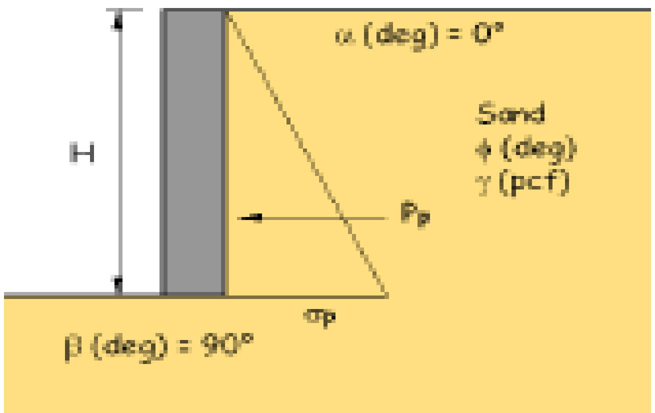


TABLE 12.4 - Coulomb's Passive Earth Pressure coefficient, K_p

Note: $\beta = 90°$ and $\alpha = 0°$

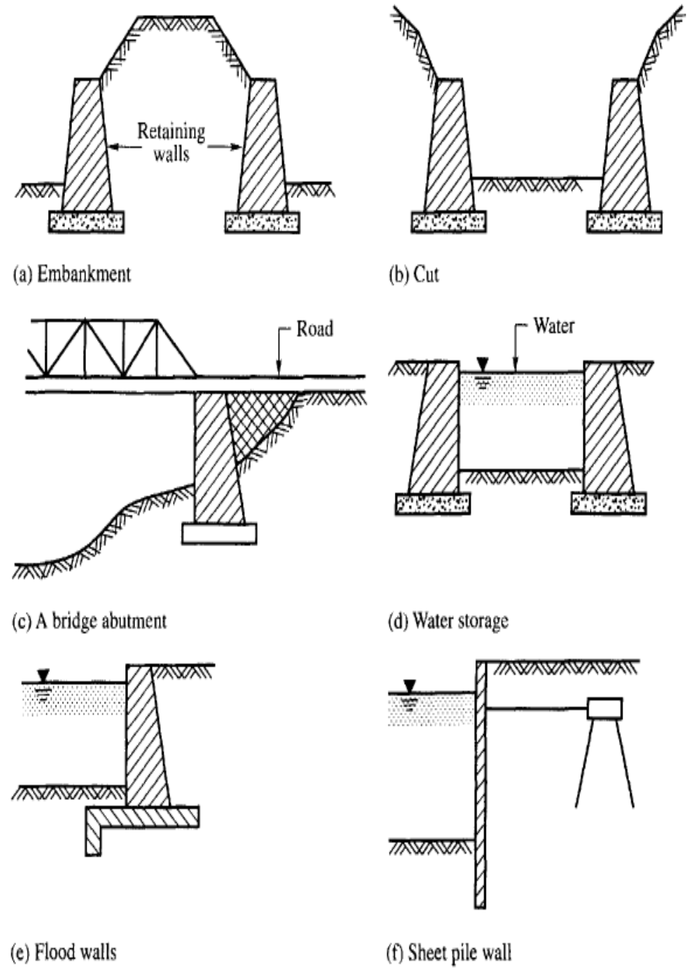
ϕ (deg)	δ (deg) 0	5	10	15	20
15	1.698	1.900	2.130	2.405	2.735
20	2.040	2.313	2.636	3.030	3.525
25	2.464	2.830	3.286	3.855	4.597
30	3.000	3.506	4.143	4.977	6.105
35	3.690	4.390	5.310	6.854	8.324
40	4.600	5.590	6.946	8.870	11.772

δ = wall friction angle

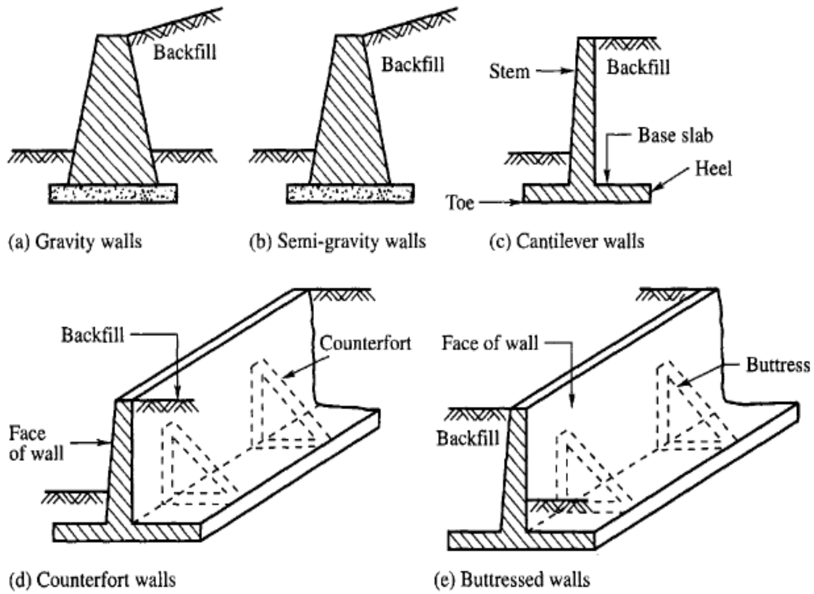


Retaining Walls and Lateral Pressure

- Structures that are built to retain vertical or nearly vertical earth banks or any other material are called retaining walls. Retaining walls may be constructed of masonry or sheet piles.
- All the walls listed in Fig have to withstand lateral pressures either from earth or any other material on their faces.
- Gravity walls resist movement because of their heavy sections. They are built of mass concrete or stone or brick masonry.
- In all these cases, the backfill tries to move the wall from its position. The movement of the wall is partly resisted by the wall itself and partly by soil in front of the wall.



Various types of Retaining walls

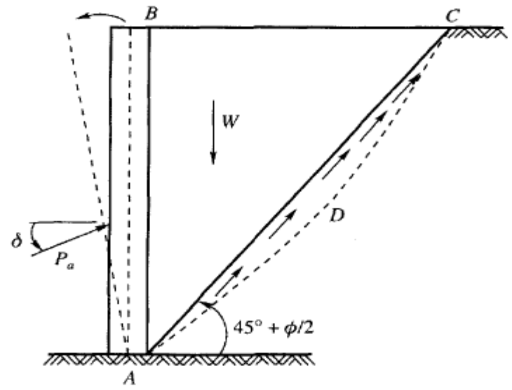


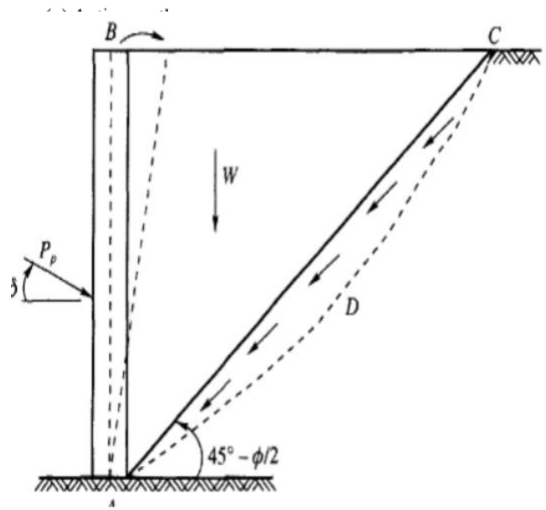
Principal types of rigid retaining walls

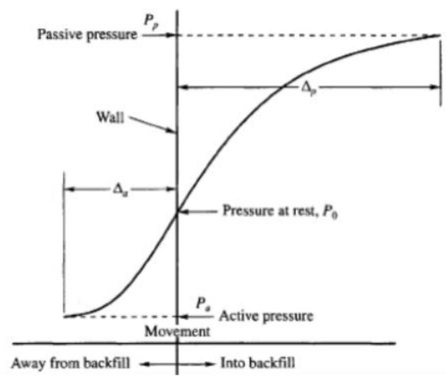
Type	Backfill material
1	Coarse-grained soil without admixture of fine soil particles, very permeable (clean sand or gravel)
2	Coarse-grained soil of low permeability due to admixture of particles of silt size
3	Residual soil with stones fine silty sand, and granular materials with conspicuous clay content
4	Very soft or soft clay, organic silts, or silty clays
5	Medium or stiff clay

Active Earth Pressure

- A retaining wall backfilled with cohesion less soil shown in Fig. If the wall does not move even after back filling, the pressure exerted on the wall is termed as pressure for the *at rest condition of the wall*
- If suppose the wall gradually rotates about point *A and moves away from the backfill, the unit pressure on the wall is* gradually reduced and after a particular displacement of the wall at the top, the pressure reaches a constant value. The pressure is the minimum possible.
- This pressure is termed the *active pressure* since the weight of the backfill is responsible for the movement of the wall.
- If the wall surface is smooth, the resultant pressure acts normal to the face of the wall. If the wall is rough, it makes an angle б with the normal on the wall. The angle б *is called the angle of wall friction. As the wall moves away* from the backfill, the soil tends to move forward.

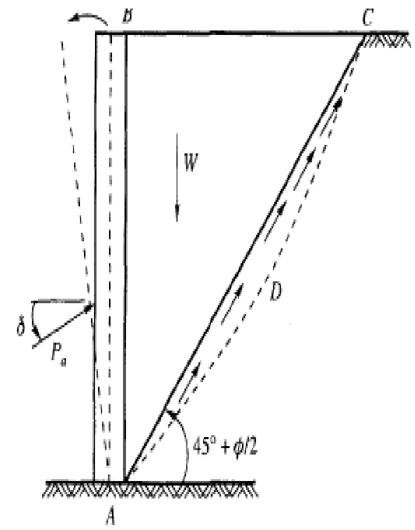




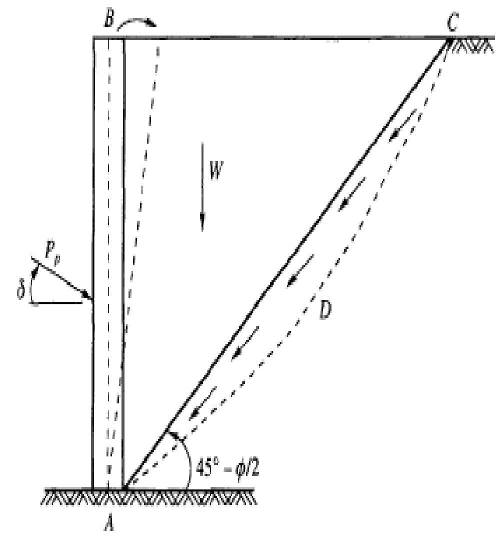


Active and Passive lateral pressure

- When the wall movement is sufficient, a soil mass of weight *W ruptures along surface ADC shown in Fig (a).* This surface is slightly curved. If the surface is assumed to be a plane surface *AC, analysis would indicate that this surface would* make an angle of $45° + \phi/2$ with the horizontal.
- If the wall is now rotated about *A towards the backfill, the actual failure plane ADC is also a* curved surface *Fig (b).*
- However, if the failure surface is approximated as a plane *AC,* this makes an angle $45° - \phi/2$ with the horizontal and the pressure on the wall increases from the value of the at rest condition to the maximum value possible.
- The maximum pressure *P that is developed* is termed the *passive earth pressure. The pressure is called passive because the weight of the* backfill opposes the movement of the wall. It makes an angle б *with the normal if the wall is rough.*

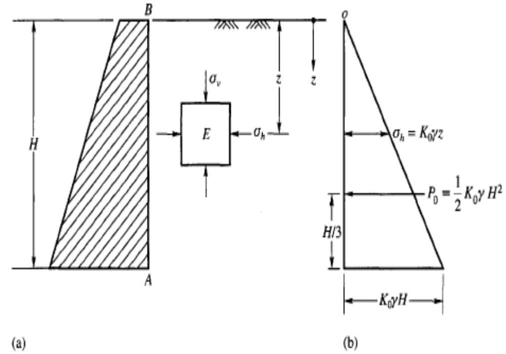


(a) Active earth pressure



(b) Passive earth pressure

Lateral Earth Pressure at rest condition



- If the wall is rigid and does not move with the pressure exerted on the wall, the soil behind the wall will be in a state of elastic equilibrium. Consider a prismatic element E in the backfill at depth z shown in Fig.
- Element *E is subjected to the following pressures.* $\sigma_z = \gamma_z$; lateral pressure $= \sigma_h$
- where γ is the effective unit weight of the soil. If we consider the backfill is homogeneous then both σ_z and σ_h *increase linearly with depth z. In such a case, the ratio* σ_z *of ah to* σ_h *remains constant with* respect to depth, that is
- where K_o is called the coefficient of earth pressure for the at rest condition or at rest earth pressure coefficient.
- The lateral earth pressure σ_h acting on the w $\frac{\sigma_h}{\sigma_v} = \frac{\sigma_h}{\gamma z} = \text{constant} = K_0$ z may be expressed as $\sigma_h = K_o \gamma z$ *and*

$$P_0 = \frac{1}{2} K_0 \gamma H^2$$

- According to Jocky (1940)

$$K_0 = 1 - \sin\phi$$

Example

- If a retaining wall 5 m high is restrained from yielding, what will be the at-rest earth pressure per meter length of the wall? Given: the backfill is cohesionless soil having ϕ= 30° and ɣ = 18 kN/m3. Also determine the resultant force for the at-rest condition.
- Solution: For soil at rest condition, the lateral coefficenet
-

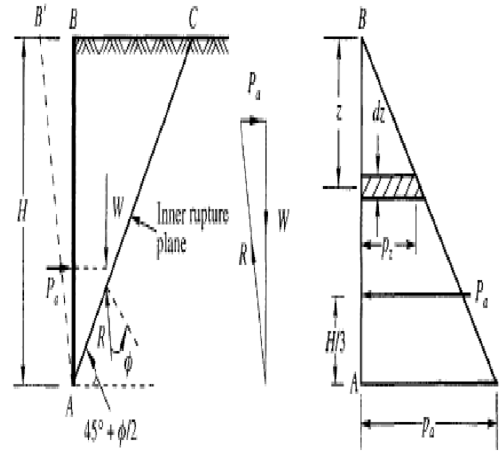
$$K_0 = 1 - \sin\phi = 1 - \sin 30^\circ = 0.5$$

$$P_0 = \frac{1}{2} K_0 \gamma H^2 = \frac{1}{2} \times 0.5 \times 18 \times 5^2 = 112.5 \text{ kN/m length of wall}$$

$$\sigma_h = K_o \gamma H = 0.5 \times 18 \times 5 = 45 \text{ kN/m}^2$$

RANKINE'S EARTH PRESSURE AGAINST SMOOTH VERTICAL WALL WITH COHESIONLESS BACKFILL

I. Backfill Horizontal-Active Earth Pressure



(a) (b)

- A semi-infinite mass is replaced by a smooth wall *AB in Fig.*
- The lateral pressure acting against smooth wall *AB is due to the mass of soil ABC above* failure line *AC which makes an angle of* $45° + \phi/2$ *with the horizontal. The lateral pressure distribution on wall AB of height H increases in simple proportion to depth. The pressure acts normal to the wall AB*
- *The lateral active pressure at A is*

$$p_a = \gamma H K_A$$

The total pressure on *AB* is therefore

$$P_a = \int_0^H p_z\, dz = K_A \int_0^H \gamma z\, dz = \frac{1}{2} K_A \gamma H^2$$

where, $K_A = \tan^2(45° - \phi/2) = \dfrac{1 - \sin\phi}{1 + \sin\phi}$

P_a acts at a height $H/3$ above the base of the wall.

II. Backfill Horizontal-Passive Earth Pressure

- If wall *AB is pushed into the mass to such an extent as to impart uniform compression throughout* the mass, soil wedge *ABC in Fig(a). will be in Rankine's passive state of plastic equilibrium.*
- *The inner rupture plane AC makes an angle* $45° + \phi/2$ *with the vertical AB. The pressure distribution on wall AB is linear as shown in Fig(b).*

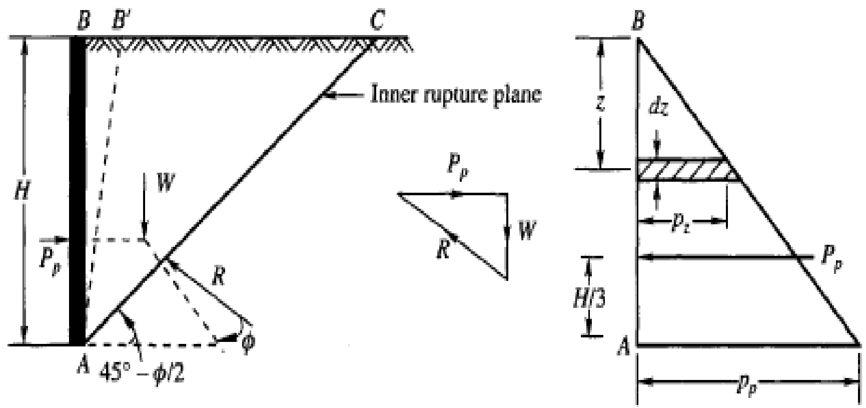
The passive pressure p_p at A is

$$p_p = \gamma H K_p$$

the total pressure against the wall is

$$P_p = \int_0^H p_z\,dz = K_p \int_0^H \gamma z\,dz = \frac{1}{2} K_p \gamma H^2$$

where, $K_p = \tan^2(45° + \phi/2) = \dfrac{1+\sin\phi}{1-\sin\phi}$



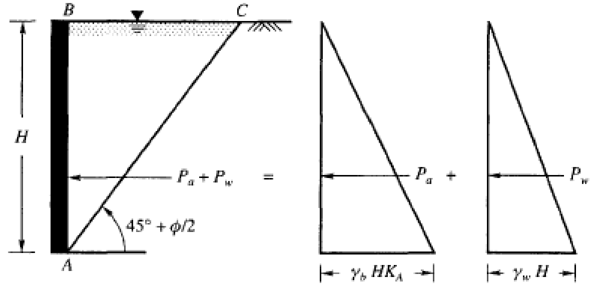
(a) (b)

- The ratio of K_p and K_A may be written as:

$$\frac{K_p}{K_A} = \frac{\tan^2(45° + \phi/2)}{\tan^2(45° - \phi/2)} = \tan^4(45° + \phi/2) \qquad K_p = \frac{1}{K_A}$$

Active Earth Pressure-Backfill Soil Submerged with the Surface Horizontal

- When the backfill is fully submerged, two types of pressures act on wall *AB, as shown in Fig*
    1. *The active earth pressure due to the submerged weight of soil*
    2. *The lateral pressure due to water*



(a) Retaining wall (b) Pressure distribution

At any depth *z the total unit pressure on the wall is*

$$\overline{p_a} = p_a + p_w = \gamma_b z K_A + \gamma_w z$$

At depth z = H, we have

$$\overline{p_a} = \gamma_b H K_A + \gamma_w H$$

- where γb *is the submerged unit weight of soil and* γw *the unit weight of water. The total pressure* acting on the wall at a height *H/3 above the base is*

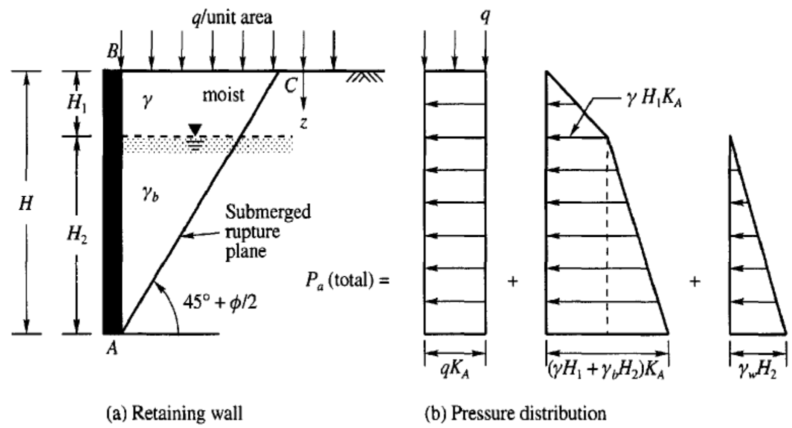
$$\overline{P_a} = P_a + P_w = \frac{1}{2}\gamma_b H^2 K_A + \frac{1}{2}\gamma_w H^2$$

with a Uniform Surcharge Load

- The soil upto depth H1 is moist whereas

Below this point for depth H2 is submerged.

At depth *H1 at the level of the water table*



(a) Retaining wall (b) Pressure distribution

- At depth *H we have* $\overline{p_a} = qK_A + \gamma H_1 K_A$ *rge, the moist soil for dept h depth H2.*

- The total pressure *Pa acting per unit length of the wall* may be written as equal to:

- The point of application of *Pa above the base of the wall can be found by taking moments of all the forces acting on the wall about A*

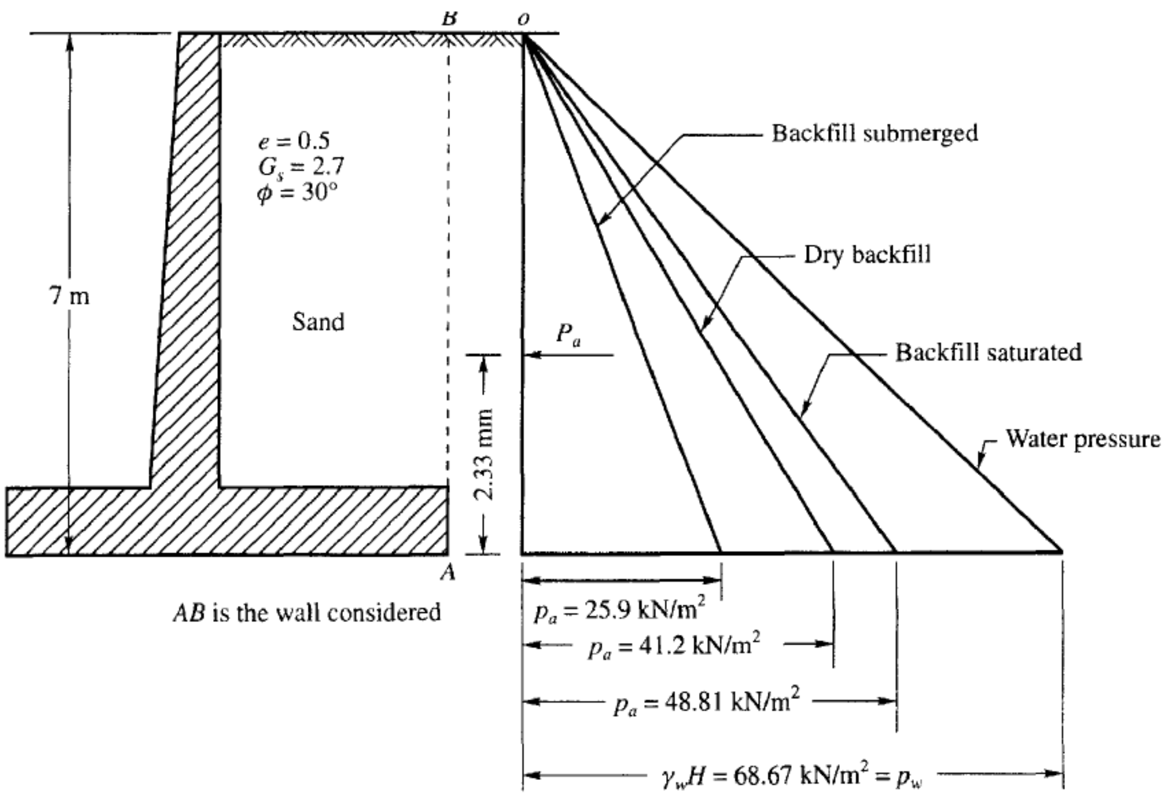
$$\overline{p_a} = qK_A + \gamma H_1 K_A + \gamma_b H_2 K_A + \gamma_w H_2 \qquad \overline{p_a} = qK_A + (\gamma H_1 + \gamma_b H_2)K_A + \gamma_w H_2$$

-

- $$\overline{P_a} = qHK_A + \frac{1}{2}\gamma H_1^2 K_A + \gamma H_1 H_2 K_A + \frac{1}{2} H_2^2 (\gamma_b K_A + \gamma_w)$$

Solved Example

- A cantilever retaining wall of 7 meter height (Fig. Ex. 11.2) retains sand. The properties of the sand are: $e = 0.5, \phi = 30°$ *and* $G = 2.7$. *Using Rankine's theory determine the active earth pressure at the*
- base when the backfill is (i) dry, (ii) saturated and (iii) submerged, and also the resultant active force in each case. In addition determine the total water pressure under the submerged condition.



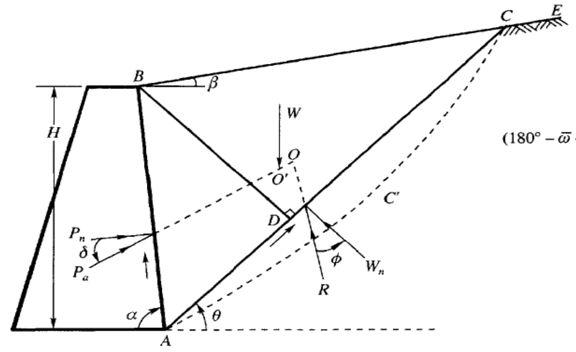
Assignment

- A rigid retaining wall 5 m high supports a backfill of cohesionless soil with $\phi = 30°$. The water table is below the base of the wall. The backfill is dry and has a unit weight of 18 kN/m3. Determine Rankine's passive earth pressure per meter length of the wall

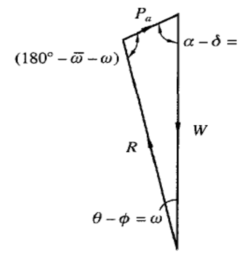
- A counter fort wall of 10 m height retains a non-cohesive backfill. The void ratio and angle of internal friction of the backfill respectively are 0.70 and 30° in the loose state and they are 0.40 and 40° in the dense state. Calculate and compare active and passive earth pressures for both the cases. Take the specific gravity of solids as 2.7.

SAND FOR ACTIVE STATE

- Coulomb made the following assumptions in the development of his theory:
- 1.The soil is isotropic and homogeneous
- 2.The rupture surface is a plane surface
- 3.The failure wedge is a rigid body
- 4.The pressure surface is a plane surface
- 5.There is wall friction on the pressure surface
- 6. Failure is two-dimensional and
- 7.The soil is cohesionless



(a) Retaining wall



(b) Polygon of forc

- *In Fig AB is the pressure face*
- The backfill surface *BE is a plane inclined at an angle ß with the horizontal*
- α is the angle made by the pressure face *AB with the horizontal*
- *H is the height of the wall*
- *AC is the assumed rupture plane surface, and*
- θ *is the angle made by the surface AC with the horizontal*
- *The weight of the wedge W length of the wall may be written as*
- $W = \gamma A$, *where* A = *area of wedge ABC*
- Area of wedge *ABC* = $A = 1/2\ AC \times BD$, where *BD is drawn perpendicular to AC.*
- From the law of sines, we have
- Making the substitution and simplifying $AC = AB\dfrac{\sin(\alpha+\beta)}{\sin(\theta-\beta)}, \quad BD = AB\sin(\alpha+\theta), \quad AB = \dfrac{H}{\sin\alpha}$
- The various forces that are acting on the wedge are shown in Fig

$$W = \gamma A = \frac{\gamma H^2}{2\sin^2\alpha}\sin(\alpha+\theta)\frac{\sin(\alpha+\beta)}{\sin(\theta-\beta)}$$

- As the pressure face AB moves away from the backfill, there will be sliding of the soil mass along the wall from B towards A. The sliding of the soil mass is resisted by the friction of the surface. The direction of the shear stress is in the direction from A towards B.
- lf P_n is the total normal reaction of the soil pressure acting on face AB, the resultant of Pn and the shearing stress is the active pressure Pa making an angle δ with the normal. Since the shearing stress acts upwards, the resulting Pa dips below the normal. The angle δ for this condition is considered positive.
- As the wedge *ABC ruptures along plane AC, it slides along this plane. This is resisted by the* frictional force acting between the soil at rest below AC, and the sliding wedge. The resisting shearing stress is acting in the direction from A towards *C. If W_n is the normal component of the* weight of wedge *W on plane AC, the resultant of the normal Wn and the shearing stress is the* reaction *R. This makes an angle ϕ with the normal since the rupture takes place within the soil itself.*

- Statical equilibrium requires that the three forces *Pa, W, and R meet at a point. Since AC is not the* actual rupture plane, the three forces do not meet at a point. But if the actual surface of failure AC'C is considered, all three forces meet at a point. However, the error due to the non concurrence of the forces is very insignificant and as such may be neglected.
- The polygon of forces is shown in Fig.

$$\frac{P_a}{\sin(\theta-\phi)} = \frac{W}{\sin(180^\circ-\alpha-\theta+\phi+\delta)}$$

$$\text{or} \quad P_a = \frac{W\sin(\theta-\phi)}{\sin(180^\circ-\alpha-\theta+\phi+\delta)}$$

- In Eq. , the only variable is θ *and all the other ter* ng for *W, we have*

$$P_a = \frac{\gamma H^2}{2\sin^2\alpha}\,\frac{\sin(\theta-\phi)}{\sin(180^\circ-\alpha-\theta+\phi+\delta)}\left(\sin(\alpha+\phi)\,\frac{\sin(\alpha+\beta)}{\sin(\theta-\beta)}\right)$$

- The maximum value for *Pa is obtained by differentiat* derivative to zero, i.e.
- The maximum value of *Pa so obtained may be written as*

$$\frac{dP_a}{d\theta} = 0$$

$$P_a = \frac{1}{2}\gamma H^2 K_A$$

Active earth pressure coefficients K_A *for* $\beta=0$ *and* $\alpha = 90$ *deg*

ϕ^0	15	20	25	30	35	40
$\delta = 0$	0.59	0.49	0.41	0.33	0.27	0.22
$\delta = +\phi/2$	0.55	0.45	0.38	0.32	0.26	0.22
$\delta = +/2/3\phi$	0.54	0.44	0.37	0.31	0.26	0.22
$\delta = +\phi$	0.53	0.44	0.37	0.31	0.26	0.22

Table 11.2b Active earth pressure coefficients K_A for $\delta = 0$, β varies from $-30°$ to $+30°$ and α from $70°$ to $110°$

$\beta =$		$-30°$	$-12°$	$0°$	$+12°$	$+30°$
$\phi = 20°$	$\alpha = 70°$	–	0.54	0.61	0.76	–
	80°	–	0.49	0.54	0.67	–
	90°	–	0.44	0.49	0.60	–
	100	–	0.37	0.41	0.49	–
	110	–	0.30	0.33	0.38	–
$\phi = 30°$	70°	0.32	0.40	0.47	0.55	1.10
	80°	0.30	0.35	0.40	0.47	0.91
	90°	0.26	0.30	0.33	0.38	0.75
	100	0.22	0.25	0.27	0.31	0.60
	110	0.17	0.19	0.20	0.23	0.47
$\phi = 40°$	70	0.25	0.31	0.36	0.40	0.55
	80	0.22	0.26	0.28	0.32	0.42
	90	0.18	0.20	0.22	0.24	0.32
	100	0.13	0.15	0.16	0.17	0.24
	110	0.10	0.10	0.11	0.12	0.15

- where K_A *is the active earth pressure coefficient.*

$$K_A = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha\sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

- The total normal component *Pn of the earth pressure on the back of the wall is*

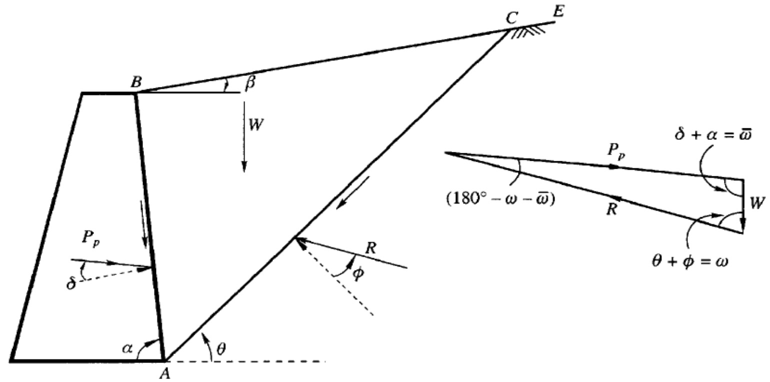
$$P_n = P_a\cos\delta = \frac{1}{2}\gamma H^2 K_A\cos\delta$$

- If the wall is vertical and smooth, and if the backfill is horizontal, we have
- *β=δ= 0 and α= 90 deg,* Substituting these values in Eq.

$$K_A = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45^\circ - \frac{\phi}{2}\right) = \frac{1}{N_\phi}$$

where $N_\phi = \tan^2\left(45^\circ + \frac{\phi}{2}\right)$

COULOMB'S EARTH PRESSURE THEORY FOR SAND FOR PASSIVE STATE



- As the wall moves into the backfill, the soil tries to move up on the pressure surface AB which is resisted by friction of the surface. Shearing stress on this surface therefore acts downward. The passive earth pressure P is the resultant of the normal pressure P and the shearing stress. The shearing force is rotated upward with an angle δ which is again the angle of wall friction. In this case S is positive.
- As the rupture takes place along assumed plane surface AC, the soil tries to move up the plane which is resisted by the frictional force acting on that line. The shearing stress therefore, acts downward. The reaction R makes an angle ϕ with the normal and is rotated upwards as shown in the figure. The polygon of forces is shown in (b) of t[illegible]

$$W = \frac{\gamma H^2}{2\sin^2\alpha}\sin(\alpha+\theta)\frac{\sin(\alpha+\beta)}{\sin(\theta-\beta)}$$

- Differentiating Eq. with respect to θ *and setting the derivative to zero, gives the* minimum value of *Pp as*

$$P_p = \frac{W\sin(\theta+\phi)}{\sin(180° - \theta - \phi - \delta - \alpha)}$$

where *Kp is called the passive earth pressure coefficient.*

$$P_p = \frac{1}{2}\gamma H^2 K_p$$

ACTIVE PRESSURE BY CULMANN'S METHOD FOR COHESIONLESS SOILS

Culmann's (1875) method is the same as the trial wedge method. In

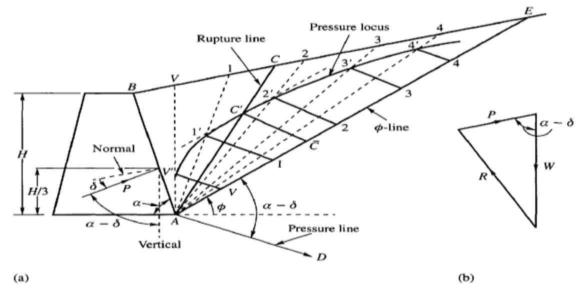
Culmann's method, the force polygons are constructed directly on the ϕ-line AE taking AE as the load line. The procedure is as follows:

1. Draw ϕ -line AE at an angle ϕ to the horizontal.

2. Lay off on AE distances, AV, A1, A2, A3,

etc. to a suitable scale to represent the weights

of wedges ABV, A51, AS2, AS3, etc. respectively.



3. Draw lines parallel to *AD from points V, 1, 2, 3 to intersect assumed rupture lines AV, A1,* A2, A3 at points *V'', 1', 2', 3', etc. respectively.*

4. Join points *V, 1', 2' 3' etc. by a smooth curve which is the pressure locus.*

5. Select point C' on the pressure locus such that the tangent to the curve at this point is parallel to the ϕ-line *AE*.

6. Draw *C'C parallel to the pressure line AD. The magnitude of C'C in its natural units* gives the active pressure *Pa*.

7. Join AC'' and produce to meet the surface of the backfill at C. AC is the rupture line. For the plane backfill surface, the point of application of Pa is at a height of H/3 from the base of the wall.

E

Pressure locus 4

Rupture line

C 2 3

V

B

p

$\sigma - \sigma$

H

Normal

W

H/3

$\sigma - \sigma$

Pressure line

Venical

D

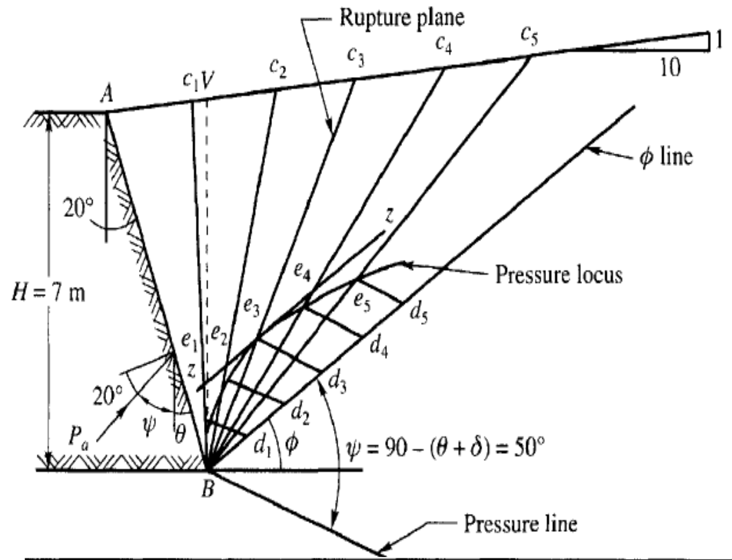
(a)

(b)

Example:

- For a retaining wall system, the following data were available: (i) Height of wall = 7 m, (ii) Properties of backfill: $\gamma_d = 16\ kN/m3, \phi = 35°$,*(iii) angle of wall friction*, $8 = 20°$,*(iv) back of* wall is inclined at 20° to the vertical (positive batter), and (v) backfill surface is sloping at 1 : 10.Determine the magnitude of the active earth pressure by Culmann's method.

(a) Fig. shows the ϕ line and pressure lines drawn to a suitable scale.
(b) The trial rupture lines Bc_1 Bc_2, Bc_3 *etc. are drawn by making* $Ac_1 = c_1c_2 = c2c3$, *etc.*
(c) The length of a vertical line from *B to the backfill surface is measured.*
(d) The areas of wedges *BAc1 BAc2, BAc3 etc. are respectively equal to l/2(base lengths Ac1,Ac2, Acy etc.) * perpendicular length.*
(e). The weights of the wedges in *(d) above per meter length of wall may be determined by* multiplying the areas by the unit weight of the soil. The results are tabulated
(f) The weights of the wedges BAc1, BAc2, etc. are respectively plotted are Bd1 Bd2, etc. on the ϕ-line.



Wedge	Weight, kN	Wedge	Weight, kN
BAc_1	115	BAc_4	460
BAc_2	230	BAc_5	575
BAc_3	345		

- (g) Lines are drawn parallel to the pressure line from points d1, d2, d3 etc. to meet respectively the trial rupture lines Bc1 Bc2, Bc3 etc. at points e1, e2, e3 etc.
- (h)The pressure locus is drawn passing through points e\, e2, ey etc.
- (i) Line zz is drawn tangential to the pressure locus at a point at which zz is parallel to the ϕ line.This point coincides with the point e3
- (j) e3d3gives the active earth pressure when converted to force units.
- Pa = 180 kN per meter length of wall,
- (k) Bc3 is the critical rupture plane.

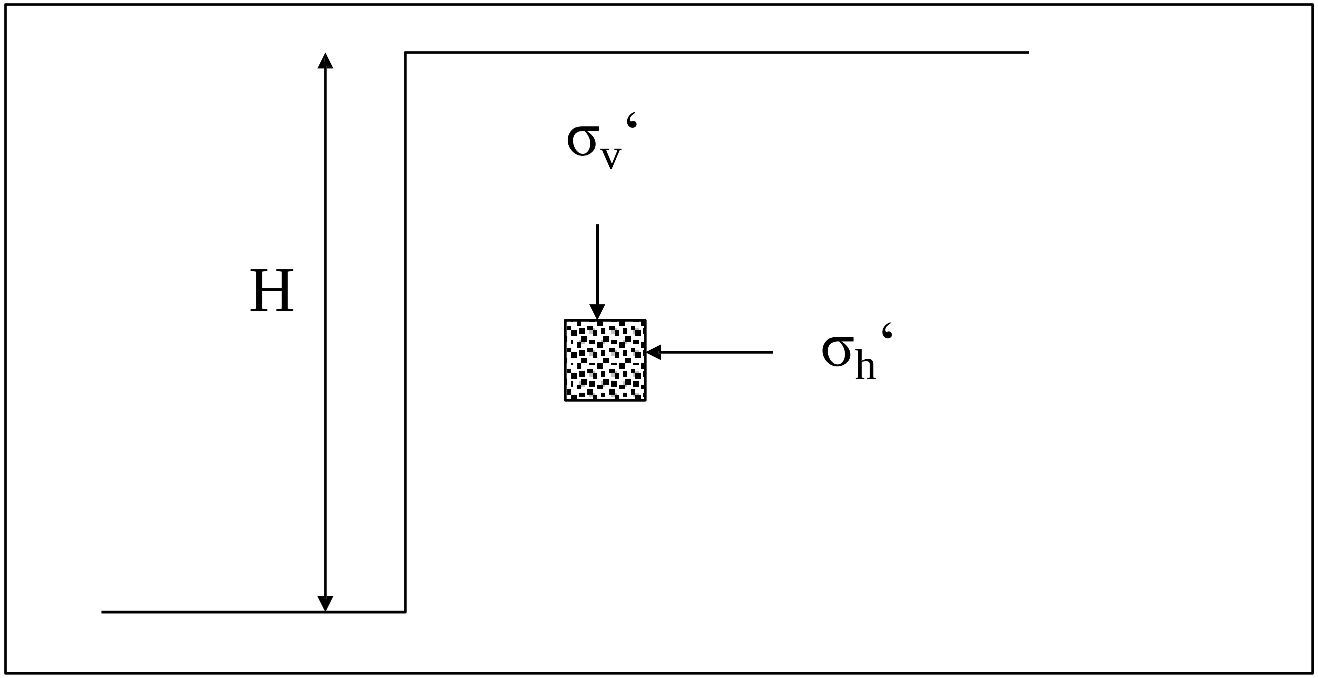
STABILITY OF RETAINING WALLS

1. Check for sliding
2. Check for overturning
3. Check for bearing capacity failure
4. Check for base shear failure

The minimum factors of safety for the stability of the wall are:

1. Factor of safety against sliding =1.5
2. Factor of safety against overturning = 2.0
3. Factor of safety against bearing capacity failure = 3.0

Lateral Earth Pressure



- We can calculate σ_v'
- Now, calculate σ_h' which is the horizontal stress
- $\sigma_h' / \sigma_v' = K$
- Therefore, $\sigma_h' = K\sigma_v'$ (σ_v' is what?)

Lateral Earth Pressure

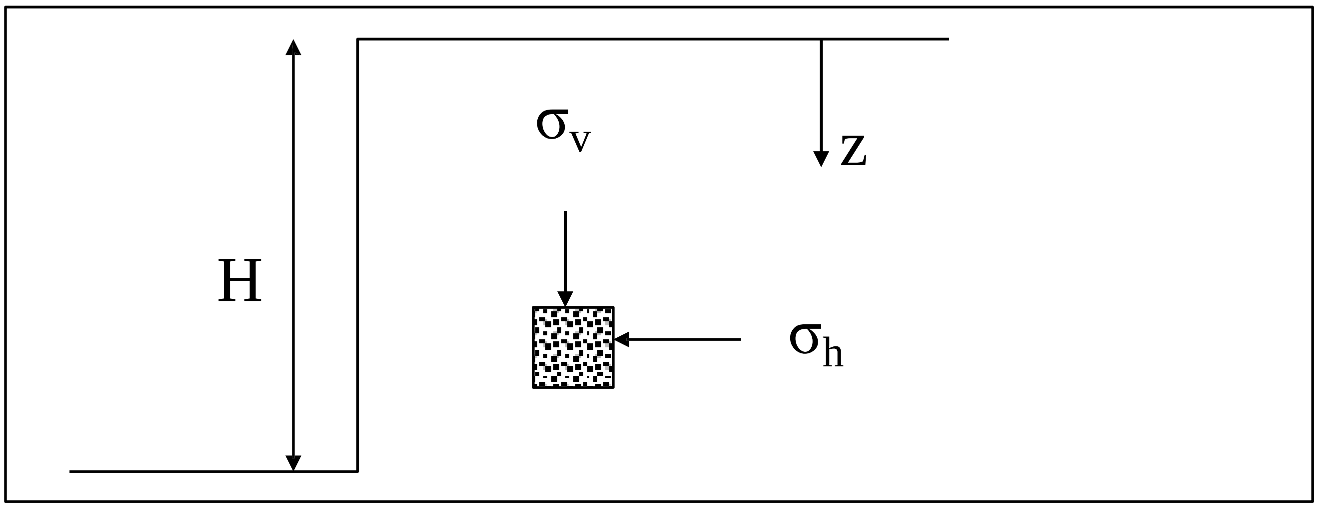
- There are 3 states of lateral earth pressure

  K_o = At Rest

  K_a = Active Earth Pressure (wall moves away from soil)

  K_p = Passive Earth Pressure (wall moves into soil)

  Passive is more like a resistance

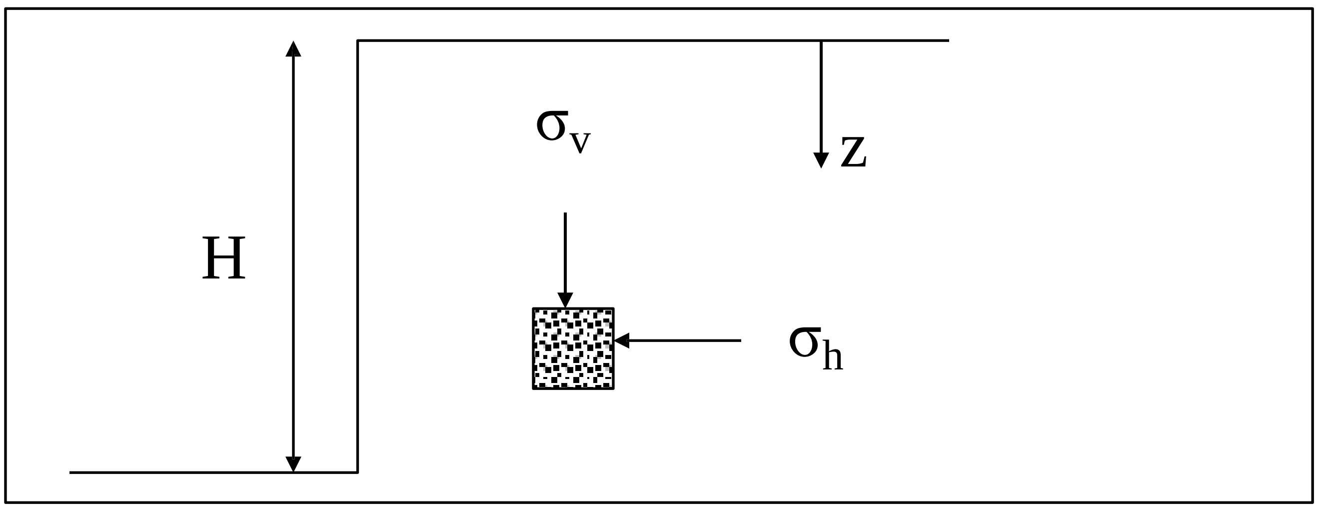


At Rest Earth Pressure

At rest earth pressure occur when there is no wall rotation such as in a braced wall (basement wall for example)

K_o can be calculated as follows:

$K_o = 1 - \sin\varphi$	for coarse grained soils
$K_o = .44 + .42\ [PI / 100]$	for NC soils
$K_{o\ (oc)} = K_{o\ (NC)}\ (OCR)^{1/2}$	for OC soils



Active Earth Pressure

Active earth pressure occurs when the wall tilts away from the soil (a typical free standing retaining wall)

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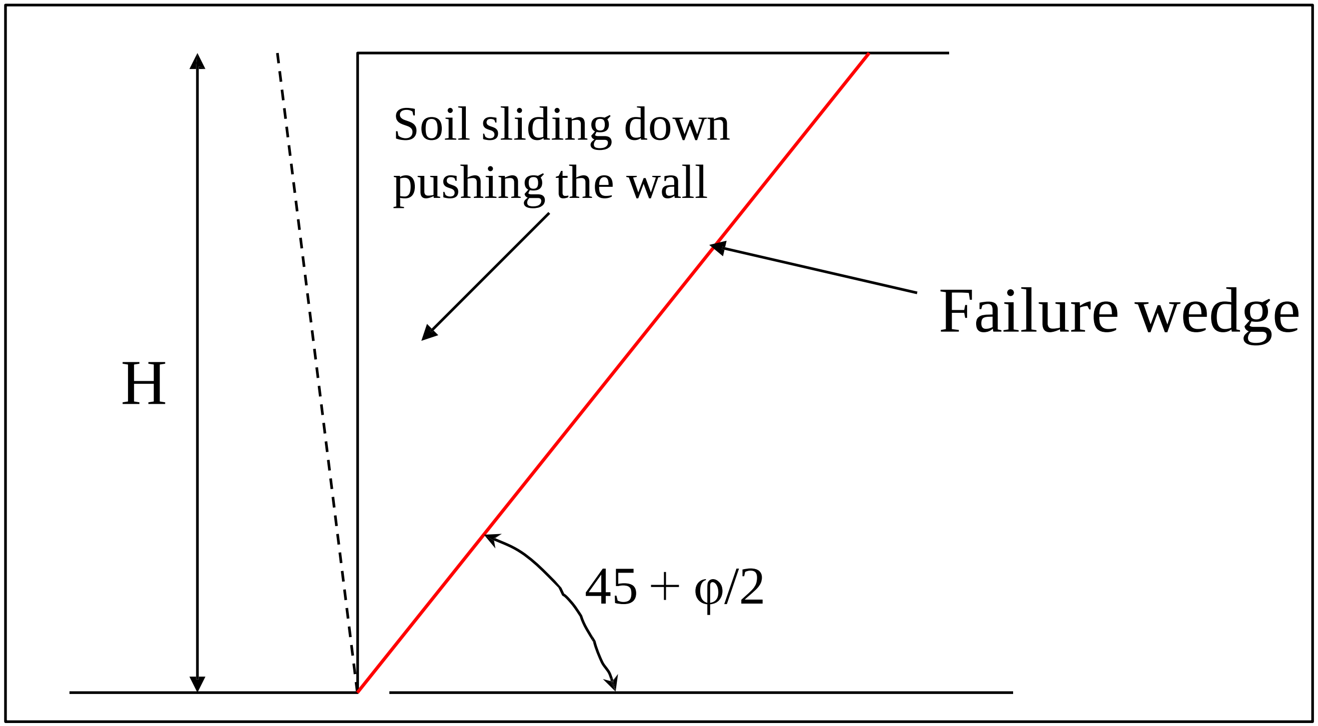
K_a can be calculated as follows:

$$K_a = \tan^2 (45 - \varphi/2)$$

thus: $\sigma_a = K_a\, \sigma_v' - 2\, c\, (K_a)^{1/2}$



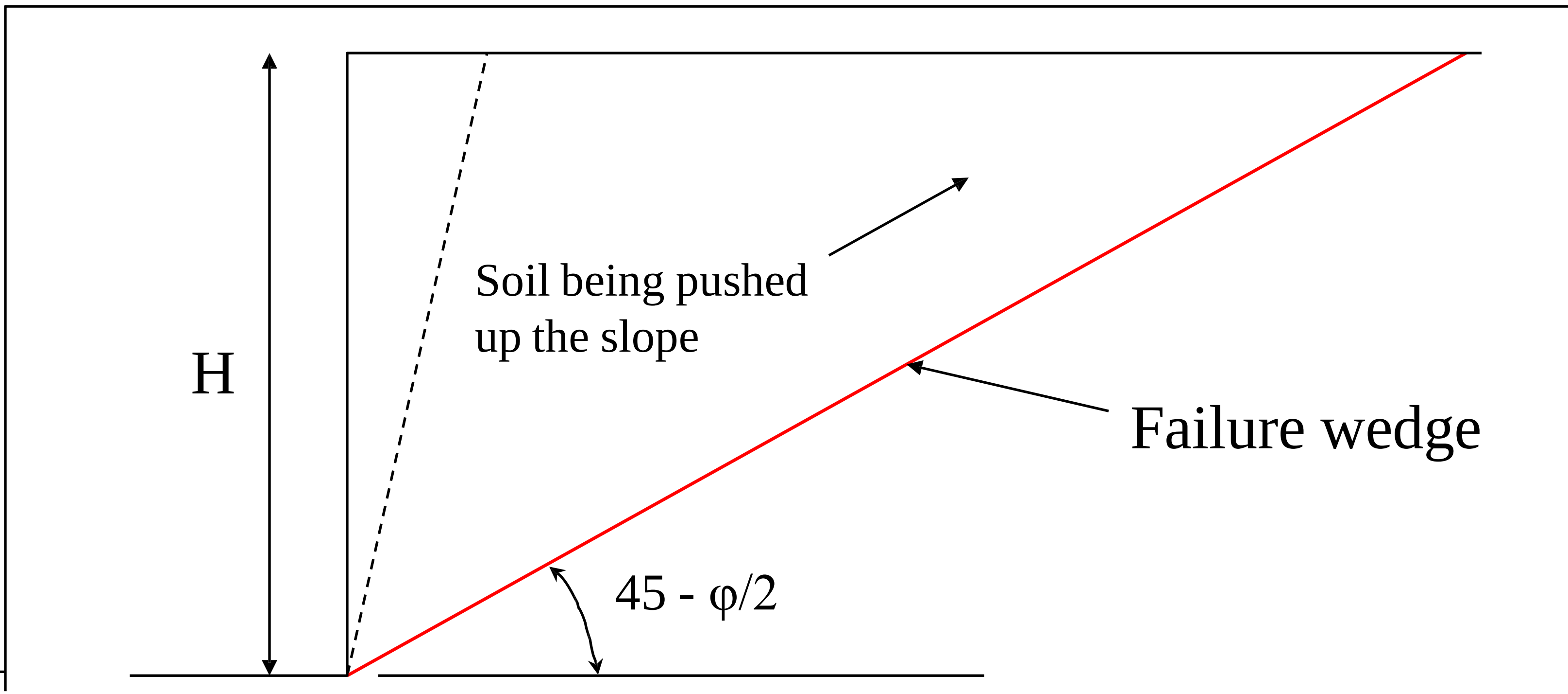
Passive Earth Pressure

Passive earth pressure occurs when the wall is pushed into the soil (typically a seismic load pushing the wall into the soil or a foundation pushing into the soil)

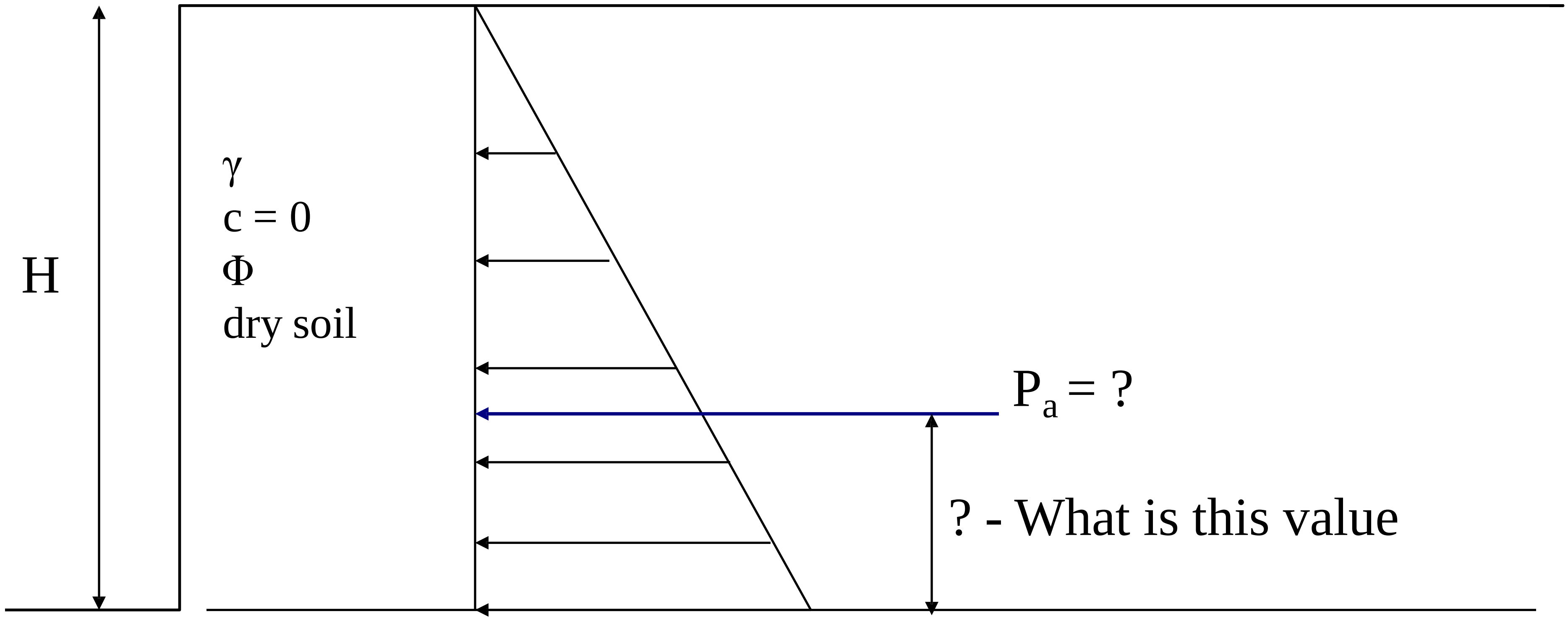
K_p can be calculated as follows:

$$K_p = \tan^2 (45 + \varphi/2)$$

thus: $\sigma_p = K_p \sigma_v' + 2 c (K_p)^{1/2}$



Active Stress Distribution (c = 0)

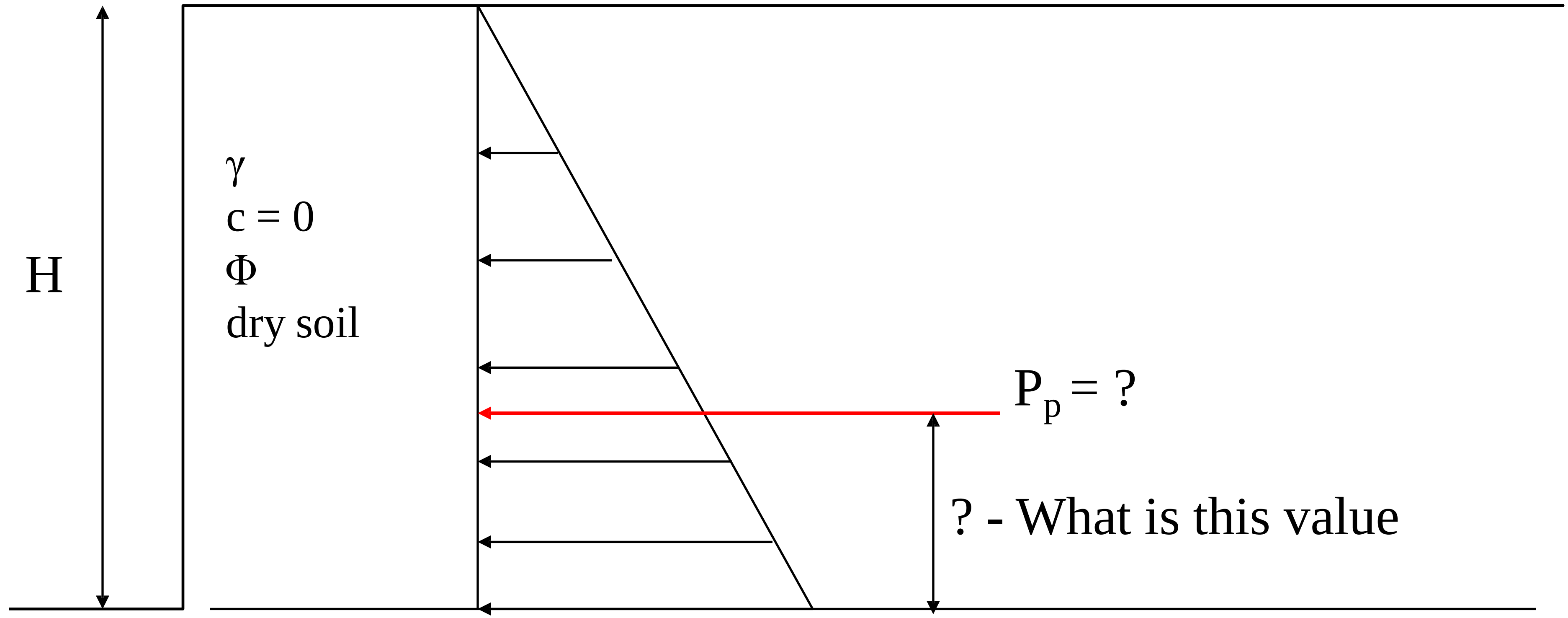


$$\sigma_a = K_a \sigma_v' - \cancelto{0}{2c(K_a)^{1/2}}$$

$$\sigma_a = K_a \sigma_v'$$

σ_a is the stress distribution
P_a is the force on the wall (per foot of wall)
How is P_a found?

Passive Stress Distribution (c = 0)



$$\sigma_p' = K_p \sigma_v' - \cancel{2c(K_p)^{1/2}} \rightarrow 0$$

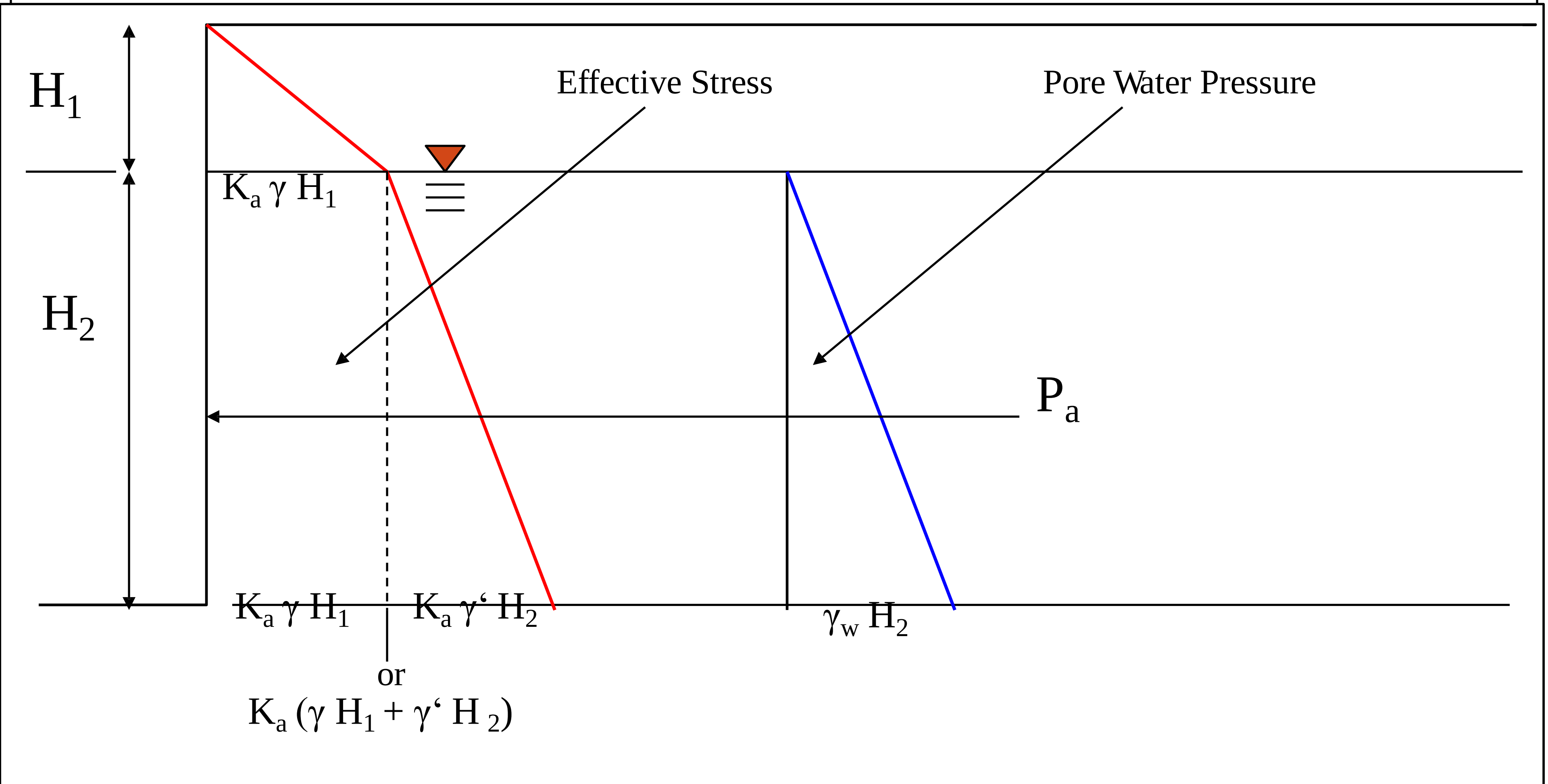
$$\sigma_p' = K_p \sigma_v'$$

σ_p' is the stress distribution

P_p is the force on the wall (per foot of wall)

How is P_p found?

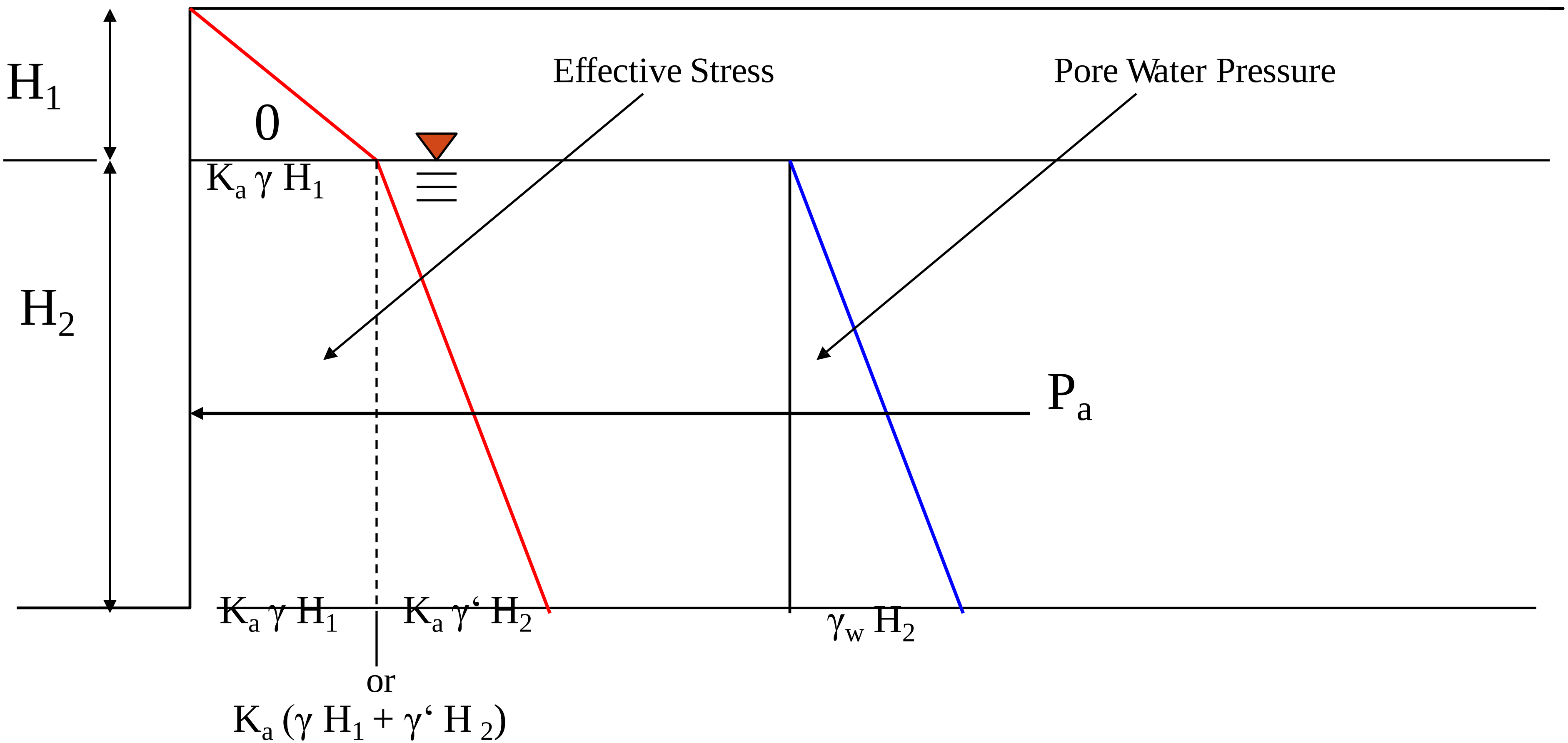
Stress Distribution - Water Table (c = 0)



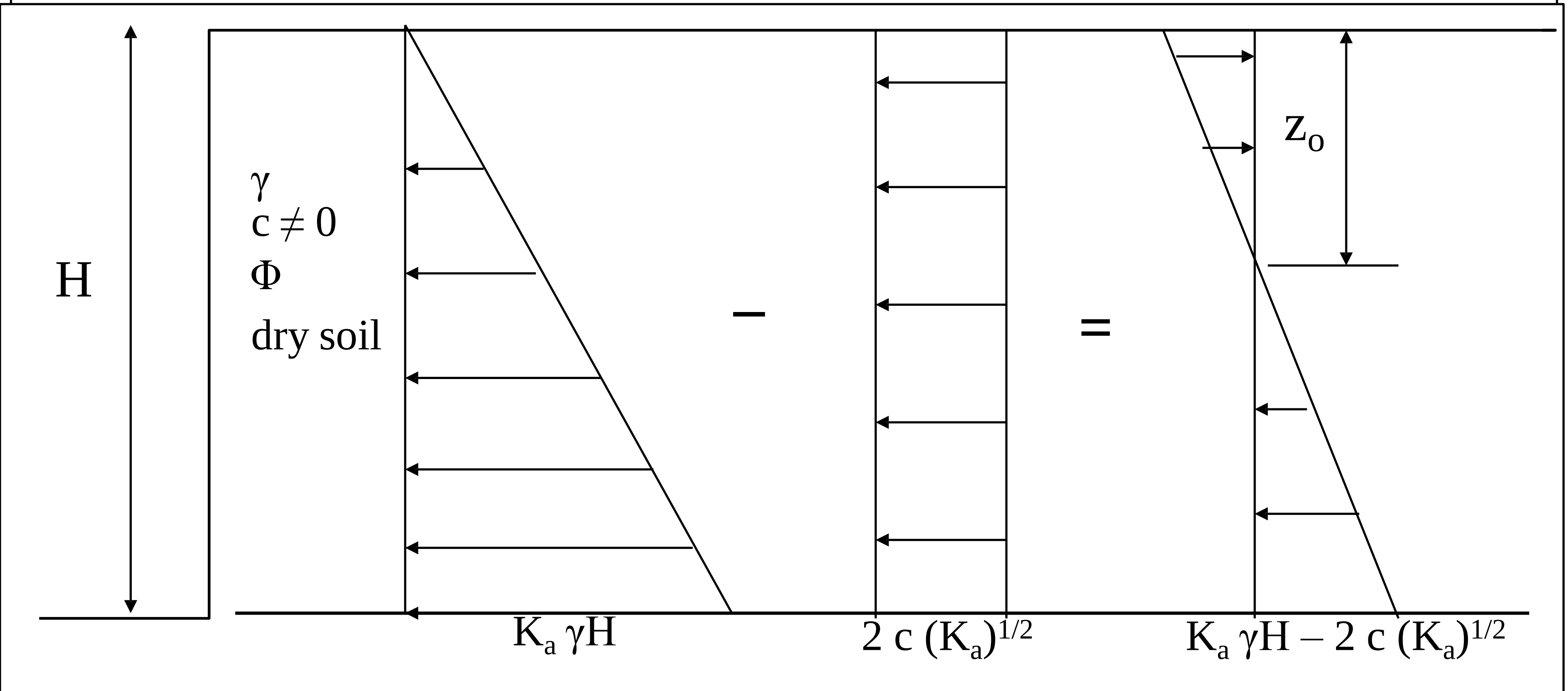
$$P_a = \Sigma \text{ areas} = \tfrac{1}{2} K_a \gamma H_1^2 + Ka\, \gamma H_1 H_2 + \tfrac{1}{2} K_a \gamma' H_2^2 + 1/2 \gamma_w H_2^2$$

Stress Distribution With Water Table

Why is the water pressure considered separately? (K)



Active Stress Distribution $(c \neq 0)$



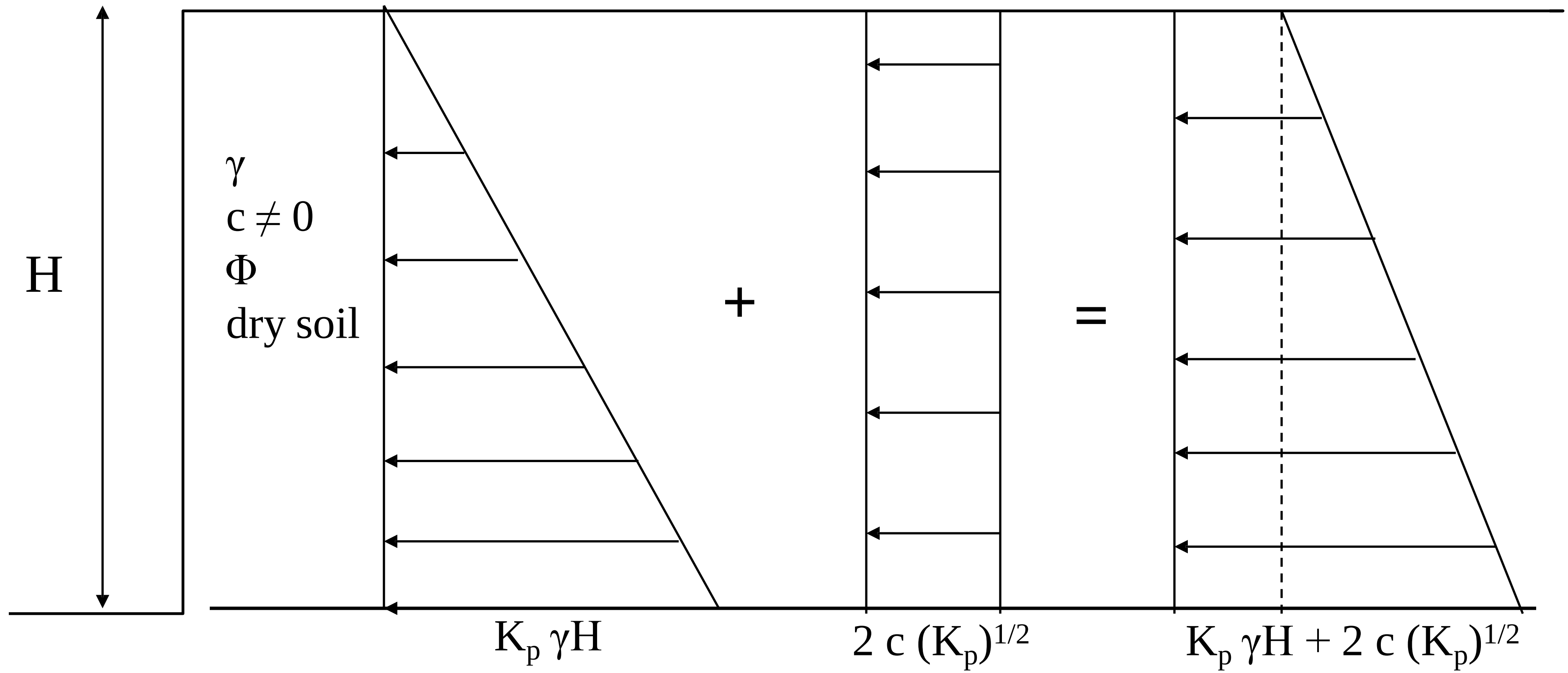
Find z_o:

$K_a \gamma z_o - 2c(K_a)^{1/2} = 0$

$Zo = 2c / \gamma (K_a)^{1/2}$

Pa = ?

Passive Stress Distribution $(c \neq 0)$



Pp = ?

RETAINING WALLS: Types of retaining walls- stability of retaining walls against overturning, sliding, bearing capacity and drainage from backfill.

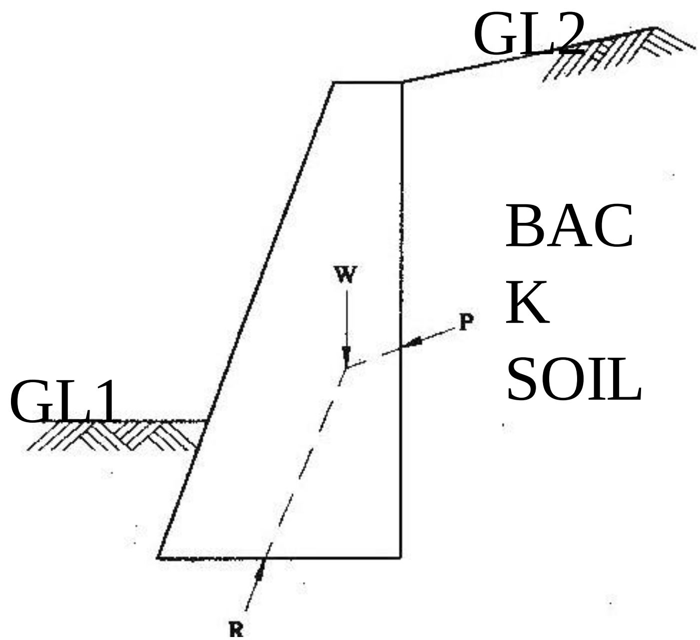
DESIGN AND DETAILING OF RETAINING WALLS

Learning Outcomes:

- After this class students will be able to do the complete design and detailing of different types of retaining walls.

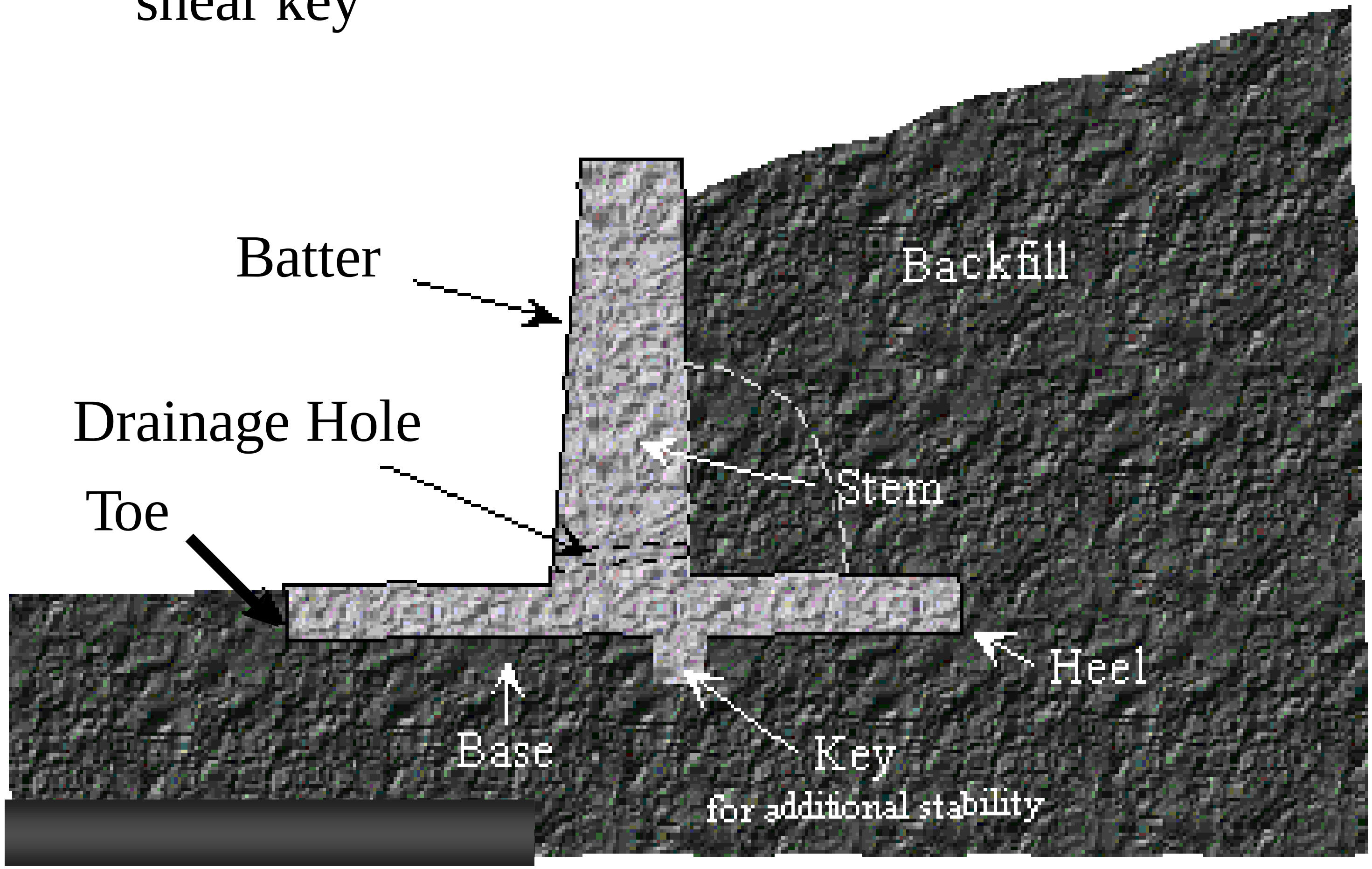
RETAINING WALL

Retaining walls are usually built to hold back soil mass. However, retaining walls can also be constructed for aesthetic landscaping purposes.

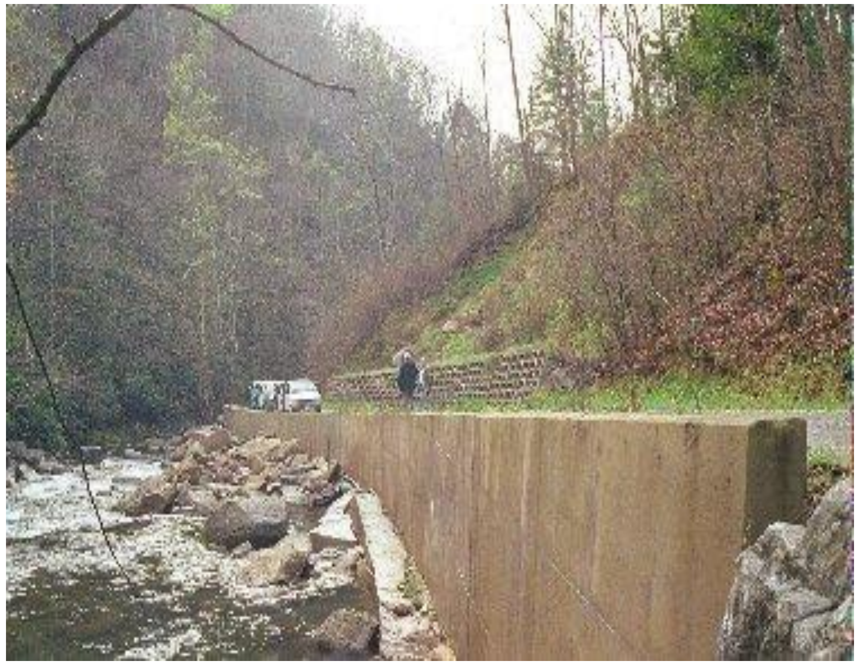


Gravity retaining wall

Cantilever Retaining wall with shear key



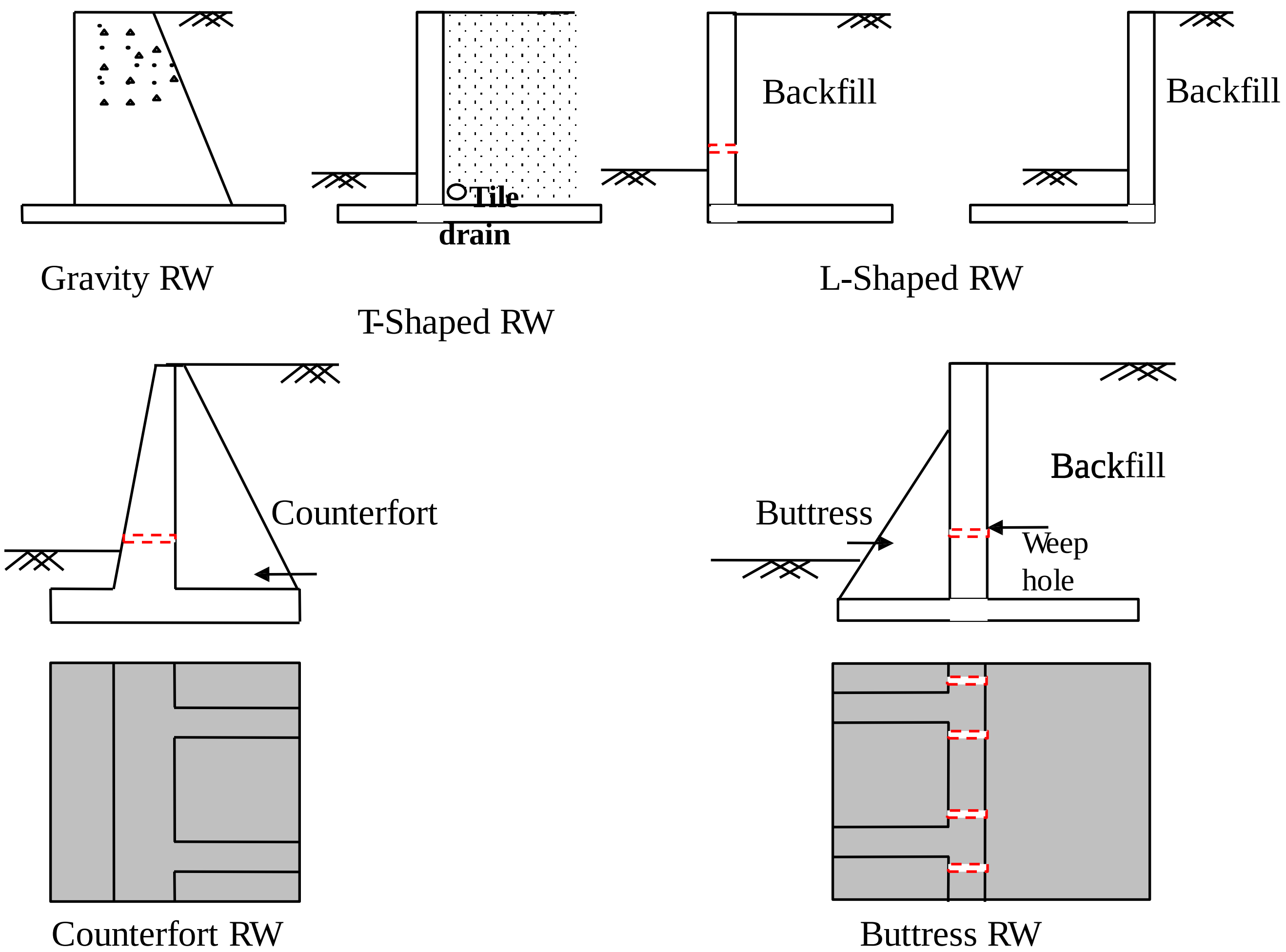
Photos of Retaining walls



Classification of Retaining walls

- Gravity wall-Masonry or Plain concrete
- Cantilever retaining wall-RCC
  (InvertedT and L)
- Counterfort retaining wall-RCC
- Buttress wall-RCC

Classification of Retaining walls



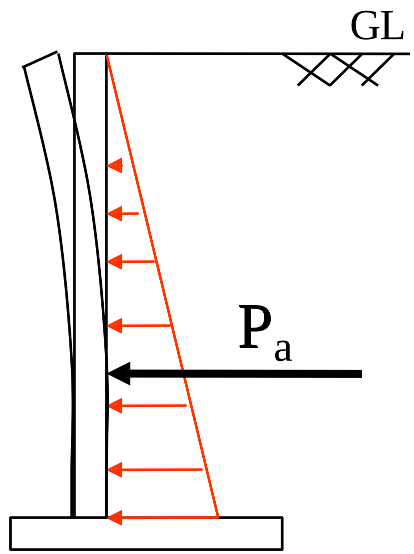
Earth Pressure (P)

- Earth pressure is the pressure exerted by the retaining material on the retaining wall. This pressure tends to deflect the wall outward.

- **Types of earth pressure** :

- Active earth pressure or earth pressure (P_a) and
- Passive earth pressure (P_p).

- Active earth pressure tends to deflect the wall away from the backfill.



Variation of Earth pressure

Factors affecting earth pressure

- Earth pressure depends on type of backfill, the height of wall and the soil conditions

Soil conditions: The different soil conditions are

- Dry leveled back fill
- Moist leveled backfill
- Submerged leveled backfill
- Leveled backfill with uniform surcharge
- Backfill with sloping surface

Analysis for dry back fills

Maximum pressure at any height, $p=k_a\gamma h$

Total pressure at any height from top,

$$p_a=1/2[k_a\gamma h]h = [k_a\gamma h^2]/2$$

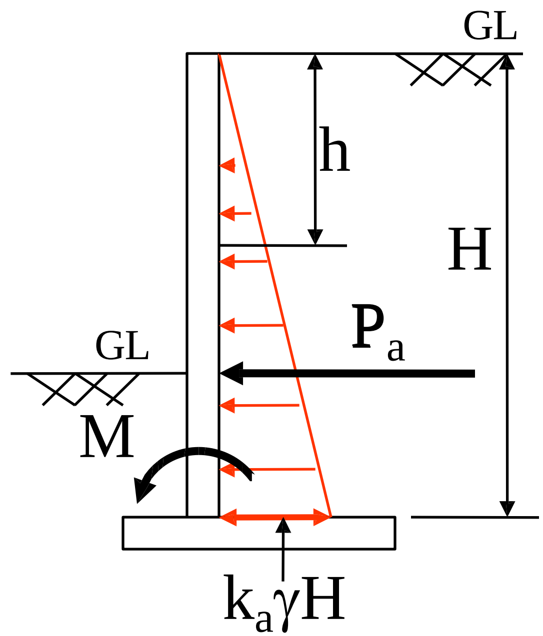
Bending moment at any height

$$M=p_a x h/3= [k_a\gamma h^3]/6$$

$\therefore$ Total pressure, $P_a= [k_a\gamma H^2]/2$

$\therefore$ Total Bending moment at bottom,

$M = [k_a\gamma H^3]/6$



H=stem height

Where, k_a= Coefficient of active earth pressure

= $(1-\sin\phi)/(1+\sin\phi)=\tan^2\phi$

= $1/k_p$, coefficient of passive earth pressure

ϕ= Angle of internal friction or angle of repose

γ=Unit weigh or density of backfill

If ϕ= 30°, k_a=1/3 and k_p=3. Thus k_a is 9 times k_p

Backfill with sloping surface

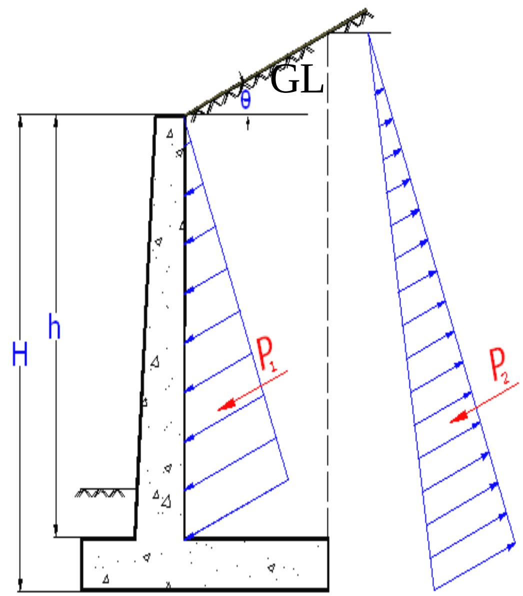
$p_a = k_a \gamma H$ at the bottom and is parallel to inclined surface of backfill

$$k_a = \cos\theta \left[\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}} \right]$$

Where θ=Angle of surcharge

$\therefore$ Total pressure at bottom

$= P_a = k_a \gamma H^2/2$



SOIL PRESSURE DUE TO INCLINED SURCHARGE

Stability requirements of RW

- Following conditions must be satisfied for stability of wall (IS:456-2000).

- **It should not overturn**
- **It should not slide**
- **It should not subside**, i.e Max. pressure at the toe should not exceed the safe bearing capacity of the soil under working condition

Check against overturning

Factor of safety against overturning

$= M_R / M_O \geq 1.55$ $(=1.4/0.9)$

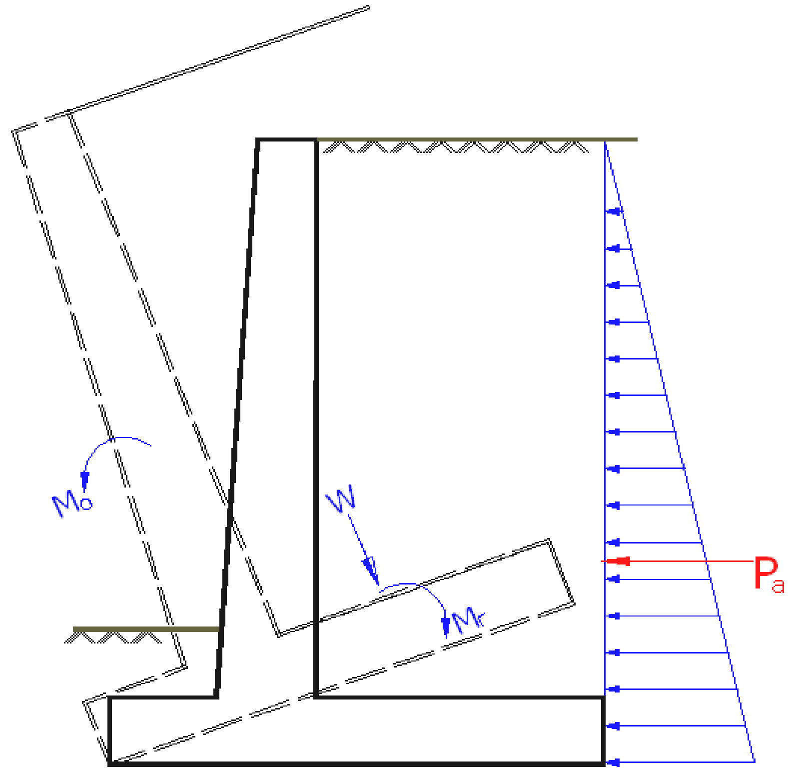
Where,

- M_R =Stabilising moment or restoring moment
- M_O =overturning moment

As per IS:456-2000,

$M_R > 1.2\ M_{O,\ ch.\ DL} + 1.4\ M_{O,\ ch.\ IL}$

$0.9\ M_R \geq 1.4\ M_{O,\ ch\ IL}$

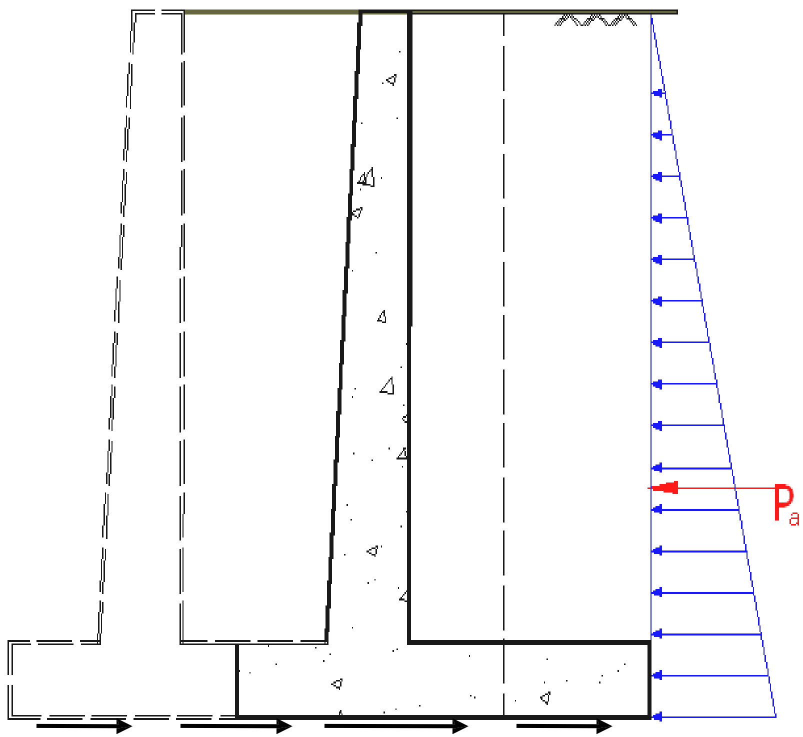


OVERTURNING OF WALL

Check against Sliding

- FOS against sliding
- = Resisting force to sliding/
- Horizontal force causing
- sliding
- $= \mu \sum W / Pa \geq 1.55$ $(=1.4/0.9)$

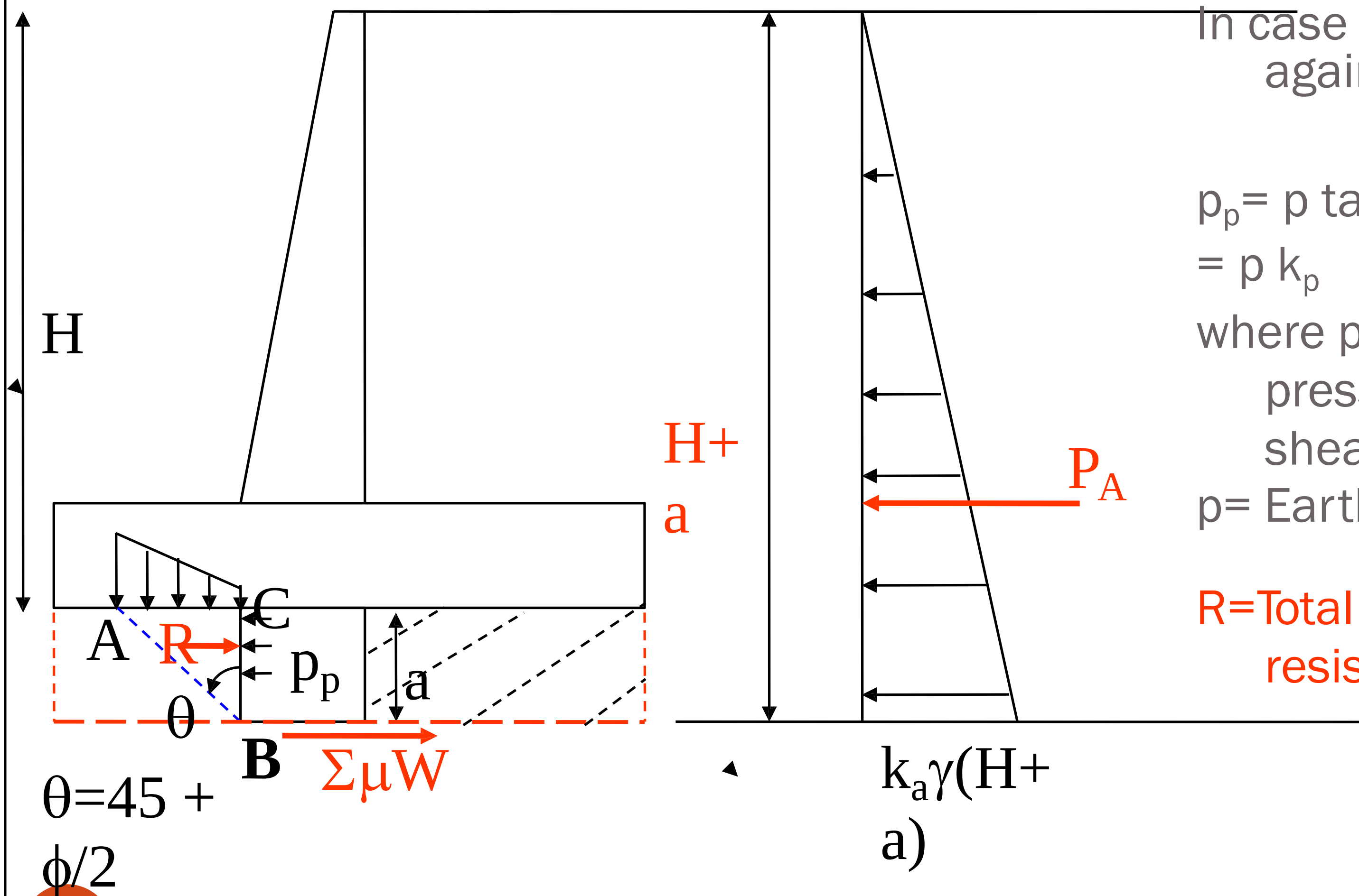
- As per IS:456:2000
- $1.4 = \mu(0.9 \sum W)/P_a$



Friction $\mu \sum W$

SLIDING OF WALL

Design of Shear key



In case the wall is unsafe against sliding

p_p= p $\tan^2(45 + \phi/2)$

= p k_p

where p_p= Unit passive pressure on soil above shearing plane AB

p= Earth pressure at BC

R=Total passive resistance=$p_p$xa

- Design of Shear key-Contd.,

If $\sum W$= Total vertical force acting at the key base

ϕ= shearing angle of passive resistance

R= Total passive force = p_p x a

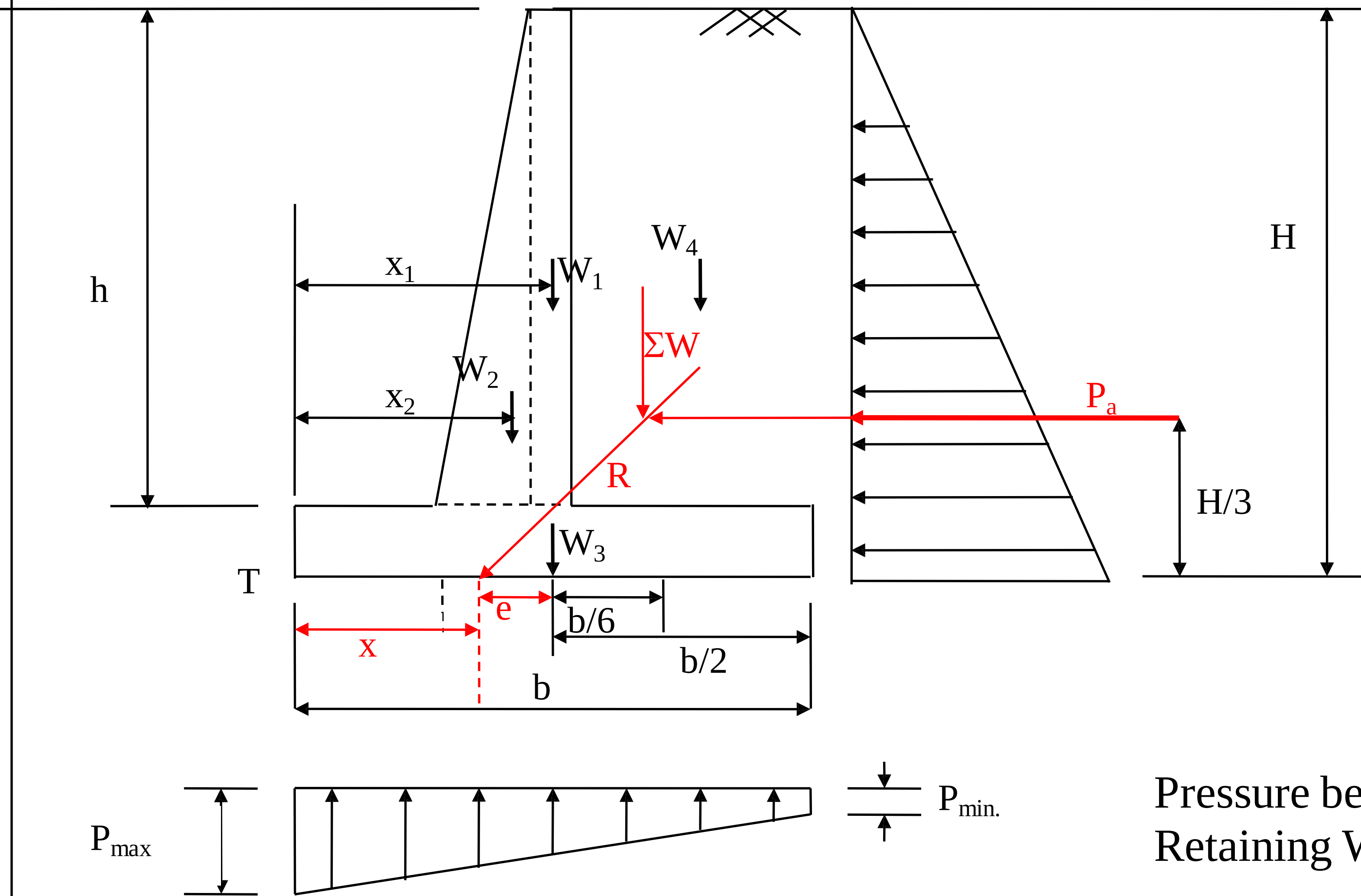
P_A=Active horizontal pressure at key base for H+a

$\mu\sum W$=Total frictional force under flat base

For equilibrium, $R + \mu\sum W = FOS \times P_A$

$FOS = (R + \mu\sum W)/ P_A \geq 1.55$

Maximum pressure at the toe



Pressure below the Retaining Wall

Let the resultant R due to $\sum W$ and P_a lie at a distance x from the toe.

$X = \sum M/\sum W$,

$\sum M$ = sum of all moments about toe.

Eccentricity of the load = e = (b/2-x) < b/6

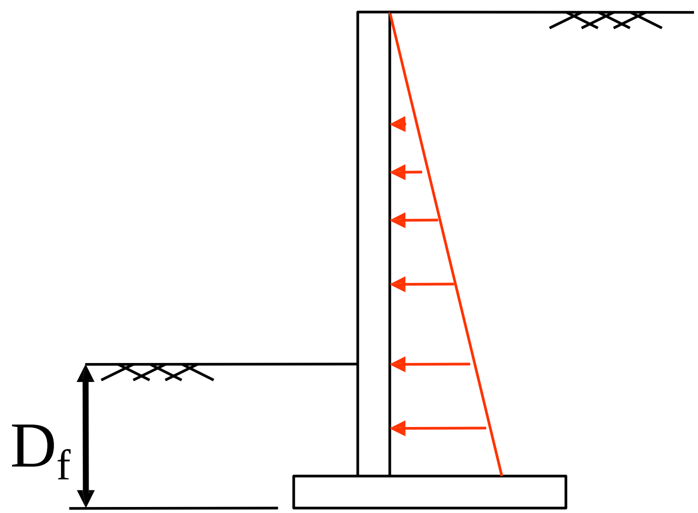
Minimum pressure at heel= $P_{min} = \frac{\sum W}{b}\left[1 - \frac{6e}{b}\right]$ >Zero.

For zero pressure, e=b/6, resultant should cut the base within the middle third.

Maximum pressure at toe= $P_{max} = \frac{\sum W}{b}\left[1 + \frac{6e}{b}\right]$ < SBC of soil.

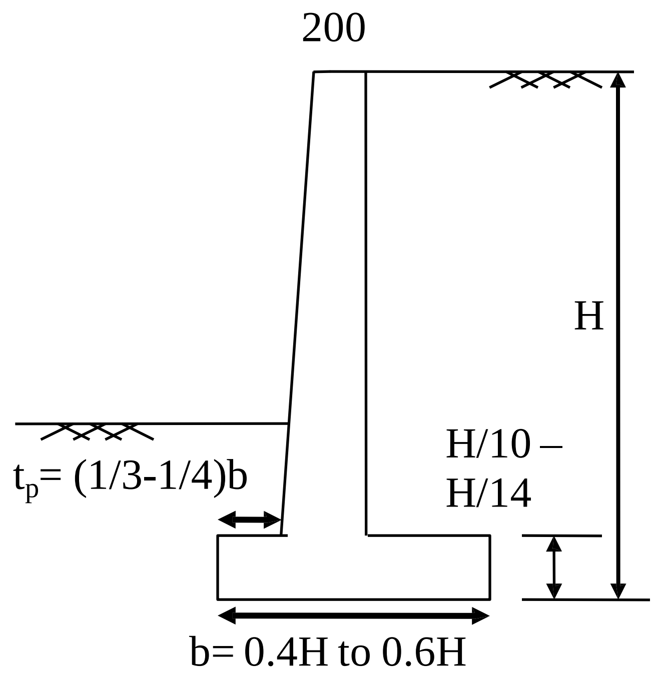
Depth of foundation

- Rankine's formula:
- $$D_f = \frac{SBC}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2$$
-
- $$= \frac{SBC}{\gamma} k_a{}^2$$
-



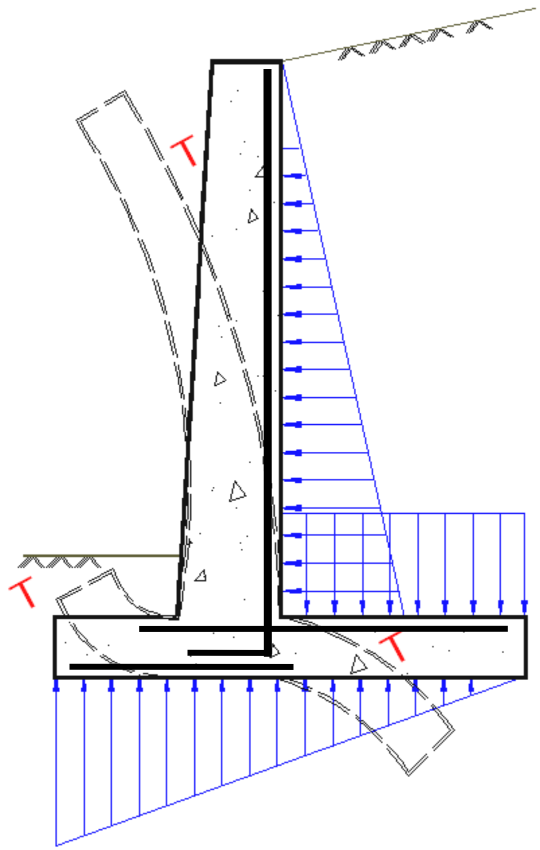
Preliminary Proportioning (T shaped wall)

- Stem: Top width 200 mm to 400 mm
- Base slab width b= 0.4H to 0.6H, 0.6H to 0.75H for surcharged wall
- Base slab thickness= H/10 to H/14
- Toe projection= (1/3-1/4) Base width



Behaviour or structural action

Behaviour or structural action and design of stem, heel and toe slabs are same as that of any cantilever slab.



DEFORMATION UNDER LOADING

Design of Cantilever RW

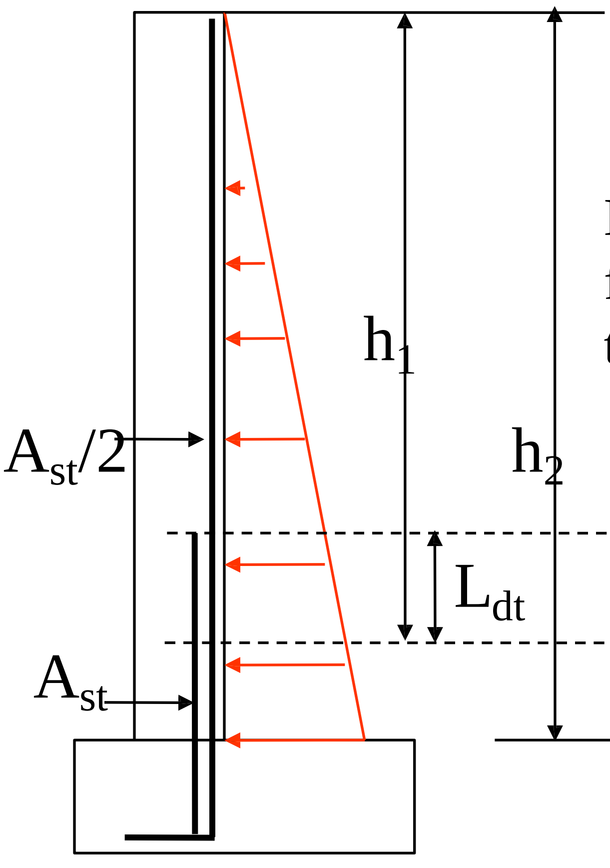
- Stem, toe and heel acts as cantilever slabs

- Stem design: M_u=psf (k_a $\gamma H^3/6$)
- Determine the depth d from $M_u = M_{u,\,lim} = Qbd^2$

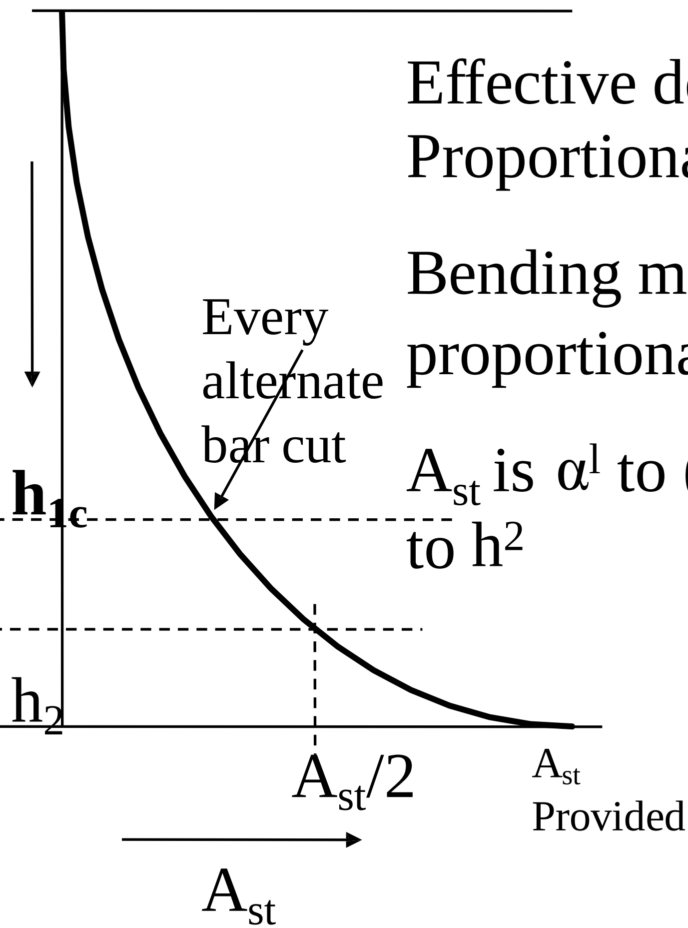
- Design as balanced section or URS and find steel

- $M_u=0.87\, f_y\, A_{st}[d - f_y A_{st}/(f_{ck} b)]$

Curtailment of bars



Cross section



Curtailment curve

Effective depth (d) is Proportional to h

Bending moment is proportional to h^3

A_{st} is α^l to (BM/d) and is α^l to h^2

$$i.e.\ \frac{A_{st1}}{A_{st2}} \approx \frac{h_1^2}{h_2^2}$$

Design of Heel and Toe

1. Heel slab and toe slab should also be designed as cantilever. For this stability analysis should be performed as explained and determine the maximum bending moments at the junction.
2. Determine the reinforcement.
3. Also check for shear at the junction.
4. Provide enough development length.
5. Provide the distribution steel

Cantilever RW design

Design a cantilever retaining wall (T type) to retain earth for a height of 4m. The backfill is horizontal. The density of soil is $18 kN/m^3$. Safe bearing capacity of soil is $200 kN/m^2$. Take the co-efficient of friction between concrete and soil as 0.6. The angle of repose is 30°. Use M20 concrete and Fe415 steel.

Solution

Data: $h' = 4m$, $SBC = 200 kN/m^2$, $\gamma = 18 kN/m^3$, $\mu = 0.6$, $\varphi = 30°$

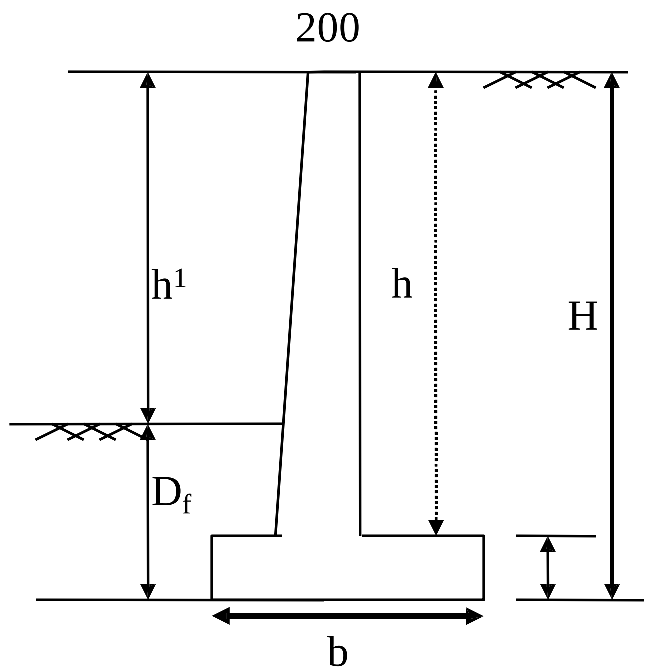
Depth of foundation

- To fix the height of retaining wall [H]
- $H = h' + D_f$

- Depth of foundation

-
- $D_f = \dfrac{SBC}{\gamma}\left[\dfrac{1-\sin\phi}{1+\sin\phi}\right]^2$

- = 1.23m say 1.2m ,
- Therefore H= 5.2m



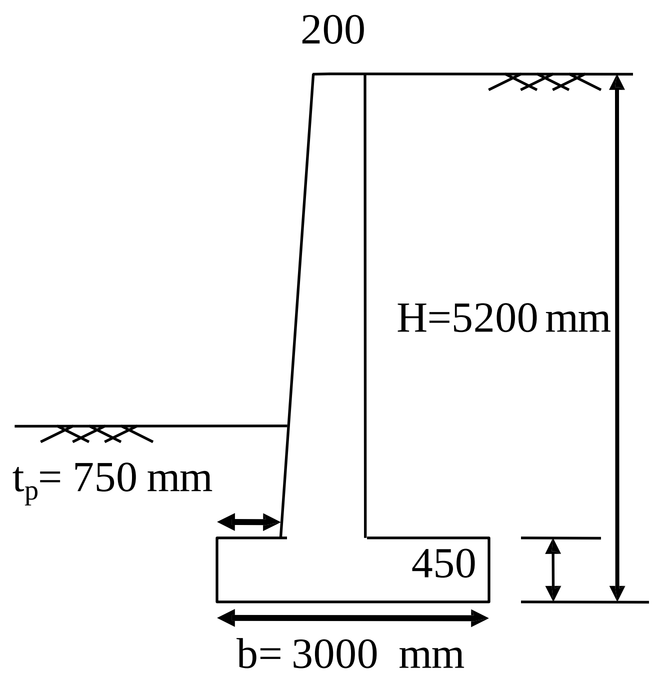
Proportioning of wall

- Thickness of base slab=(1/10 to1/14)H
- 0.52m to 0.43m, say 450 mm

- Width of base slab=b = (0.5 to 0.6) H
- 2.6m to 3.12m say 3m

- Toe projection= pj= (1/3 to ¼)H
- 1m to 0.75m say 0.75m

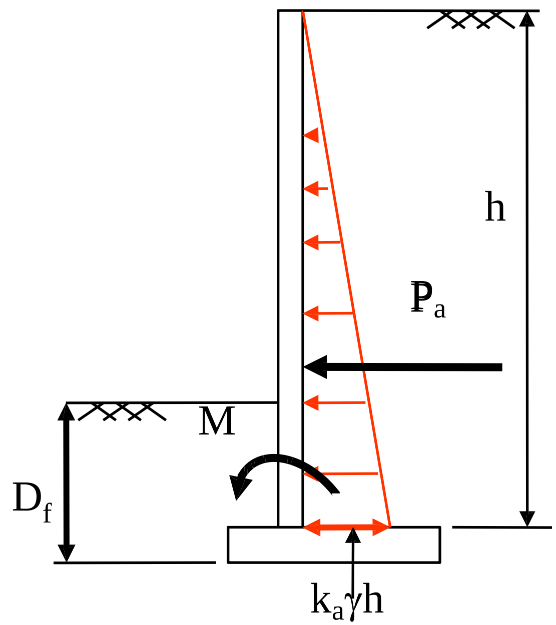
- Provide 450 mm thickness for the stem at the base and 200 mm at the top



Design of stem

P_h= ½ x 1/3 x 18 x 4.75^2=67.68 kN
M = P_h h/3 = 0.333 x 18 x 4.75^3/6
= 107.1 kN-m
M_u= 1.5 x M = 160.6 kN-m

Taking 1m length of wall,
M_u/bd^2= 1.004 < 2.76, URS
(Here d=450- eff. Cover=450-50=400 mm)
To find steel
P_t=0.295% <0.96%
A_{st}= 0.295x1000x400/100 = 1180 mm^2
#12 @ 90 < 300 mm and 3d ok
A_{st} provided= 1266 mm^2 [0.32%]



Or $M_u = [k_a \gamma H^3]/6$

Curtailment of bars-Stem

Curtail 50% steel from top

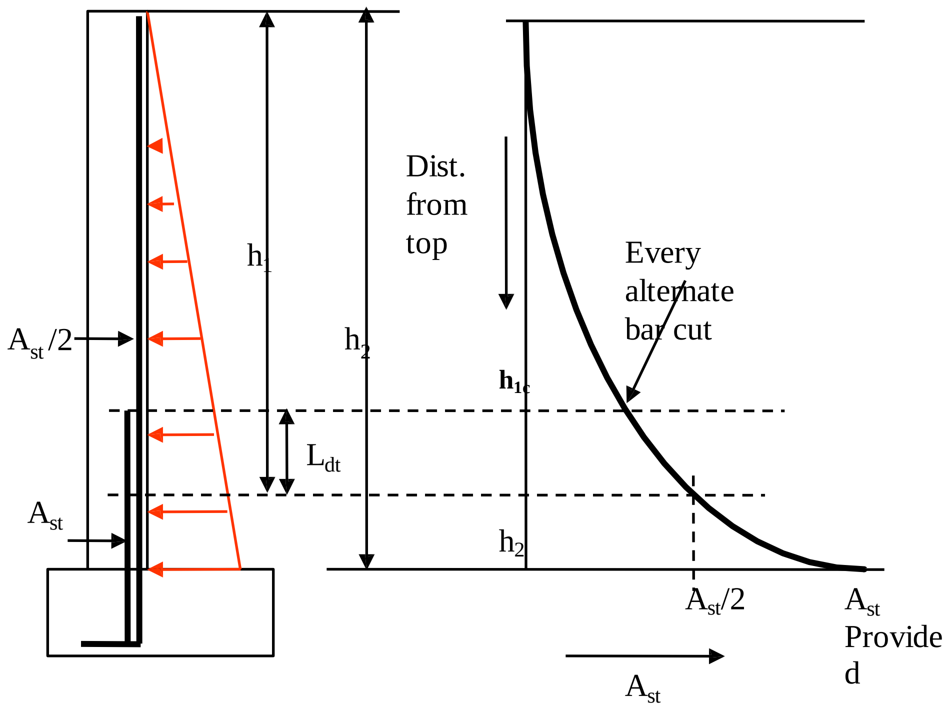
$(h_1/h_2)^2 = 50\%/100\% = ½$

$(h_1/4.75)^2 = ½$, $h_1 = 3.36$m

Actual point of cutoff

$= 3.36 - L_d = 3.36 - 47\ \phi_{bar} = 3.36 - 0.564 = 2.74$m from top.

Spacing of bars = 180 mm c/c < 300 mm and 3d ok



Design of stem-Contd.,

Development length (Stem steel)

$L_d = 47\ \varphi_{bar} = 47 \times 12 = 564$ mm

Secondary steel for stem at front

0.12% GA

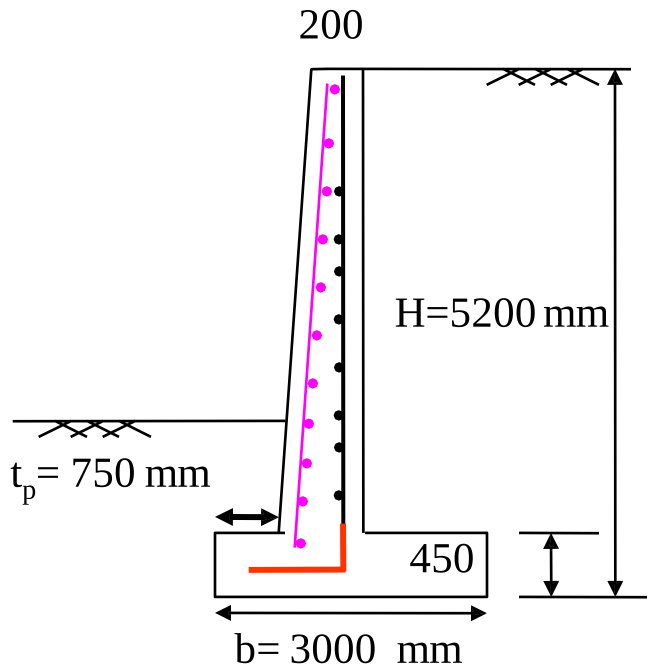
= 0.12x450 x 1000/100 = 540 mm^2

#10 @ 140 < 450 mm and 5d ok

Distribution steel

= 0.12% GA = 0.12x450 x 1000/100
= 540 mm^2

#10 @ 140 < 450 mm and 5d ok



Check for shear

Max. SF at Junction, xx = P_h=67.68 kN

Ultimate SF= V_u=1.5 x 67.68 = 101.52 kN

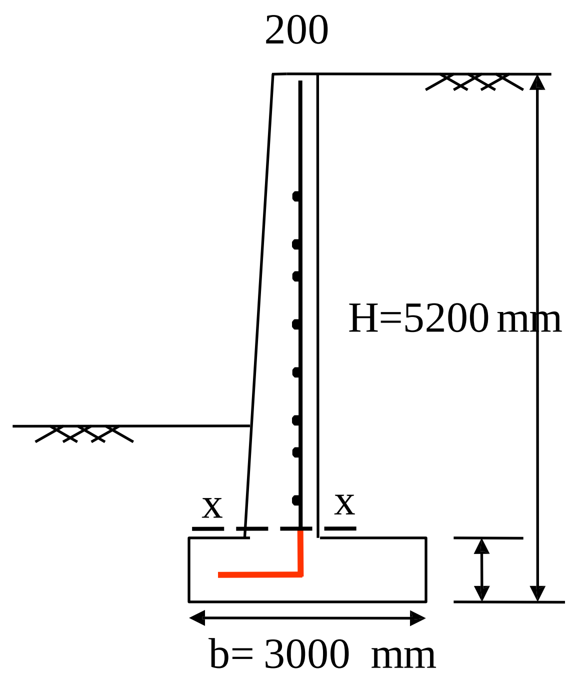
Nominal shear stress =ζ_v=V_u/bd

= 101.52 x 1000 / 1000x400 = 0.25 MPa

To find ζ_c: 100A_{st}/bd = 0.32%,

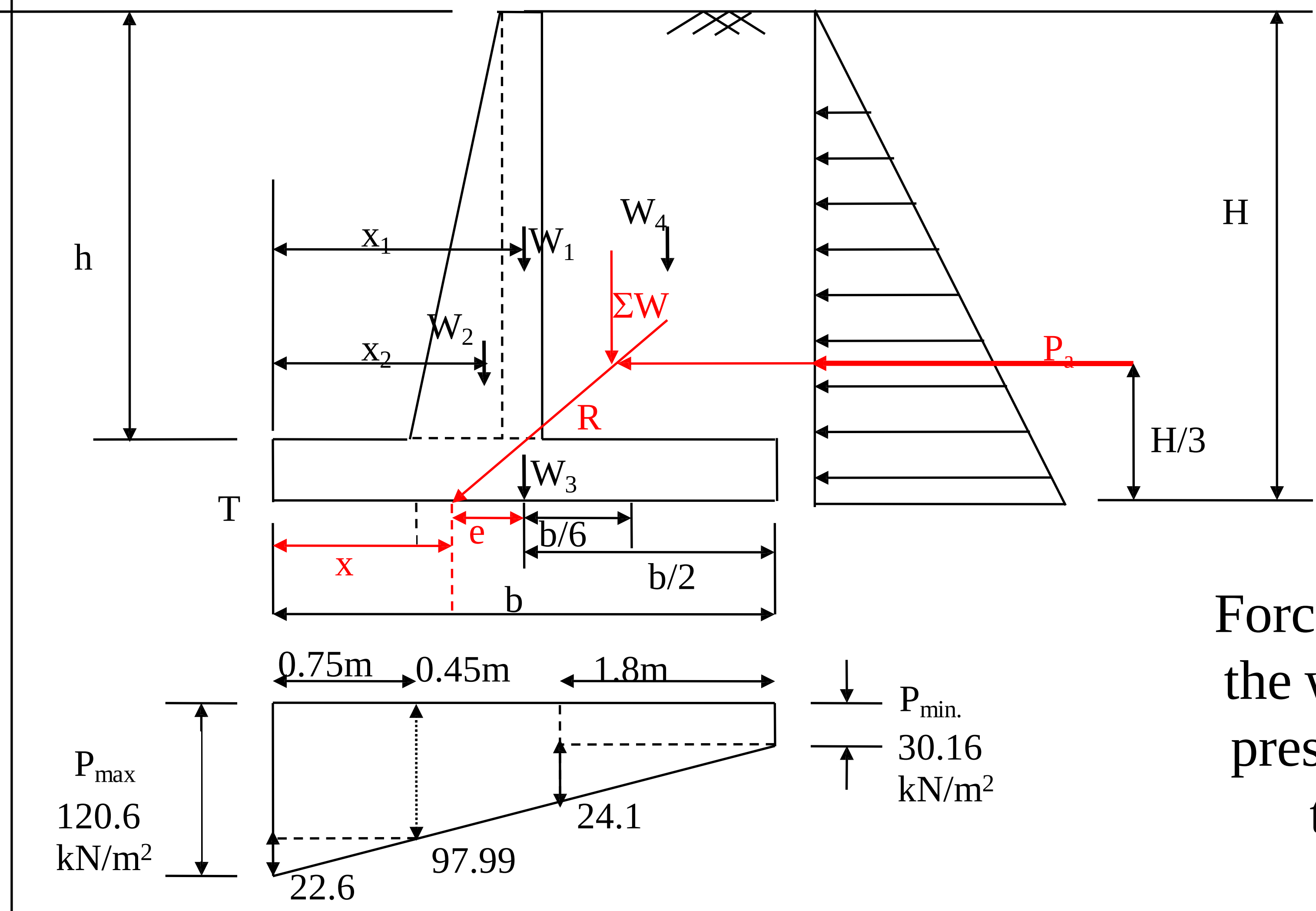
From IS:456-2000, ζ_c= 0.38 MPa

$\zeta_v < \zeta_c$, Hence safe in shear.



Stability analysis

Load	Magnitude, kN	Distance from A, m	BM about A kN-m
Stem W1	0.2x4.75x1x25 = 23.75	1.1	26.13
Stem W2	½ x0.25x4.75x1x25 = 14.84	0.75 + 2/3x0.25 =0.316	13.60
B. slab W3	3.0x0.45x1x25=33.75	1.5	50.63
Back fill, W4	1.8x4.75x1x18 = 153.9	2.1	323.20
Total	**ΣW= 226.24**		**ΣM_R=413.55**
Earth Pre. =P_H	P_H =0.333x18x5.2^2/2	H/3 =5.2/3	**M_O=140.05**



Pressure below the Retaining Wall

Forces acting on the wall and the pressure below the wall

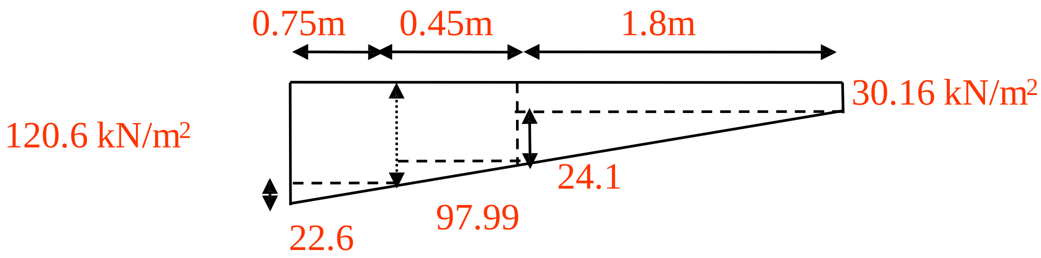
Stability checks

- Check for overturning
- FOS = $\Sigma M_R / M_O$ = 2.94 >1.55 safe

- Check for Sliding
- FOS = $\mu \Sigma W / P_H$ = 2.94 >1.55 safe

- Check for subsidence
- $X = \Sigma M / \Sigma W$ = 1.20 m > b/3 and e = b/2 – x = 3/2 – 1.2 = 0.3m < b/6

- Pressure below the base slab
- P_{Max} = 120.66 kN/m^2 < SBC, safe
- P_{Min} = 30.16 kN/m^2 > zero, No tension or separation, safe



Pressure below the Retaining Wall

Load	Magnitude, kN	Distance from C, m	BM, $M_{C,}$ kN-m
Backfill	153.9	0.9	138.51
Heel slab	0.45x1.8x25 = 27.25	0.9	18.23
Pressure dist. rectangle	30.16 x 1.8 =54.29	0.9	-48.86
Pressure dist. Triangle	½ x 24.1 x1.8=21.69	1/3x1.8	-13.01
Total Load		**Total**	**ΣM_C=94.86**

Design of heel slab

Unit –IV: SHALLOW FOUNDATIONS: Strength Criteria, Settlement criteria &PILE FOUNDATION

Types, choice of foundation, location of depth, safe bearing capacity, Terzaghi's, Meyerhof, Skempton and IS methods, safe bearing pressure based on N- value, allowable bearing pressure, safe bearing capacity, plate load test, allowable settlements of structures, types of piles, load carrying capacity of piles based on static pile formulae in dynamic pile formulae, pile load tests, load carrying capacity of pile groups in sands and clays, Settlement of pile groups.

SHALLOW FOUNDATION

- Foundation is shallow if its depth is equal to or less than its width.

- **Types of shallow foundations**

1. Spread footings
2. Combined footings
3. Strap or cantilever footings
4. Mat or raft foundation

1. Spread footings

- Spreads the super-imposed load of a wall or a column over a larger area.
- Used where the loads are light or there are strong shallow soils.
- Spread footing may be of the following types –

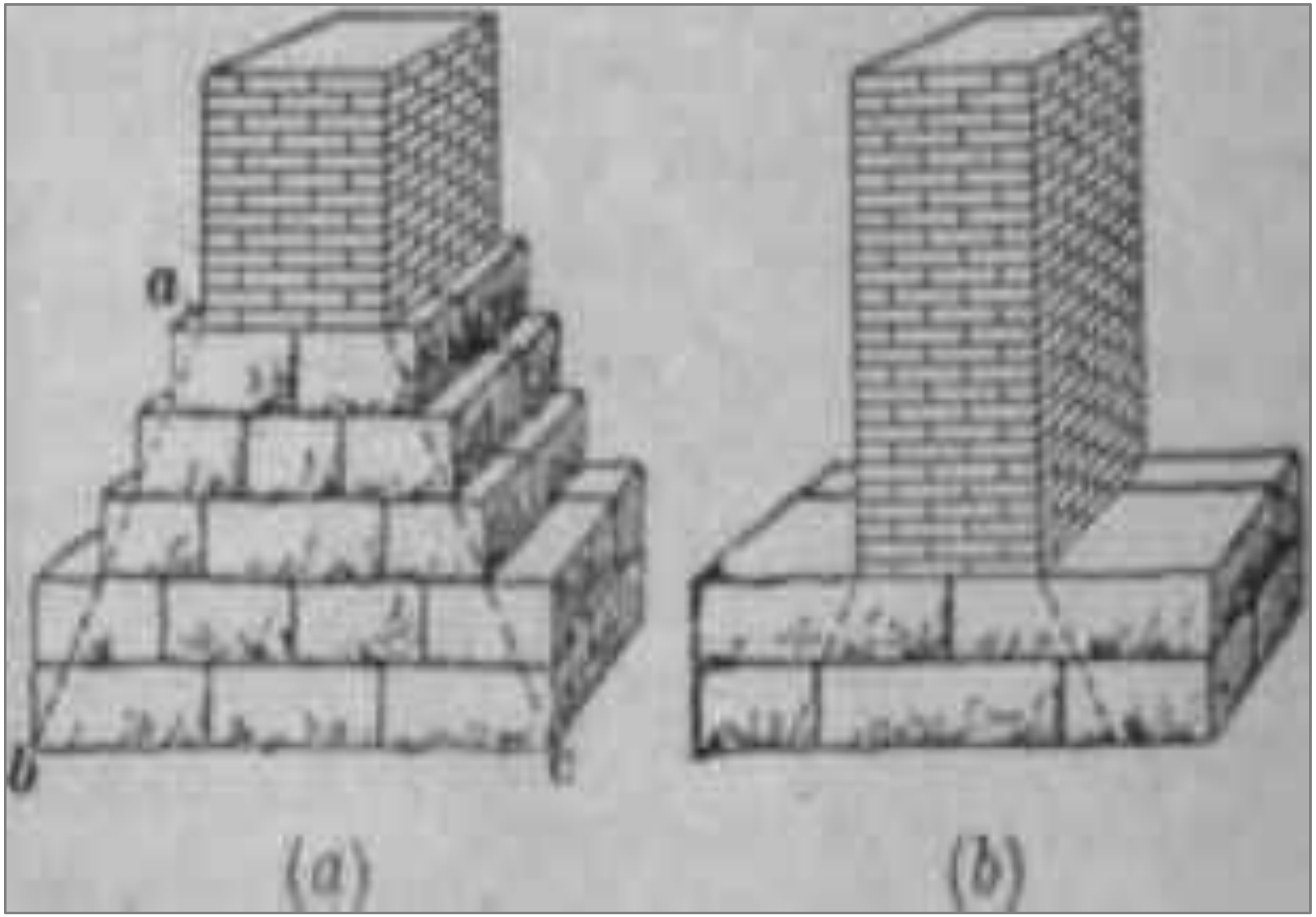
1) Single column footing
2) Stepped column footing

Pad footing

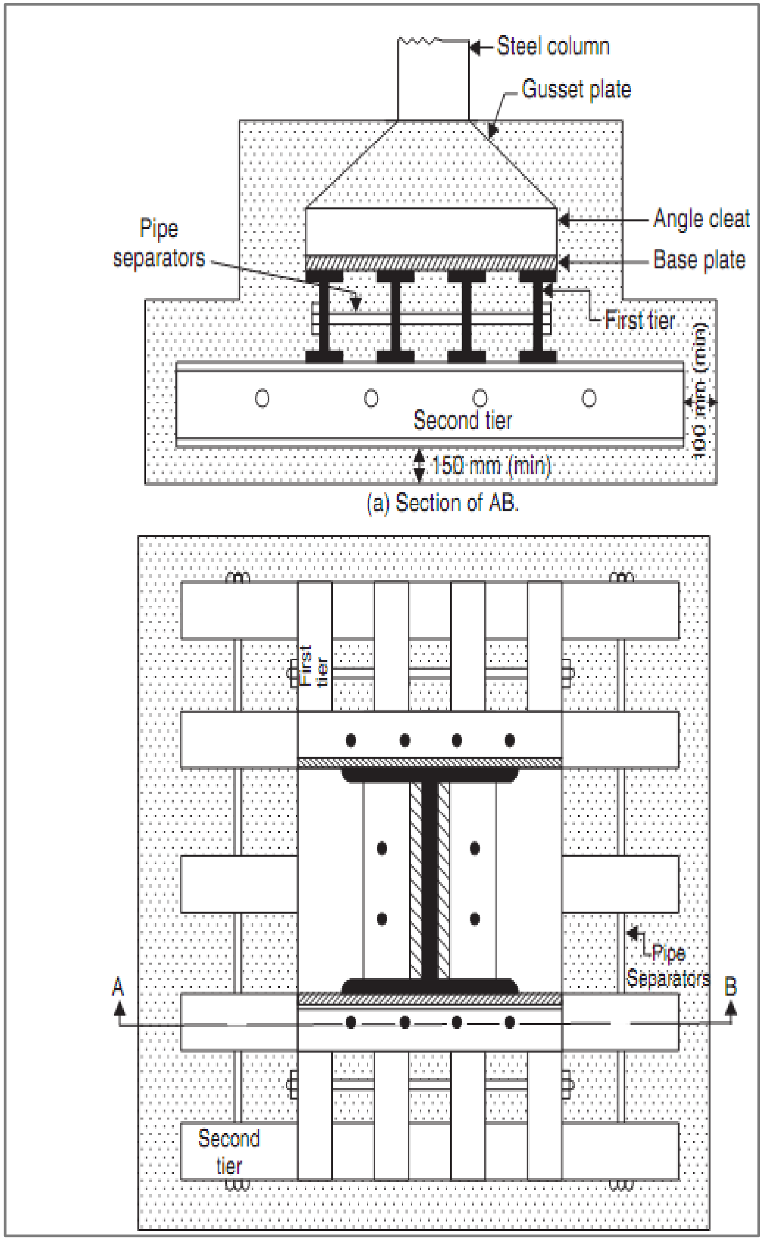
3) Slopped column footing
4) Simple Wall footing

strip footing

5) Stepped wall footing
6) Grillage foundation



a)Stepped wall footing
b)Simple Wall footing



Grillage foundation

2. Combined footings

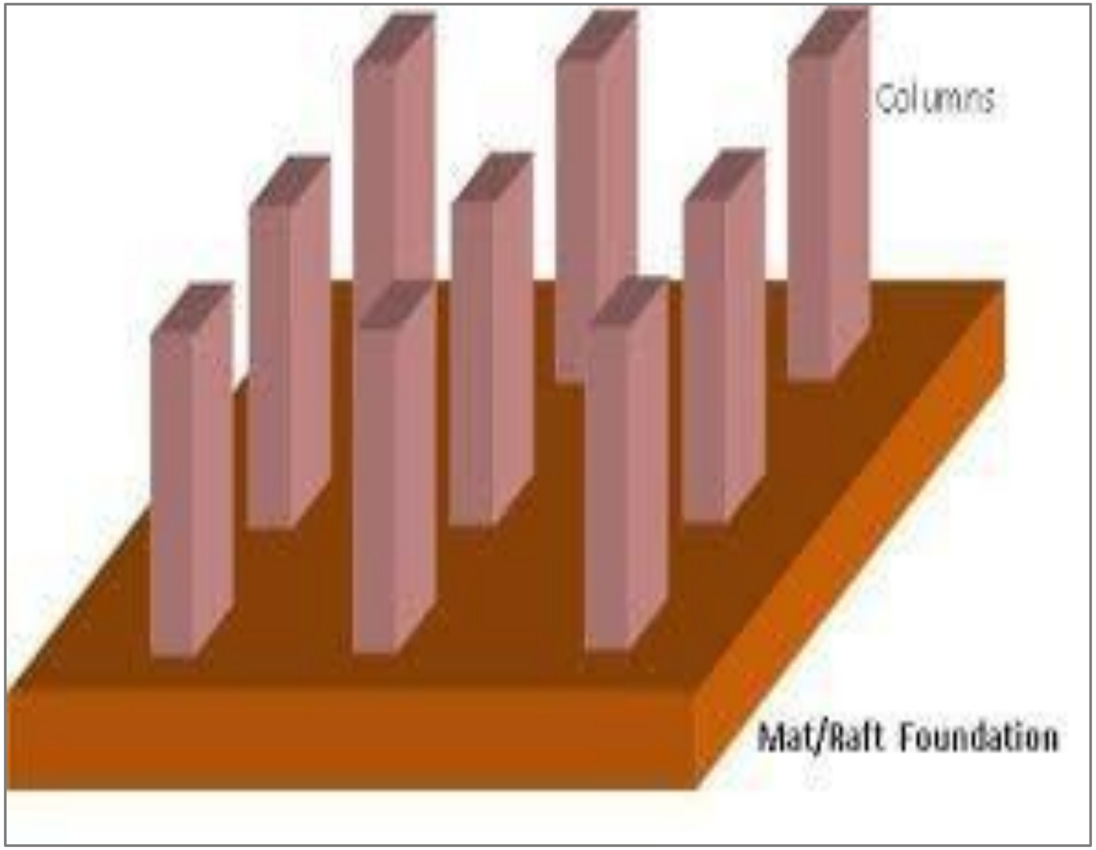
- Are preferred when 2 individual column are close to each other.
- It is economical.
- Provided when bearing capacity of soil is less, requiring more area under individual footing.
- Combined footing may be rectangular, Trapezoidal or column-wall.
- If the columns carry equal loads, the footing is of rectangular shape, otherwise its trapezoidal shape.

3. Strap or cantilever footings

- Independent footings of two columns are connected by a beam.
- The strap beam does not remain in contact with soil, and thus does not transfer any pressure to the soil.

4. Mat or raft foundation

- It is a combined footing that covers the entire area beneath a structure and supports all the walls and columns.
- Used on soft or loose soils with low bearing capacity as they can spread loads over larger area.


Columns
Mat/Raft Foundation

Pile Foundations

Types of piles, load carrying capacity of piles based on static pile formulae in dynamic pile formulae, pile load tests, load carrying capacity of pile groups in sands and clays, settlement of pile groups.

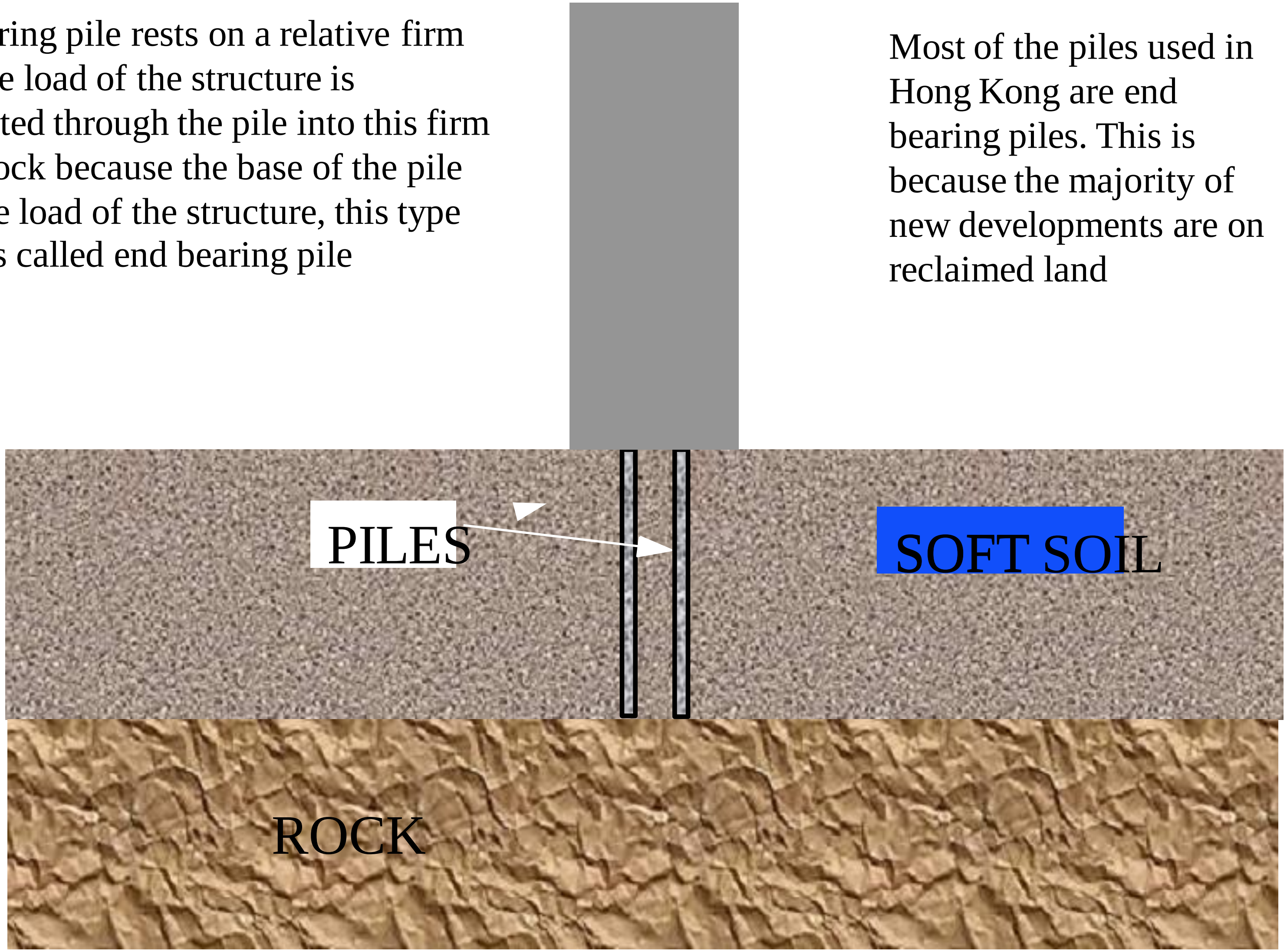
Pile Foundations

- BS8004 defines deep foundation with D>B or D>3m.
- Pile foundation always more expensive than shallow foundation but will overcome problems of soft surface soils by transferring load to stronger, deeper stratum, thereby reducing settlements.
- Pile resistance is comprised of
  - end bearing
  - shaft friction
- For many piles only one of these components is important. This is the basis of a simple classification

End Bearing Piles

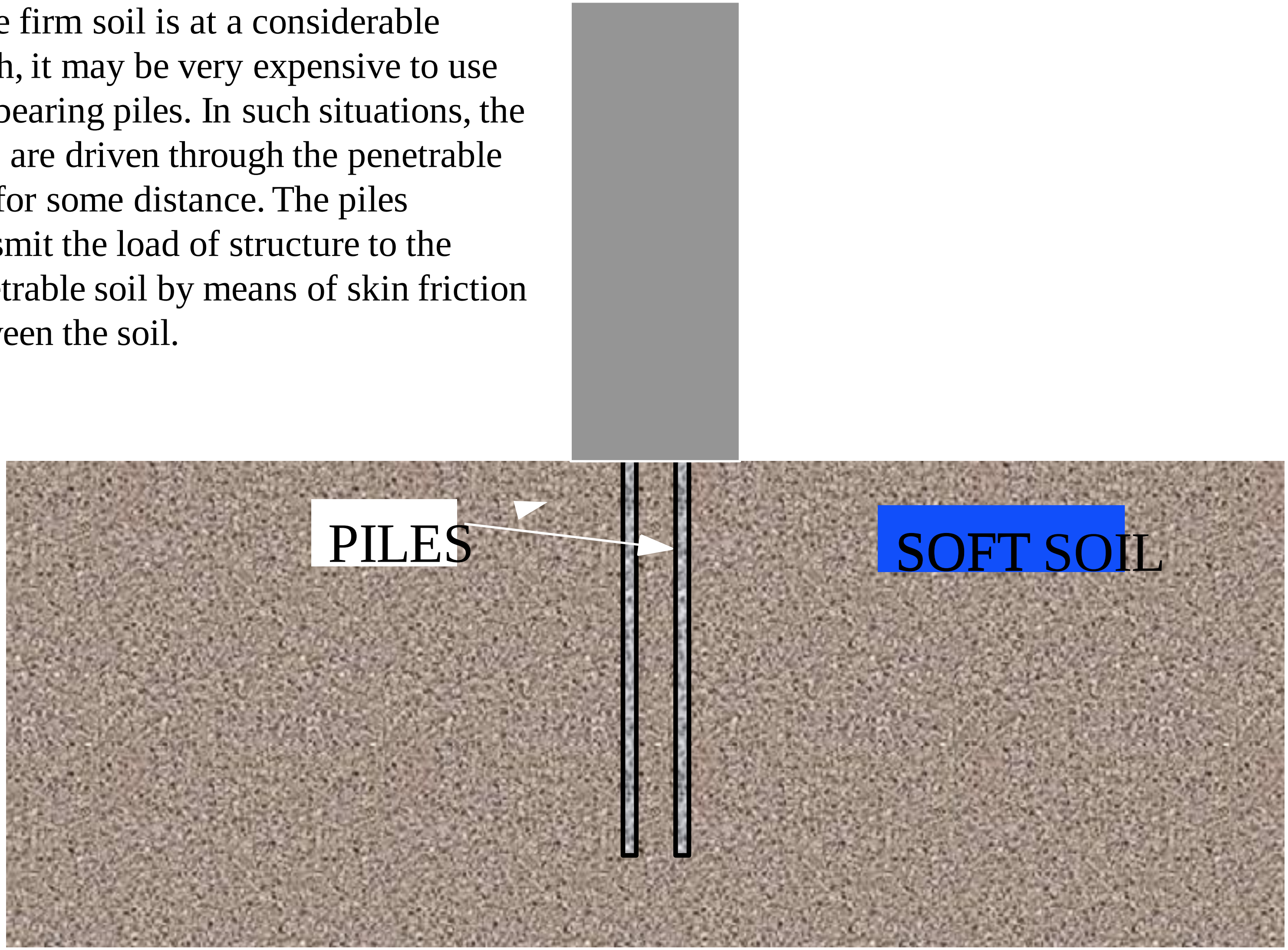
End bearing pile rests on a relative firm soil . The load of the structure is transmitted through the pile into this firm soil or rock because the base of the pile bears the load of the structure, this type of pile is called end bearing pile

Most of the piles used in Hong Kong are end bearing piles. This is because the majority of new developments are on reclaimed land



Friction Piles

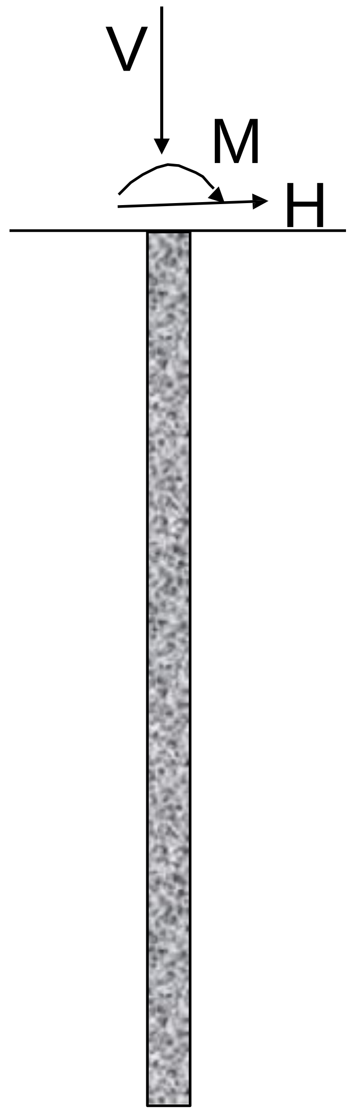
If the firm soil is at a considerable depth, it may be very expensive to use end bearing piles. In such situations, the piles are driven through the penetrable soil for some distance. The piles transmit the load of structure to the penetrable soil by means of skin friction between the soil.



Types of Pile

- The pile installation procedure varies considerably, and has an important influence on the subsequent response
- Three categories of piles are classified by method of installation as below:
  - Large displacement piles
    - They encompass all solid driven piles including precast concrete piles, steel or concrete tubes closed at the lower end
  - Small displacement piles
    - They include rolled steel sections such as H-pile and open-end tubular piles
  - Replacement piles
    - They are formed by machine boring, grabbing or hand-digging

Loads applied to Piles



- Combinations of vertical, horizontal and moment loading may be applied at the soil surface from the overlying structure
- For the majority of foundations the loads applied to the piles are primarily vertical
- For piles in jetties, foundations for bridge piers, tall chimneys, and offshore piled foundations the lateral resistance is an important consideration
- The analysis of piles subjected to lateral and moment loading is more complex than simple vertical loading because of the soil-structure interaction.
- Pile installation will always cause change of adjacent soil properties, sometimes good, sometimes bad.

Modes of failure

- The soil is always failure by punching shear.
- The failure mode of pile is always in buckling failure mode.

Total and Effective Stress Analysis

- To determine drained or undrained condition, we may need to consider the following factors:
  - Drainage condition in the various soil strata
  - Permeability of soils
  - Rate of application of loads
  - Duration after the application of load
- A rough indicator will be the Time Factor ($T_v = c_v t / d^2$)

Displacement Pile (A/D)

Advantage	Disadvantages
Pile material can be inspected for quality before driving	May break during driving
Construction operation affect by ground water	Noise and vibration problems
Can driven in very long lengths	Cannot be driven in condition of low headroom
Construction operation not affected by ground water	Noise may prove unacceptable. Noise permit may be required
Soil disposal is not necessary	Vibration may prove unacceptable due to presence of sensitive structures, utility installation or machinery

Replacement Pile (A/D)

Advantage	Disadvantages
Less noise or vibration problem	Concrete cannot be inspected after installation
Equipment can break up practically all kinds of obstructions	Liable to squeezing or necking
Can be installed in conditions of low headroom	Raking bored pile are difficult to construct
No ground heave	Drilling a number of pile groups may cause ground loss and settlement of adjacent structures
Depth and diameter can varied easily	Cannot be extended above ground level without special adaptation

Ultimate capacity of axially load single pile in soil

Estimated by designer based on soil data and somewhat empirical procedures. It is common practice that the pile capacity be verified by pile load test at an early stage such that design amendment can be made prior to installation of the project piles.The satisfactory performance of a pile is, in most cases, governed by the limiting acceptable deformation under various loading conditions. Therefore the settlement should also be checked.

Basic Concept

The ultimate bearing capacity (Q_u)of a pile may be assessed using soil mechanics principles. The capacity is assumed to be the sum of skin friction and end-bearing resistance, i.e

Q_u=Q_b+Q_s-W ……………………(1)

where

Q_u total pile resistance,

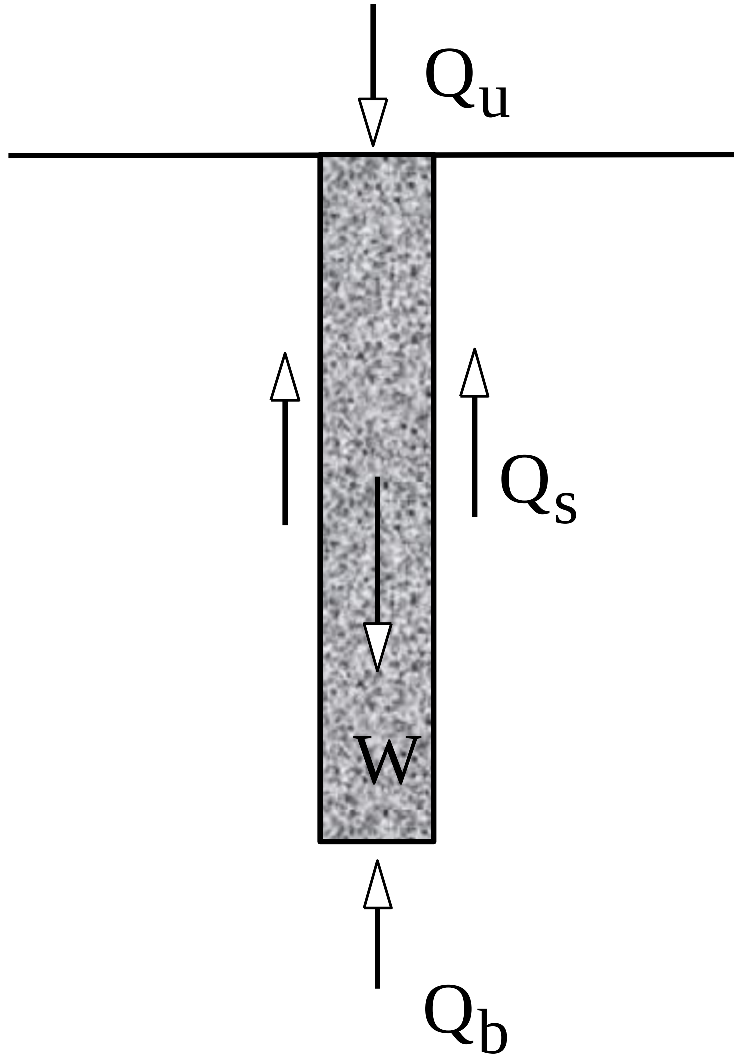
Q_b is the end bearing resistance and

Q_s is side friction resistance

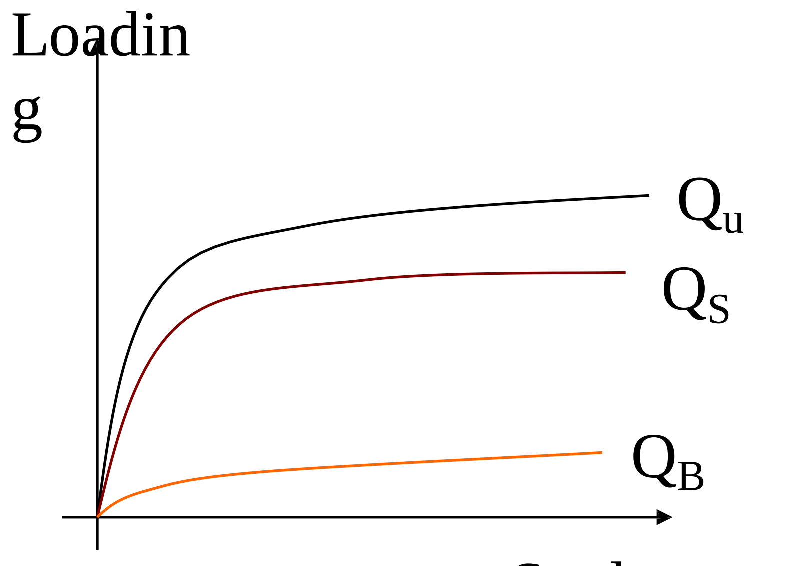
General behaviour

Shaft resistance fully mobilized at small pile movement (<0.01D)

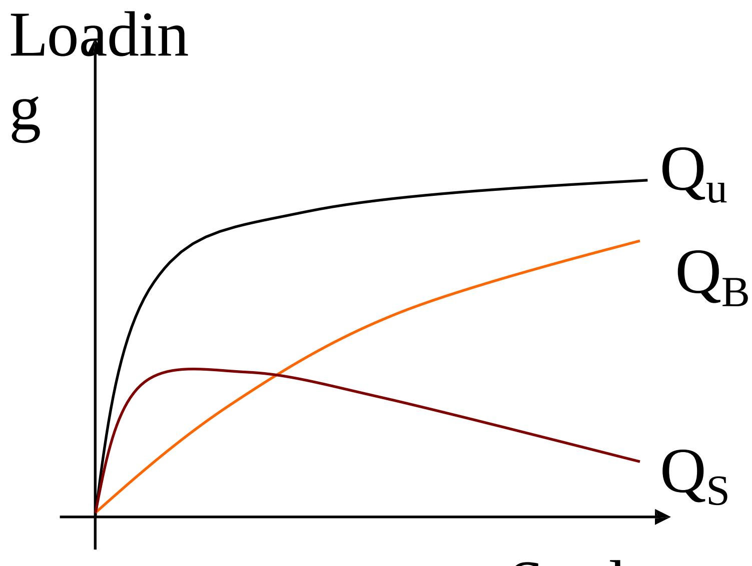
Base resistance mobilized at large movement (0.1D)



$$Q_u = Q_s + Q_b - W$$



Behaviour of Frictional Pile



Behaviour of End Bearing Pile

- **Piles founded on dense soils**
  - **Important to adopt good construction practice to enhance shaft friction and base resistance**
  - **Shaft and base grouting useful in enhancing pile capacity**

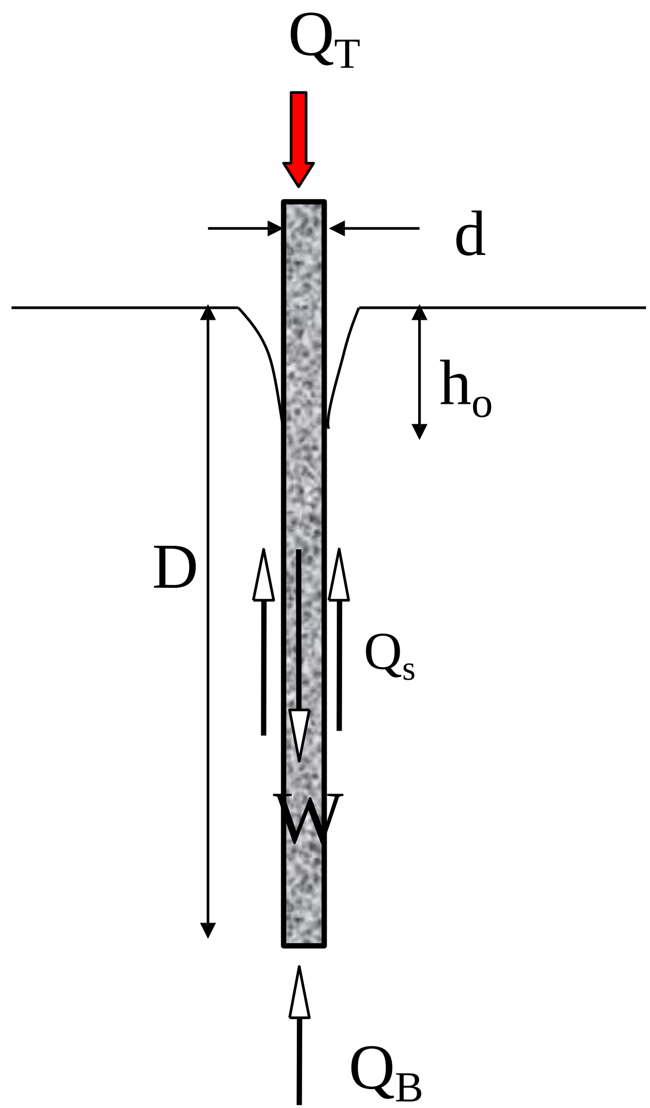
- Piles founded on strong stratum
  - Not much benefit in enhancing base resistance
  - Important to adopt good construction practice to enhance shaft friction
  - Shaft grouting useful in enhancing pile capacity

Ultimate Limit State Design

$$Q_{DES} = Q_B/F_B + Q_s/F_s - W \ldots\ldots\ldots(2)$$

Where F_B and F_S is the factor of safety of components of end bearing strength and shaft friction strength

$$Q_U = Q_B + Q_s - W \ldots\ldots(3)$$



End Bearing Resistance

Assumptions

1. The weight of the pile is similar to the weight of the soil displaced of the pile

   $\Rightarrow W_p = A_b P_o$

2. The length (L) of the pile is much greater than its width d

   $\Rightarrow W_p = A_b P_o + A_b \gamma \, d N_\gamma / 2$

3. Similarly for $\phi > 0$, N_q approximately equal to $N_q - 1$

$$Q_b = A_b[c_b N_c + P_o(N_q - 1) + \cancel{\gamma d/2 N_\gamma} + \cancel{P_o}] \cancel{- W_p}$$

$$\Rightarrow Q_b = A_b[c_b N_c + P_o N_q]$$

End Bearing resistance for Bore pile in granular soils

Due to the natural of granular soil, the c' can be assumed equation to zero. The ultimate end bearing resistance for bored pile in granular soils may be express in terms of vertical effective stress, σ'_v and the bearing capacity factors N_q as :

$$Q_B = A_B\ N_q\ \sigma_v'$$

N_q is generally related to the angle of shearing resistance ϕ'. For general design purposed, it is suggested that the N_q value proposed by Berezantze et al (1961) as presented in Figure ?? are used. However, the calculated ultimate base stress should conservatively be limited to 10Mpa, unless higher values have been justified by load tests.

Shaft Friction Resistance

The ultimate shaft friction stress q_s for piles may be expressed in terms of mean vertical effective stress as :

$q_s = c' + K_s \sigma_v' \tan\delta_s$

$q_s = \beta \sigma_v'$ (when $c'=0$)

Where

K_s= coefficient of horizontal pressure which depends on the relative density and state of soil, method of pile installation, and material length and shape of pile. Ks may be related to the coefficient of earth pressure at rest,

$K_0 = 1 - \sin\phi$ as shown in Table 1.

Q_v' = mean vertical effective stress

σ_s' = angle of friction along pile/soil interface (see table2)

β= shafte friction coefficient (see Table 3)

$Q_s = pLq_s$

Where p is the perimeter of the pile and L is the total length of the pile

Driven pile in Granular soils

The concepts of the calculation of end-bearing capacity and skin friction for bored piles in granular soils also apply to driven piles in granular soils. The pile soil system involving effects of densification and in horizontal stresses in the ground due to pile driving. In Hong Kong, it is suggested that the value of q_b be range from 16 to 21Mpa.

Bored pile in Clays

The ultimate end bearing resistance for piles in clays is often related to the undrained shear strength, c_u, as

$q_B = N_c c_u$

$Q_B = A_B N_c c_u$

where

$N_c = 9$ when the location of the pile base below ground surface exceeds fours times the pile diameter

Bored pile in Clays

The ultimate shaft friction (q_s) for soils in stiff over-consolidated clays may be estimated on the semi-empirical method as:

$q_s = \alpha C_u$

α is the adhesion factor (range from 0.4 to 0.9)

Driven Pile in Clays

The design concepts are similar to those presented for bored piles in granular soils. However, based on the available instrumented pile test results, a design curve is put forward by Nowacki et al (1992)

Prediction of Ultimate Capacity of Pile

Pile Driving Formula

Pile driving formula relate the ultimate bearing capacity of driven piles to final set (i.e. penetration per blow). In Hong Kong, the Hiley formula has been widely used for the design of driven piles as:

$R_d = (\eta_h W_h d_h)/(s + c/2)$

Where

R_d is driving resistance, η_h is efficiency of hammer, W_h is the weight of hammer, d_h is the height of fall of hammer, s is permanent set of pile and c is elastic movement of pile

Note: Test driving may be considered at the start of a driven piling contract to assess the expected driving characteristics.

Prediction of Ultimate Capacity of Pile

Pile Load Test

Static pile load test is the most reliable means of determining the load capacity of a pile. The test procedure consists of applying static load to the pile in increments up to a designated level of load and recording the vertical deflection of the pile. The load is usually transmitted by means of a hydraulic jack placed between the top of the pile and a beam supported by tow or more reaction piles. The vertical deflection of the top of the pile is usually measured by mechanical gauges attached to a beam, which span over the test pile.

UNIT 5: WELL FOUNDATIONS

types, different shapes of wells, components of wells, sinking of well, tilts and shifts.

COURSE OUTLINES

- Introduction
- Types of Well or Caissons
- Components of a Well Foundation
- Shapes of Wells
- Depth of a Well Foundation
- Forces Acting on Well Foundation
- Lateral Stability of Well Foundation
- Construction and Sinking of a Well

INTRODUCTION

What is a Well or Caisson?

- **Large size prismatic or cylindrical shells which are built deep into the ground to support heavy loads.**

What is a Well Foundation?

- **Large hollow open-ended structure which is generally built in parts and sunk through ground or water to its final position, where it forms part of the permanent foundation.**

What is Caisson?

Caisson is a water tight structure made of wood, steel, R.C.C i.e. reinforced cement constructed in connection with excavation for the foundation of bridges, piers in rivers, dock structures etc.

Introduction

- **Well foundation is the most commonly adopted foundation for major bridges in India. Since then many major bridges across wide rivers have been founded on wells.**
- **Well foundation is preferable to pile foundation when foundation has to resist large lateral forces.**

Uses of Well or Caisson Foundation

- **Usually very suitable in deep sandy or soft soils.**
- ***Specially for Boulder Stratum.***
- **Used to -**
  - **Support and Transfer Heavy Loads (Vertical and Horizontal) and Moments**
  - **Resist Uplift Forces**
  - **Reduce Differential Settlement**
  - **Support Heavy Structures including**
- **High Rise Buildings,**
- **Major Bridges on Rivers or Sea from Piers, Abutments of Bridges,**
- **Foundations of Heavy Machinery,**
- **Harbour Structures (Docks, Break Waters, etc.),**
- **High Tension Towers**
- **Shore Protection Works**
- **Foundations of Lighthouse, Pumphouse, etc.**

TYPES OF WELLS OR CAISSONS

- *Depending on the Method of Installation- 3Types*

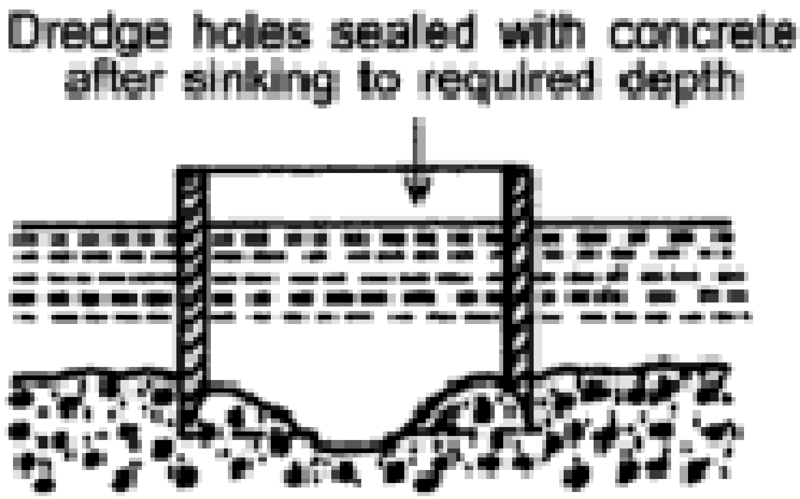
(a) **Open Caisson or Well**

(b) **Box Caisson or Floating Caisson**

(c) **Pneumatic Caisson**

OPEN CAISSON OR WELL

- Top and Bottom are kept Opened during Construction.
- Install into the ground by excavation of soil within the shaft so that it may sink into the ground either under its own weight or by addition of surcharge load.
- Normally used on sandy soils or soft bearing stratum and where no firm bed is available at a higher depth.



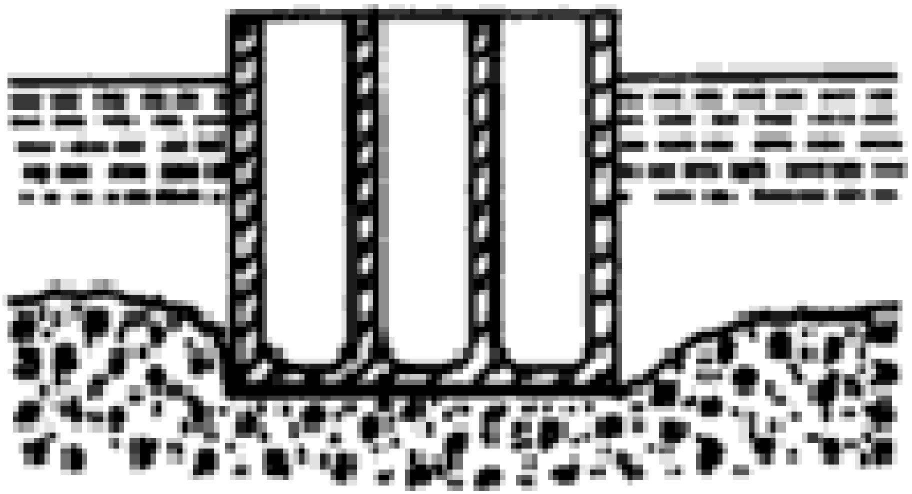
(a) Open caisson

TYPES OF OPEN CAISSON

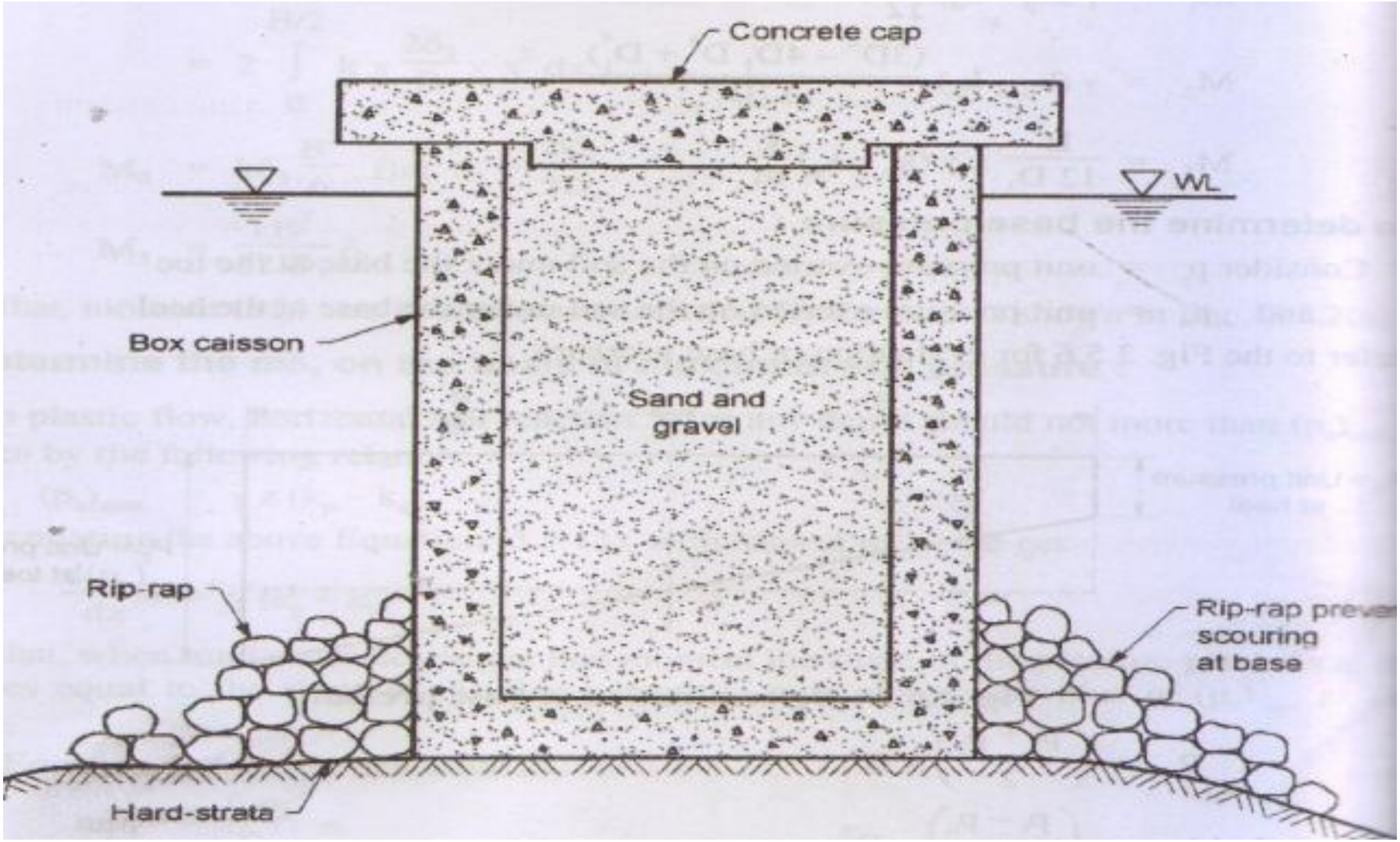
- According to the shape of caissons, open caissons can be further classified into three types as;
- **Single Wall Open Caisson**
- **Cylindrical Open Caisson**
- **Open Caissons with Dredging Wells**.

BOX CAISSON OR FLOATING CAISSON

- Open at theTop and Closed at the Bottom before Sinking.
- First built on the ground and then towed to the site where it is sunk to a prepared foundation base



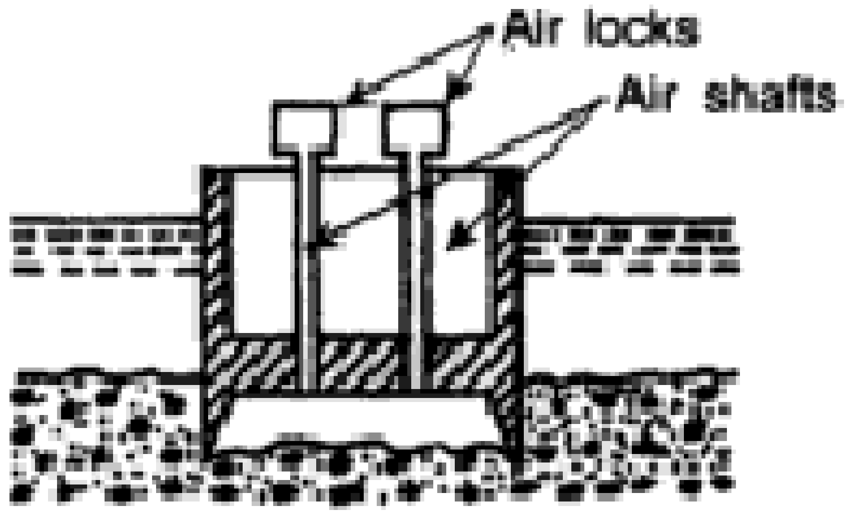
(b) Box caisson



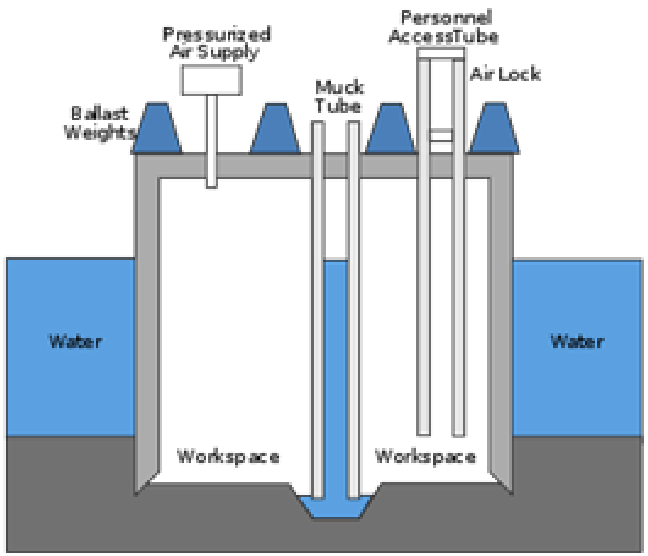
Concrete cap
WL
Box caisson
Sand and
gravel
Rip-rap
Rip-rap preve
scouring
at base
Hard-strata

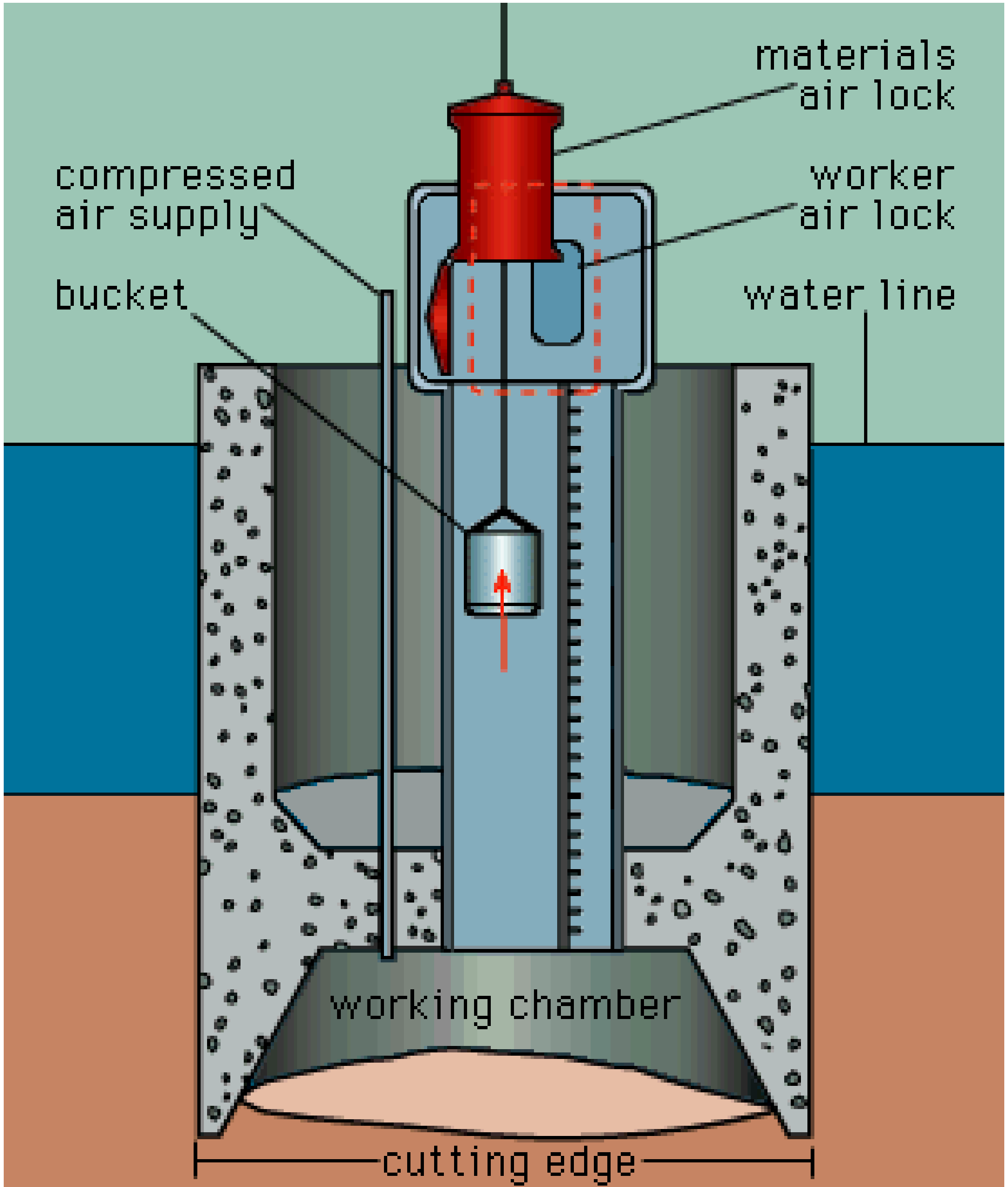
PNEUMATIC CAISSON

- Closed atTop and Open (during construction) at Bottom.
- Working chamber at the Bottom which is kept Dry by maintaining a High Air Pressure to prevent water from entering into the chamber during sinking operations.
- Generally used in underwater construction projects.



(c) Pneumatic caisson

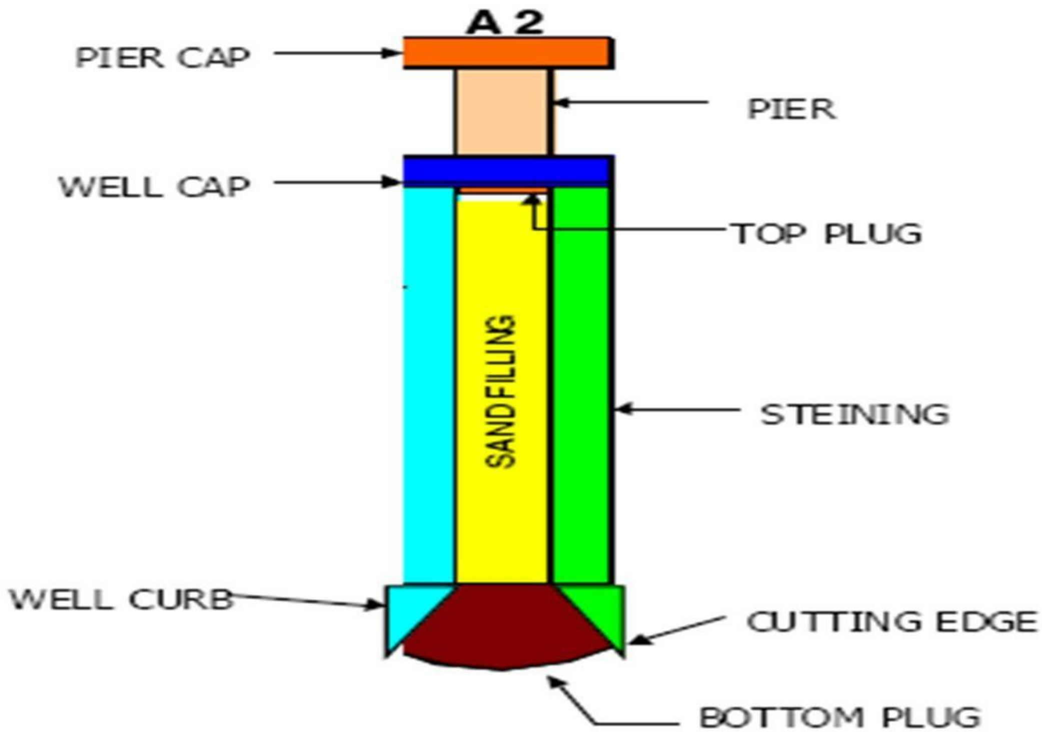


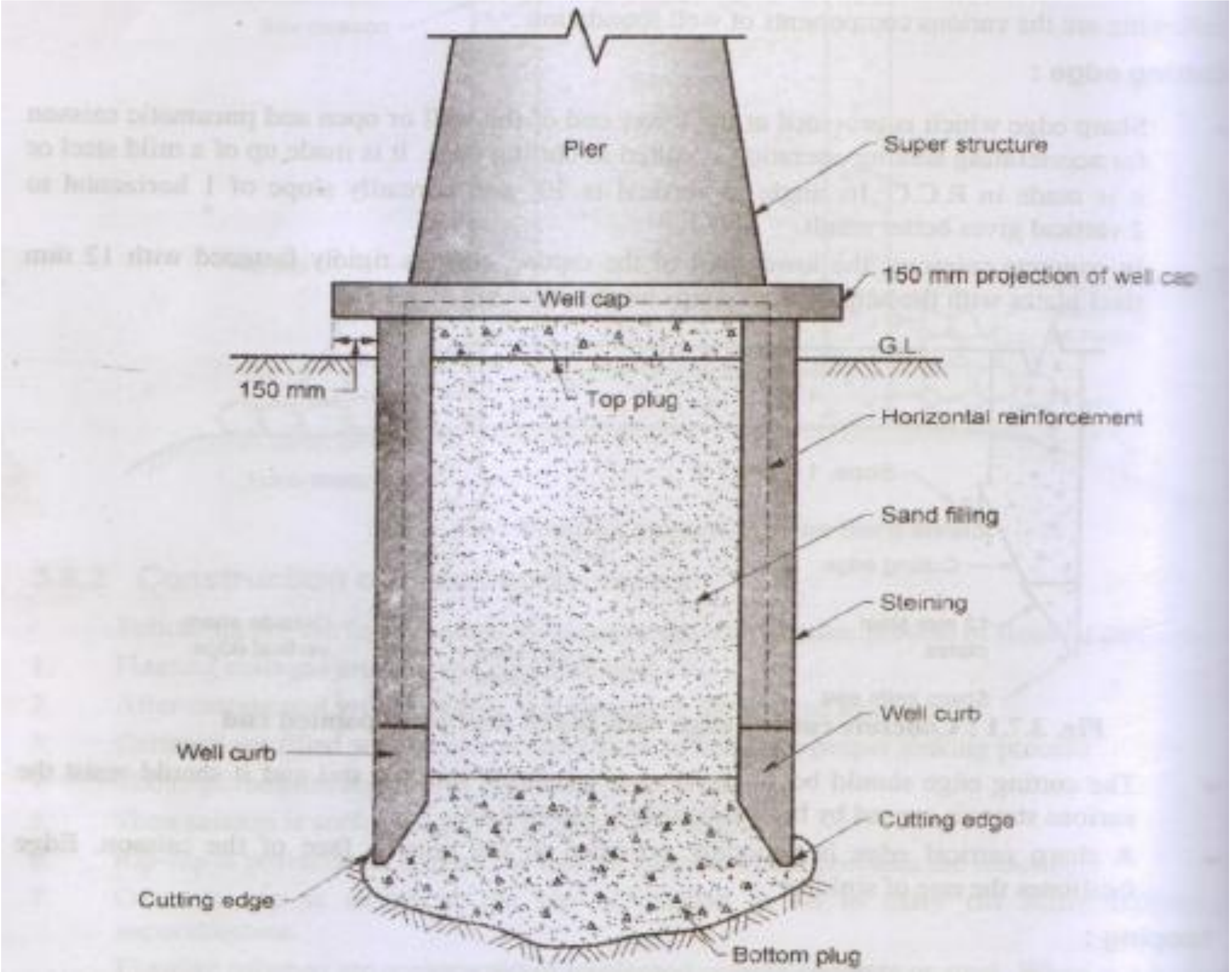


materials air lock
compressed air supply
worker air lock
bucket
water line
working chamber
cutting edge

COMPONENTS OF A WELL FOUNDATION

1. Well Cap
2. Well Steining
3. Well Curb
4. Cutting Edge
5. Bottom plug
6. Top Plug
7. Filling (Sand




Pier
Super structure
150 mm projection of well cap
Well cap
G.L.
150 mm
Top plug
Horizontal reinforcement
Sand filling
Steining
Well curb
Well curb
Cutting edge
Cutting edge
Bottom plug

SHAPES OF WELLS

1. Circular Well
2. Double D-Well
3. Dumb-Bell Well
4. Broad Necked Twin Well with Circular Dredge Holes
5. Double Octagonal with Circular Dredge Hole Well
6. Multiple Dredge Hole Well
7. Double Octagonal Well
8. Double Rectangular Well

DEPTH OF A WELL FOUNDATION

Depth of a well foundation must be such that the following requirements are met:

- **In erodible soil, there is a *minimum grip length of one third the maximum anticipated depth of scour below high flood level(HFL).***
- **In non-erodible strata, there is adequate seating and embedment on sound rock or soil and**
- **The base pressure is within permissible limits.**

SCOUR DEPTH

- For Natural Streams in Cohesionless Soils, the scour depth may be determined from Lacey's Formula.
- For Preliminary design the Normal Depth of Scour, d(in meter), below the HFL is given by

$$d = 0.473\left(\frac{Q}{f}\right)^{\frac{1}{3}}$$

Where, Q = maximum flood discharge (m^3/s)

f = silt factor

The maximum depth of scour below the HFL is given by:

D_{max} = 2 d in the vicinity of piers

$$\left.\begin{aligned} &= 1.27d\ (scour\ restrained) \\ &= 2d\ (scour\ all\ round) \end{aligned}\right\}\text{near abutments}$$

- If the river bed is not readily susceptible to scouring effect of floods, the First formula for scour depth shall not apply.
- In such cases, the maximum depth of scour shall be assessed from actual

ALLOWABLE BEARING PRESSURE

for footings on sand as modified from Teng (1962).

$$q_a = 0.14 C_b (N - 3)\left(\frac{B + 0.3}{2B}\right)^2 R'_w C_d s_a$$

where q_a = allowable net soil pressure in t/m^2,
C_b = correction factor (Peck and Bazzaraf 1969)
= 2.0 for gravelly soil,
= 1.5 for coarse to medium sand, and
= 1.0 for fine and silty sand,
N = standard penetration resistance corrected for overburden and dilatancy,
B = diameter or equivalent width of well in m,
R'_w = water table correction factor
= 0.5 for water level at or above the base of well,
D = depth of well from maximum scour level in m, and
C_d = depth correction factor = $1 + (D/B) \leq 2$

FORCES ACTING ON WELL FOUNDATION

Vertical loads

- **Self-weight of well**
- **Buoyancy**
- **Dead load of super structure, substructure**
- **Live load**
- **Kentledge during sinking operation**
- **Impact load due to live load only in the design of pier cap and bridge seat on the abutment**

FORCES ACTING ON WELL FOUNDATION

Horizontal loads

- Braking and tractive effort of moving vehicles
- Forces due to resistance of bearing
- Forces due to water current or waves
- Centrifugal force, if the bridge is situated on a curve
- Wind forces
- Seismic forces
- Earth pressure
- Other horizontal and uplift forces due to provision of transmission line tower
- Temperature stresses

CONSTRUCTION AND SINKING OF A WELL

CONSTRUCTION OF A WELL

1) Construction of caisson curb or well curb

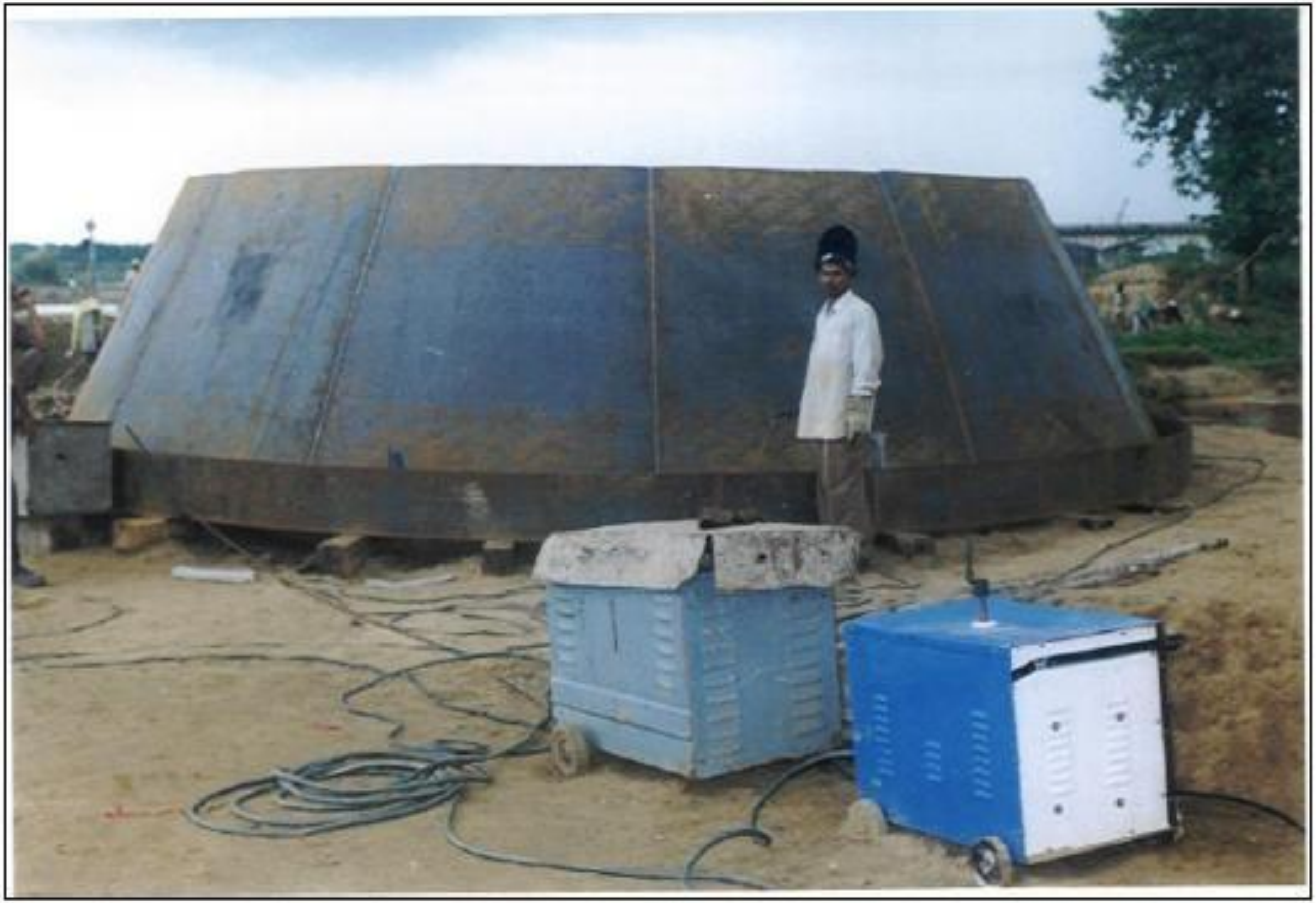
- Well curb or caisson curb is built in case of a dry river bed, so as to place at the correct position after excavating the bed for about 15cm for seating.
- If the depth of water is upto 5m, then sand island is constructed.
- For even distribution of load, wooden sleeps can be placed below cutting edge.
- When the shuttering of caisson curb is done, then reinforcement for the curb is placed in position.
- Concreting of curb is done in one stroke and it should be done without gap so as to obtain monolithic concreting structure.

Dimension of sand Island > 3 times dia of well

Cutting Edge

Cutting Edge

Fabrication of well curb



Well Curb Reinforcement

Well Curb Reinforcement

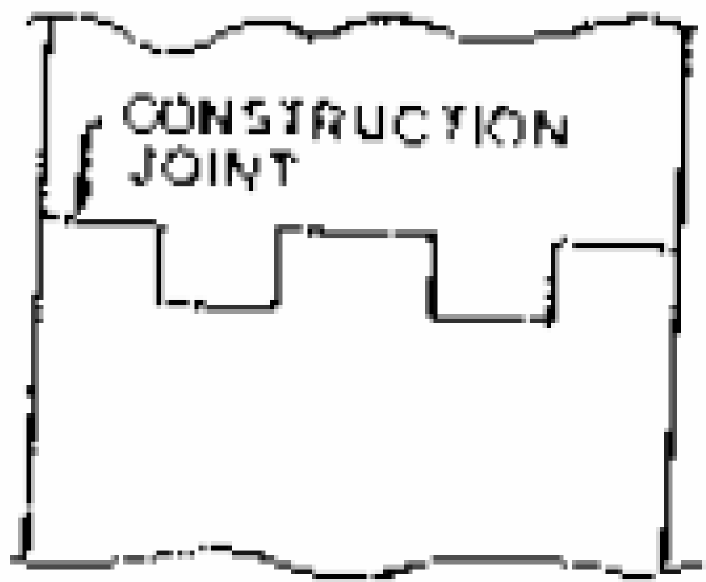
CONSTRUCTION OF A WELL

2) Construction of Caisson steining or well stening:

- **For a height of 1.5 m, the caisson steining is constructed at a time when the caisson reaches at a depth of 6 m below ground steining can be raised with a height of 3 m at a time.**
- **Sinking of Caisson is commenced after concrete is set for 24 hrs.**

Well Steining

Shear keys in the construction joint of well steining



SINKING OF A WELL

- **Sinking operation consists of the following steps:**

1) **Laying the well curb**
2) **Well steining**
3) **Sinking process**

SINKING PROCESS

- When the curb is cast, then sinking operation is started.
- The first stage of stening is ready after curing. In inner material if comes as a obstruction can be excavated manually or mechanically. If hard rock comes in the way, then blasting may be done.
- For proper sinking operation, additional loading termed as kentledge is used if required. Sand bags can be used as kentledge which can be placed on a suitable platform on the top surface of caisson.
- When caisson reaches at a depth of 10 m, dewatering is done by pumping. Jetting of water is also helpful in sinking operation. Proper care has been done by adopting the proper measures and techniques so as to avoid shifts and tilts of caisson during sinking

Top Plugging

Tilting and Shifting of Well or Caissons

- **At the time of sinking process well or caisson should sink exact vertically downward, straight and at the corner position without any tilting of well.**
- **If the well tilt any one side from its position while sinking operation, then it is called as tilting of well.**
- **During sinking operation, it may also shift away from the required position.**
- **Hence it is much essential to take the suitable precautions so as to avoid tilting and shifting of well.**

Precautions to be taken to Avoid Tilts and Shift

- **The cutting edge of well or caisson must be thick and sharp pointed.**
- **The external surface of steining and well or caisson curb should be smooth.**
- **Dredging should be done uniformly on all sides and in all pockets of well or caisson.**
- **Well or Caisson should be symmetrically placed.**
- **The diameter (D) of the curb must be placed from 40 mm to 80 mm or larger or more than external diameter of steining.**

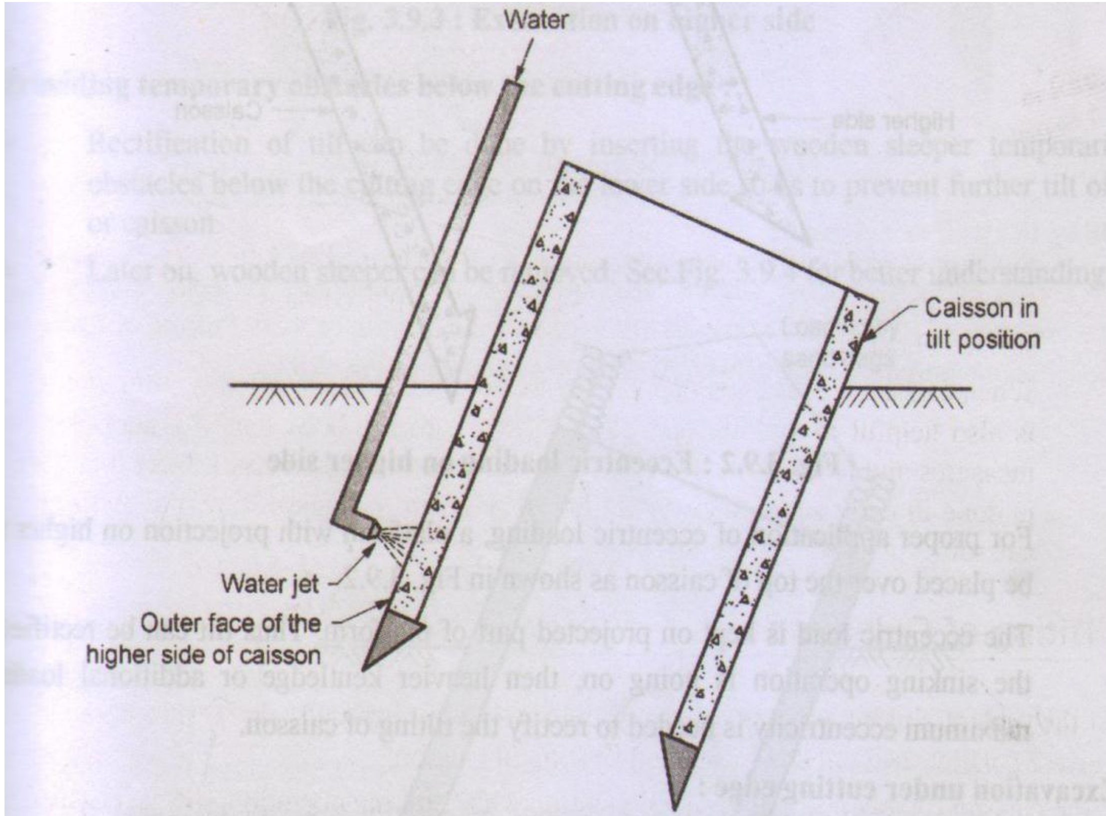
Remedial Measures to Rectify Tilt and Shift

- *Following are the remedial measures to be carefully implemented to avoid tilting and shifting of well or caisson during sinking process:*

1) **Water Jetting**
2) **Eccentric Loading or Kentledge**
3) **Excavation under Cutting Edge**
4) **Regulation of Excavation**
5) **Providing Temporary Obstacles Below the Cutting Edge**
6) **Pushing the Well with Jack**
7) **Pulling the Well**
8) **Strutting the Well**

1) Water Jetting

- *Used to prevent tilting*.
- **In this method, water jet is forcedly applied on tilt.**
- **Application of *water jetting on higher side reduces skin friction.* Thus the tilting is rectified.**
- ***Not more effective but gives the better result if used with the combination of other methods.***



Water

Caisson in tilt position

higher side of caisson

2) Eccentric Loading or Kentledge

- Well is normally given the additional loading called kentledge in order to have necessary sinking effort. In this method, eccentric loading or kentledge is applied in higher side so as to have greater sinking effort.
- For proper application of eccentric loading a platform with projection on higher side can be placed over the top of caisson
- The eccentric load is kept on projected part of platform. Thus tilt can be rectified.

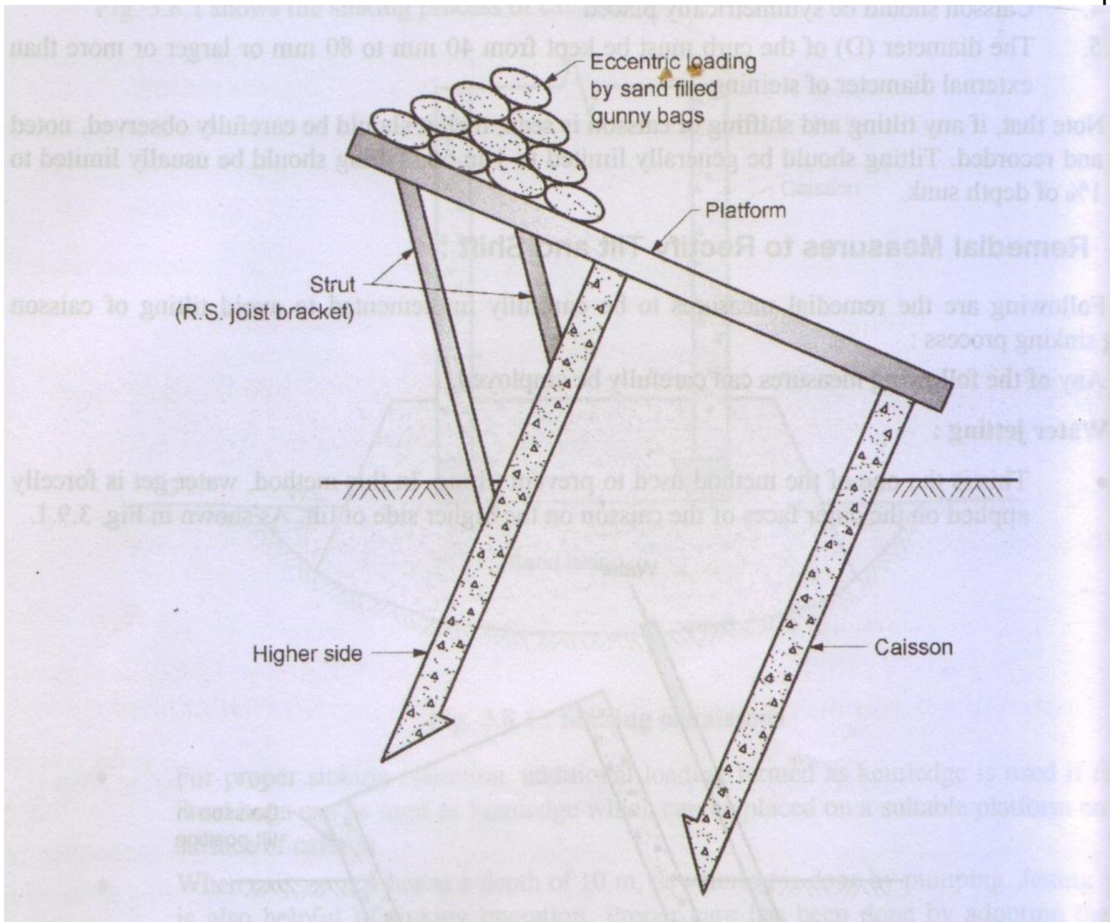


Fig. 3.9.2 : Eccentric loading on higher side

3) Excavation under Cutting Edge

- During sinking process, filled well will not set or straighten due to unbroken stiff strata on its higher side.
- In such situation, dewatering is preferably done to loosen stiff strata. If dewatering is not possible or unsafe, then drivers are sent to loosen the stiff strata.
- Sometimes if possible and safe, an open excavation is done under the cutting edge.

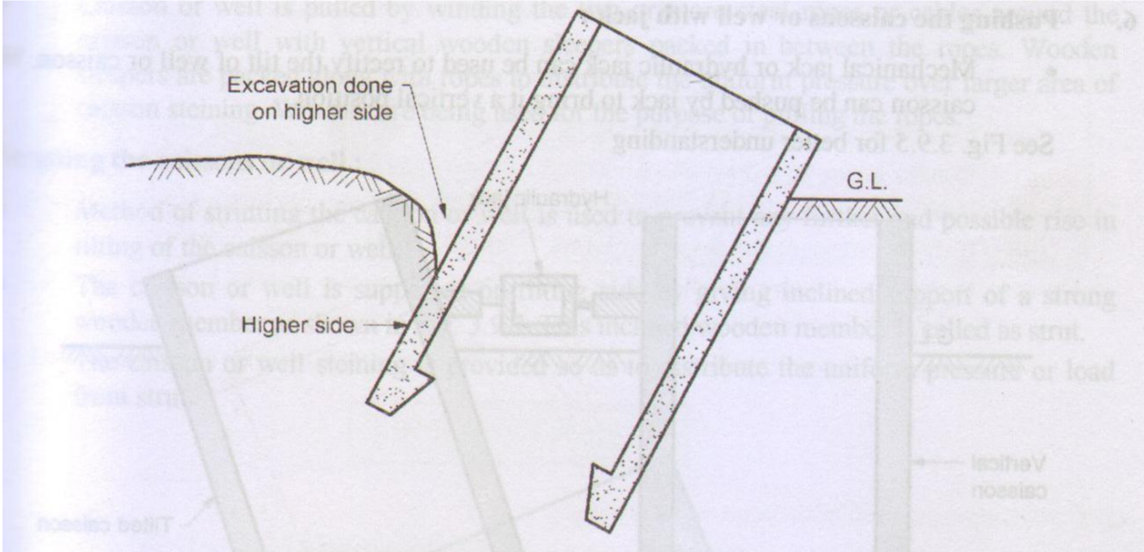


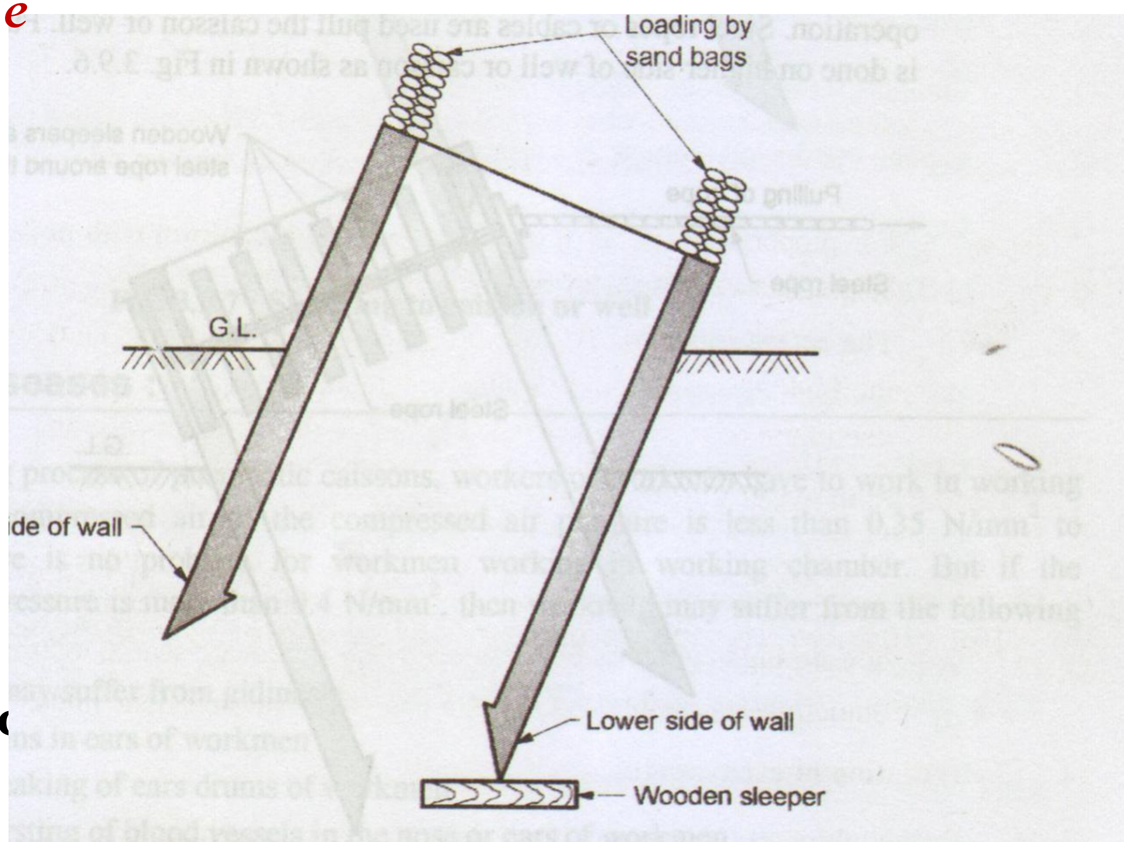
Fig. 3.9.3 : Excavation on higher side

4) Regulation of Excavation

- **Sinking of well on higher side due to excess excavation is more.**
- **This is all right in the early stages, otherwise dewatering of caisson or well is needed and open excavation may be done on higher side.**

5) Providing Temporary Obstacles Below the Cutting Edge

- **Rectification of tilt can be done by *inserting the wooden sleeper temporarily as an obstacles below the cutting edge* on the lower side so as to prevent further tilt of the well or caisson.**
- **Later on, wooden sleeper can be removed for better understanding.**



6) Pushing the Well with Jack

- Mechanical jack or hydraulic jack can be used to rectify the tilt of well or caisson.
- Well or caisson can be pushed by jack to bring it a vertical position.

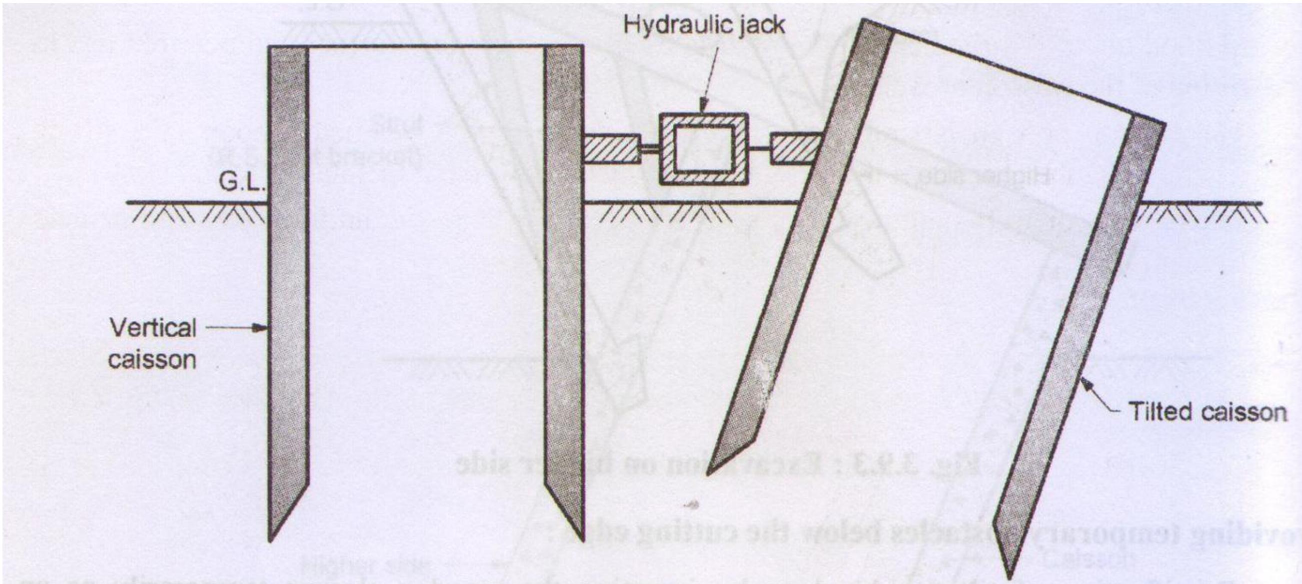
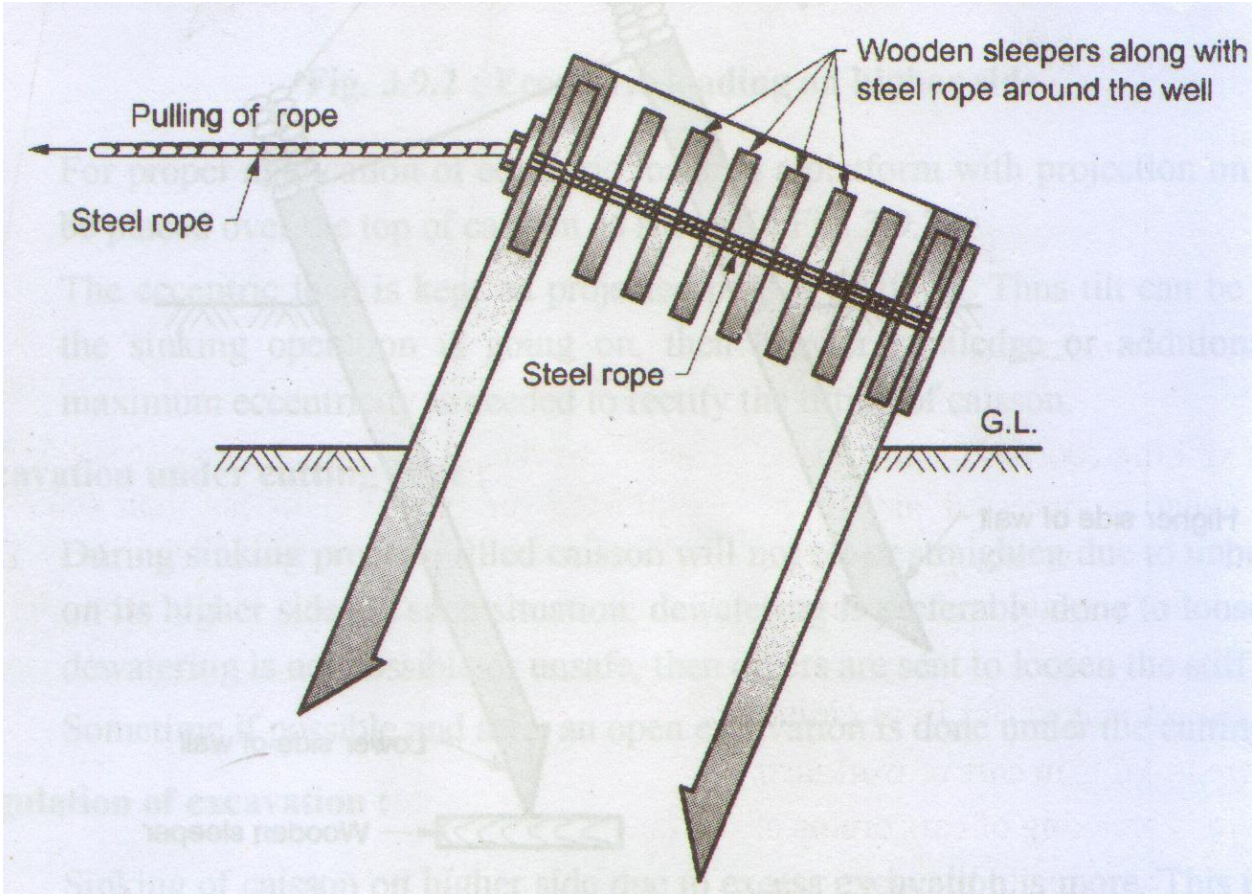


Fig. 3.9.5 : Pushing of caisson by jack

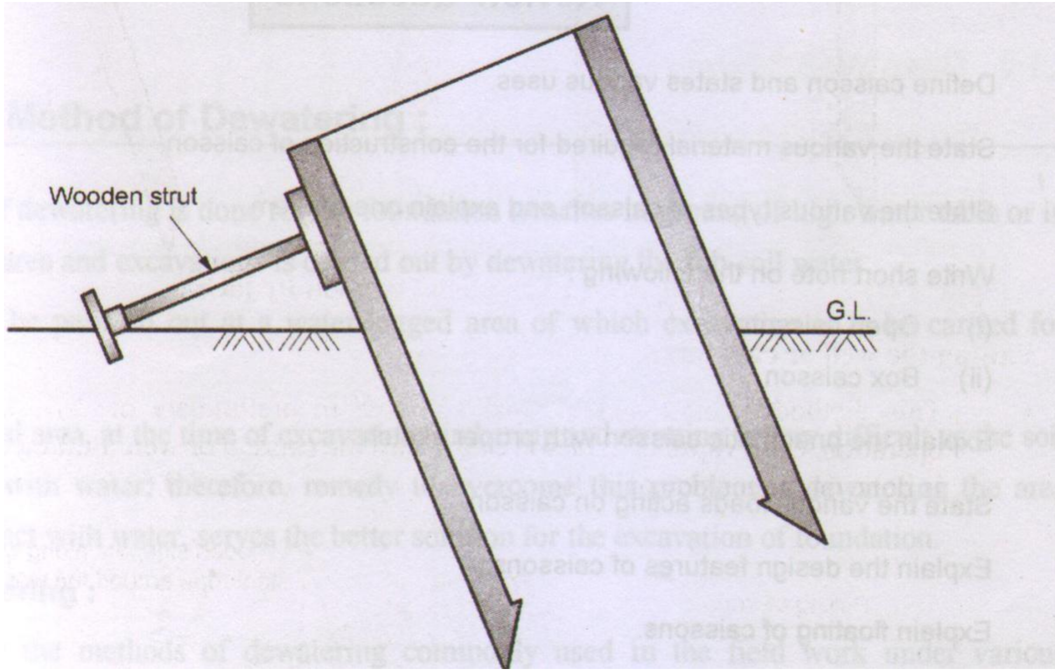
7) Pulling the Well

- This method is most suitable and effective in preliminary or early stages of sinking operation.
- Steel ropes or cables are used pull the well.
- Pulling of well is done on higher side of well.



7) Strutting the Well

- Method of strutting the well is used to prevent any further and possible rise in tilting of the well.
- The well is supported on the tilting side by giving inclined support of a strong wooden member.
- This inclined wooden member is called as a strut.
- The well steining is provided so as to distribute the uniform pressure or load from strut.

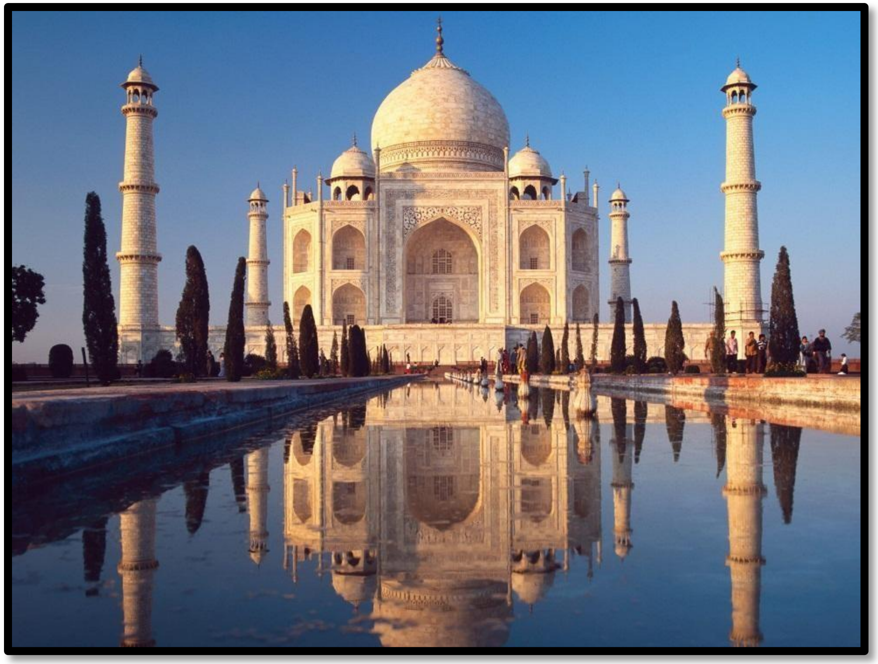


Introduction

- The construction principles of well foundation are similar to the conventional wells sunk for underground water.
- But relatively rigid and engineering behaviour.

Introduction

- **Well foundations have been used in India for centuries.**
- **The famous Taj Mahal at Agra stands on well foundation.**



Objectives

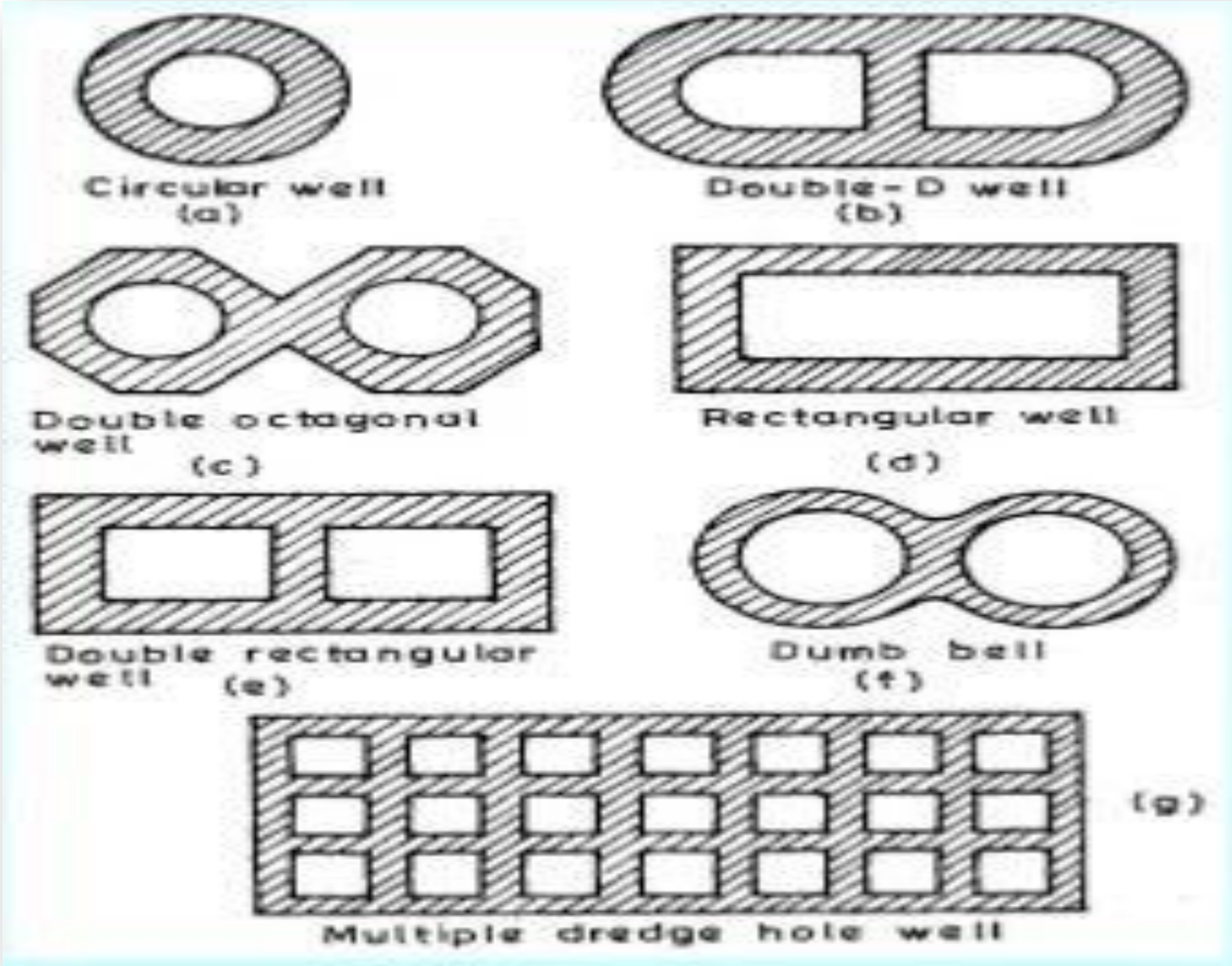
- To know the construction of well foundation.
- To know the different types and shapes of well foundations.
- To know which type of well foundation is suitable for different types of soil strata.

Shapes of well foundations

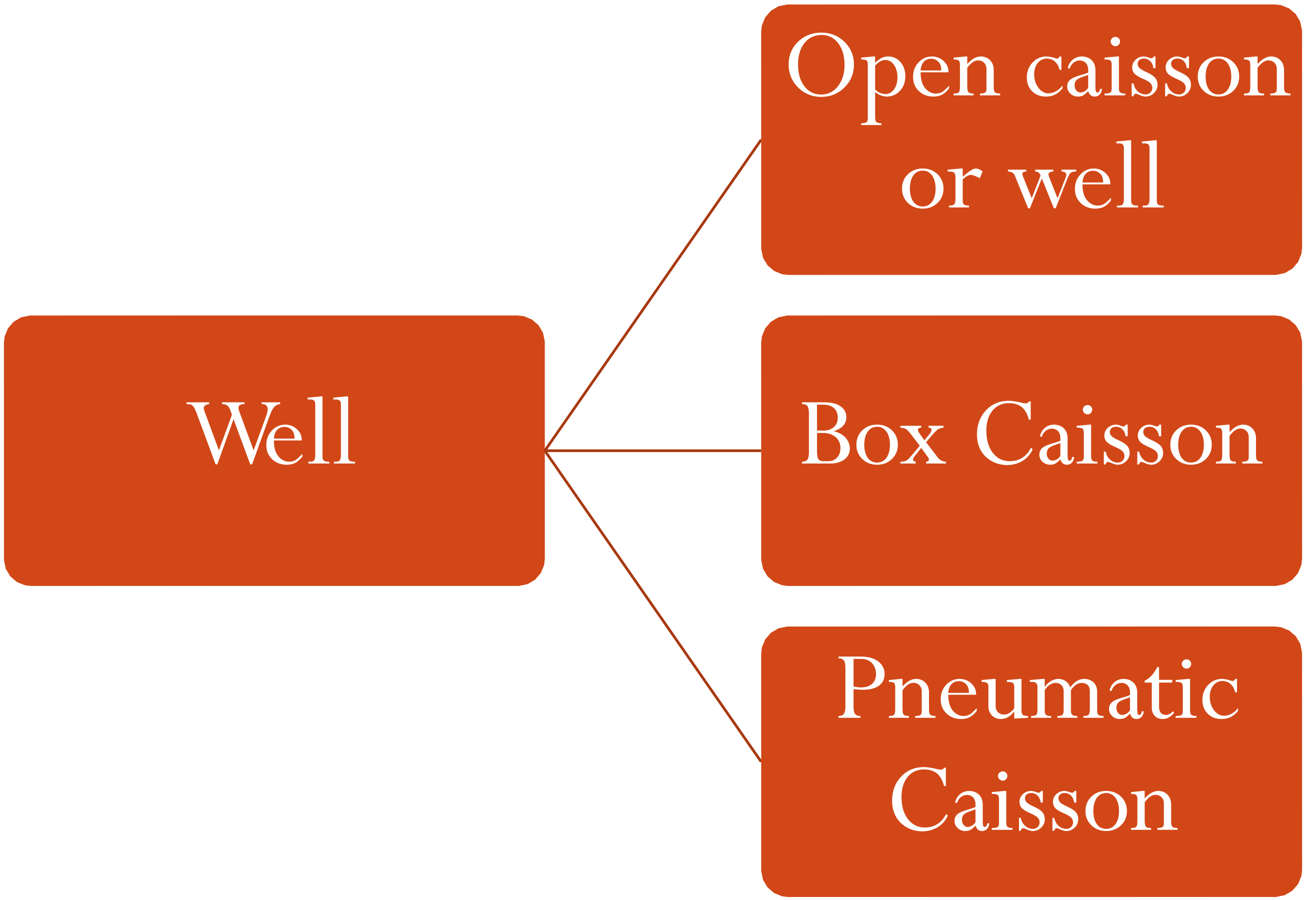
- **Wells have different shapes and accordingly they are named as:-**

1. **Circular well,**
2. **Double D well, Twin**
3. **circular well, Double**
4. **octagonal well,**
5. **Rectangular well.**

Shapes of well foundation



types of well foundation



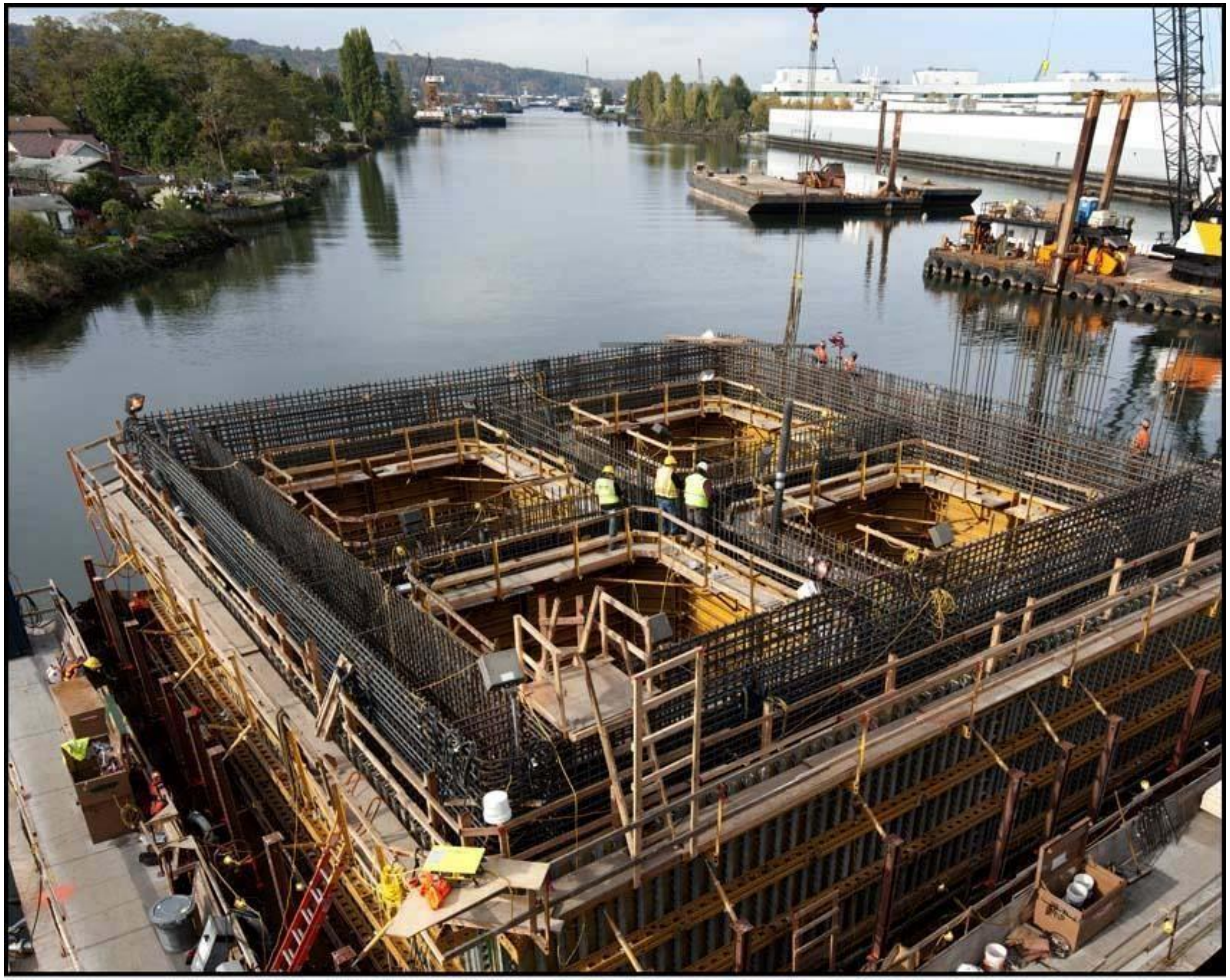
Types of well foundation:-

- **Open caisson or well**: The top and bottom of the caisson is open during construction. It may have any shape in plan.

- **Box caisson**: It is open at the top but closed at the bottom.

- **Pneumatic caisson**: It has a working chamber at the bottom of the caisson which is kept dry by forcing out water under pressure, thus permitting excavation under dry conditions.

Open caisson



2011-11.2_0310+311 First walls in Pier 3 caisson are ready to be poured. photo: John Stamets

Box caisson

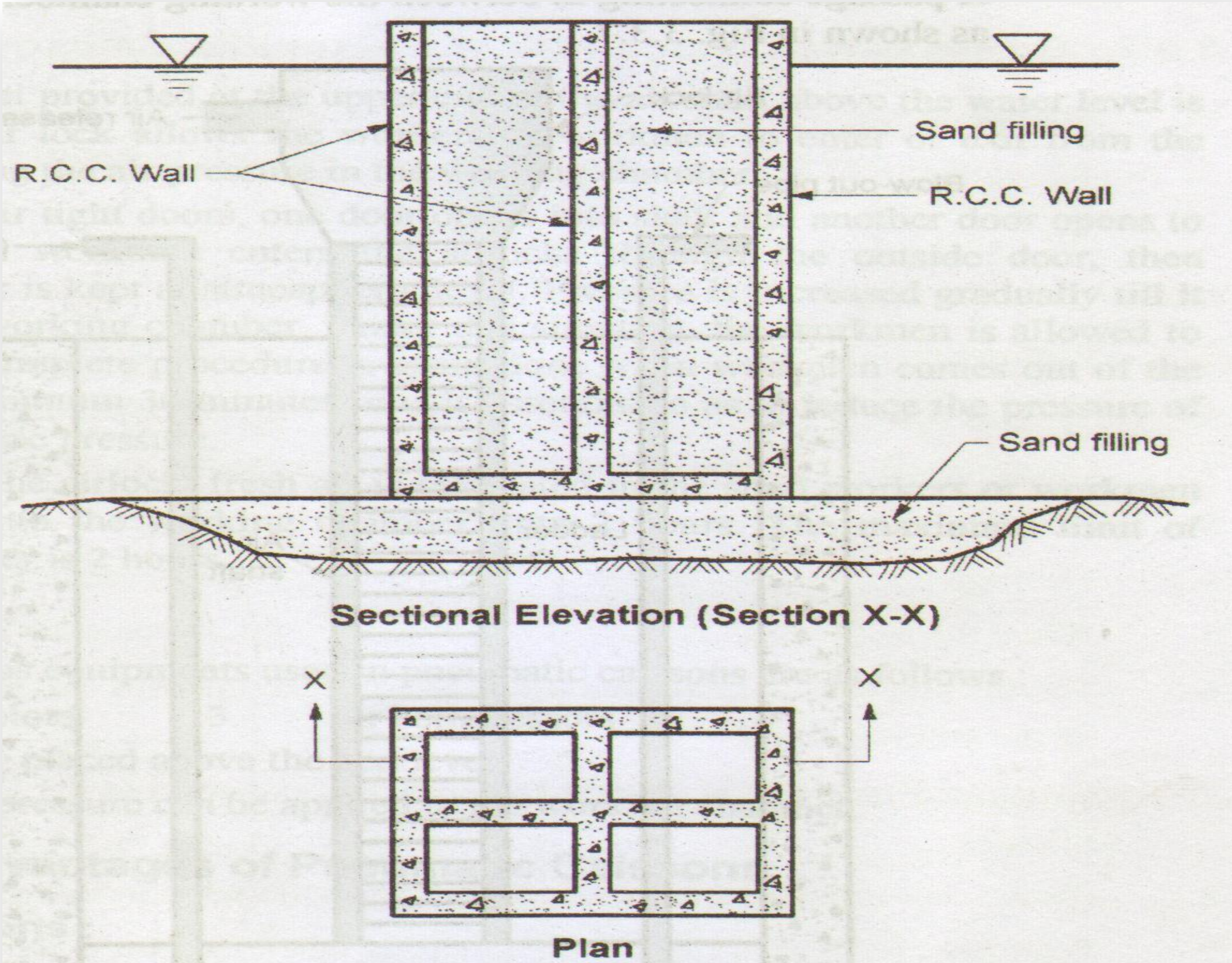
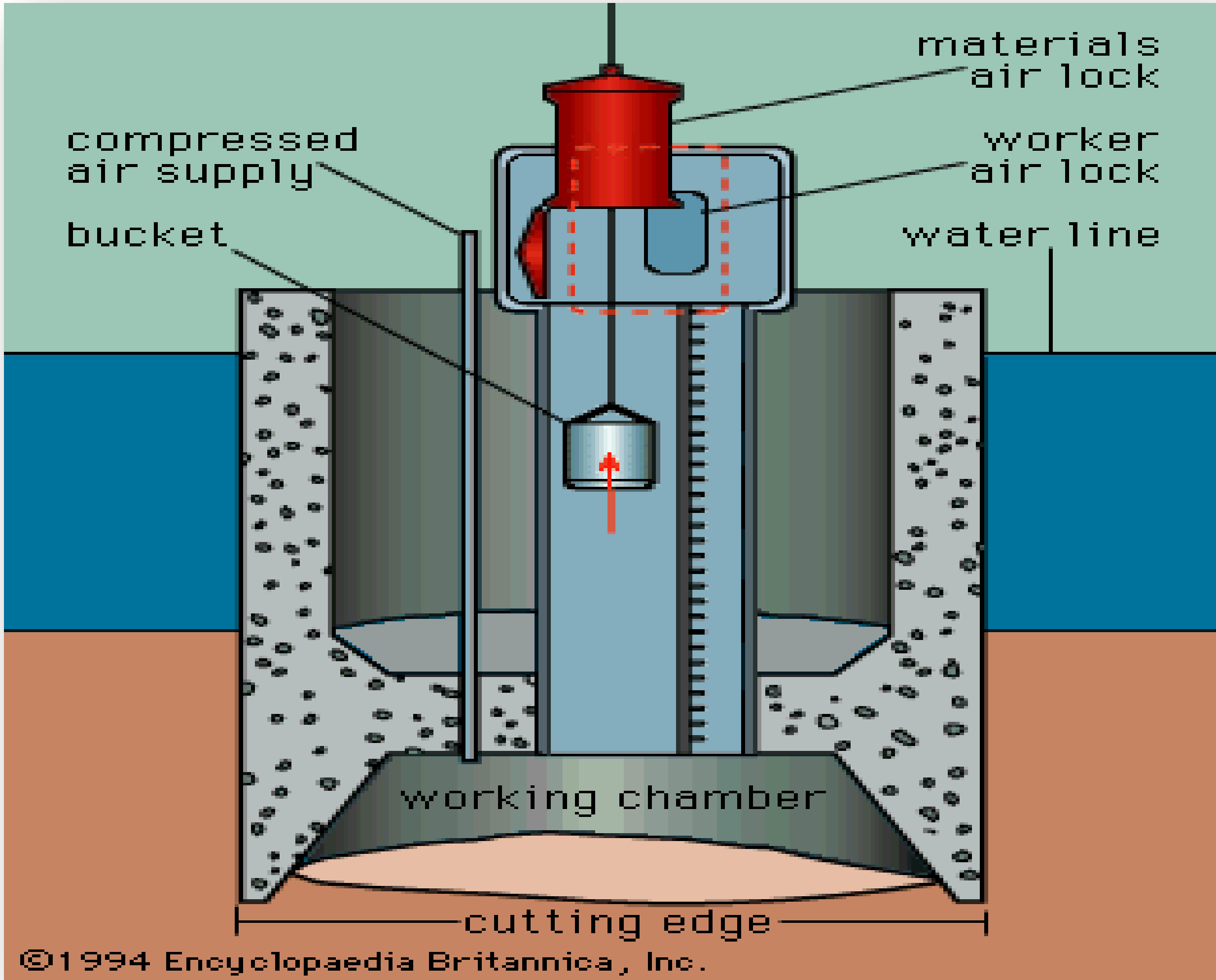


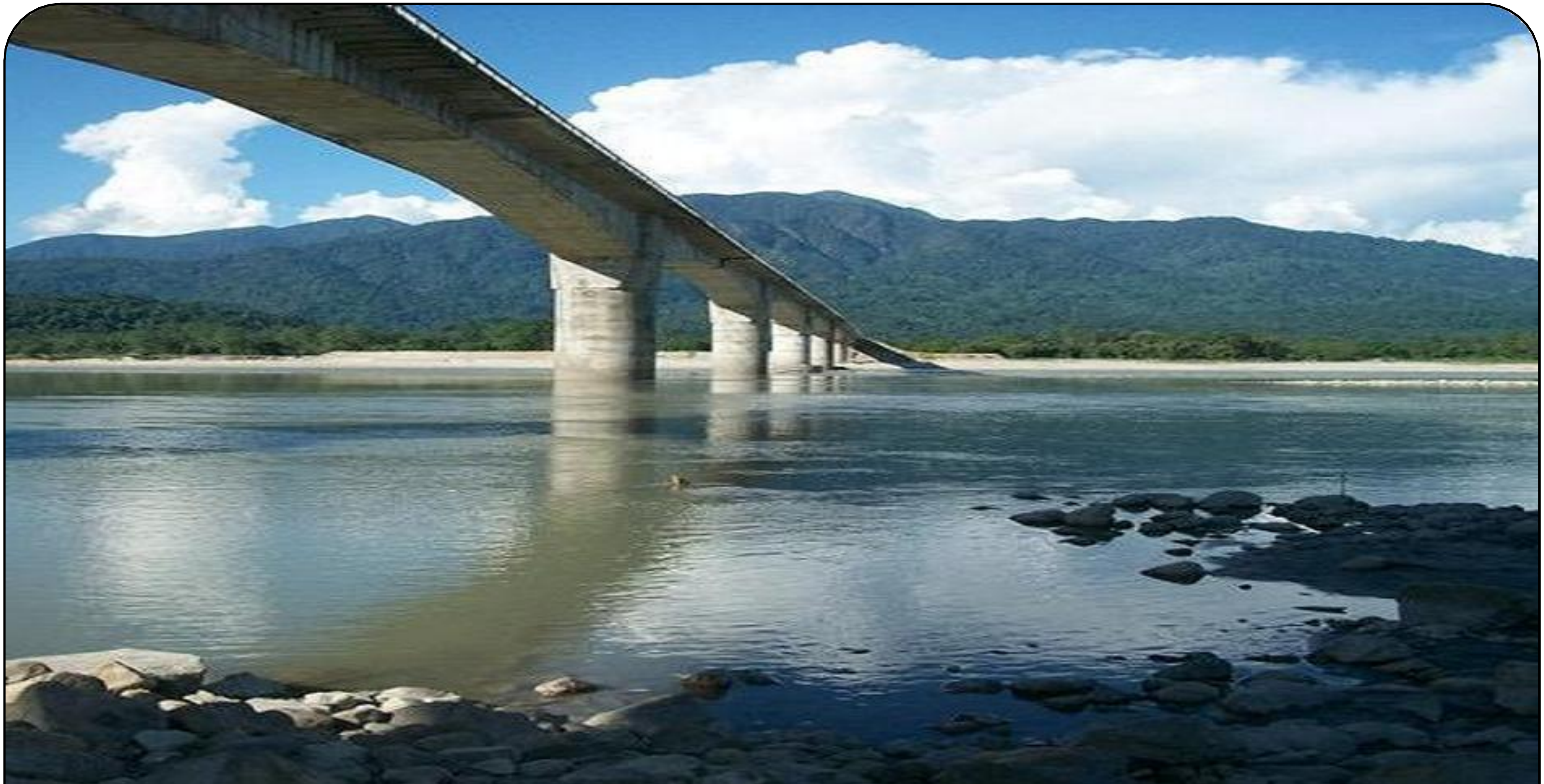
Fig. 3.3.6 : Box caisson

Box caisson

Pneumatic caissons



Well foundation construction in bouldery bed strata



Pasighat Bridge

Andhra Pradesh

IMPORTANT FEATURES OF THE BRIDGE

The Important Factors of the bridge were as follows..

- **Length of the bridge - 704 mts.**
- **Foundation:-**

i.	**Type**	**-Circular Well.**
ii.	**Outer Diameter**	**-11.7 mtrs.**
iii.	**Inner Dia.**	**-6.64 mtrs**
iv.	**Steining thickness**	**-2.53 mtrs.**
v.	**Well curb height**	**-4.5 mtrs.**
vi.	**Angle of cutting edge**	**-33 degrees.**
vii.	**Grade for steining concrte**	**-M25**

Pasighat Bridge

Boulder dredged during well sinking

Construction methodology

- **The construction in flowing water was carried out by means of heavy machinery like dozers , dump trucks and excavators , etc.**
- **Pneumatic sinking and conventional sinking method is adopted for construction purpose.**
- **Sand was taken from the river and cleaned before transportation and then is used as sand filling.**

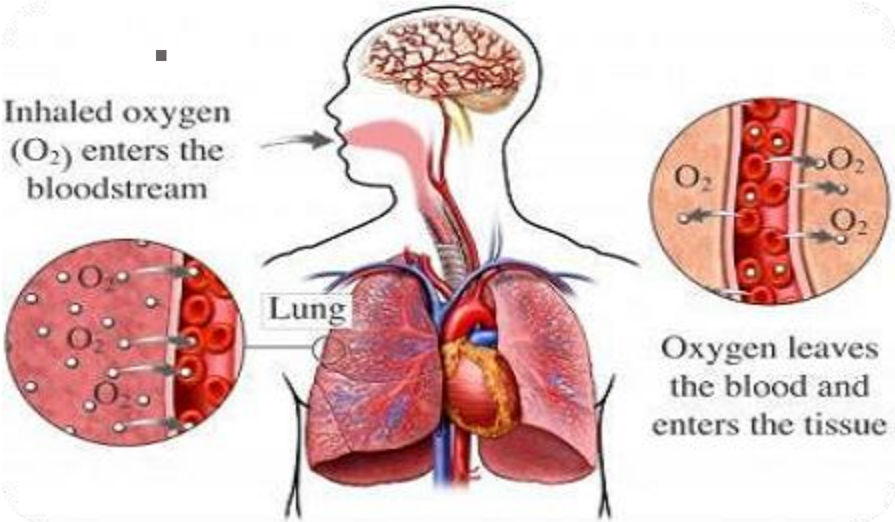
Construction problems

1. The sinking of the well becames very difficult due to the presence of large size of boulders in the strata.
2. Basically there were difficulties in finally deciding the foundation level on such strata.
3. Due to heavy rainfall in the area a considrably reduced working period was available in the region.

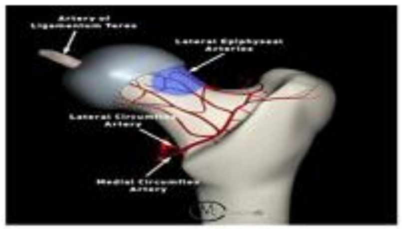
Caisson disease

- In case of sinking process of pneumatic caisson , workers under pressure may suffer from “Decompression sickness”(Caisson disease).due to rapid change in pressure.

- Construction of the Brooklyn Bridge, which was built with the help of caissons, resulted in numerous workers being either killed or permanently injured by caisson disease during its construction, including the designer's son and Chief Engineer of the project.



Caisson Disease



DEFINITION

Caisson is a French word which means‘a large chest or‘a box’.

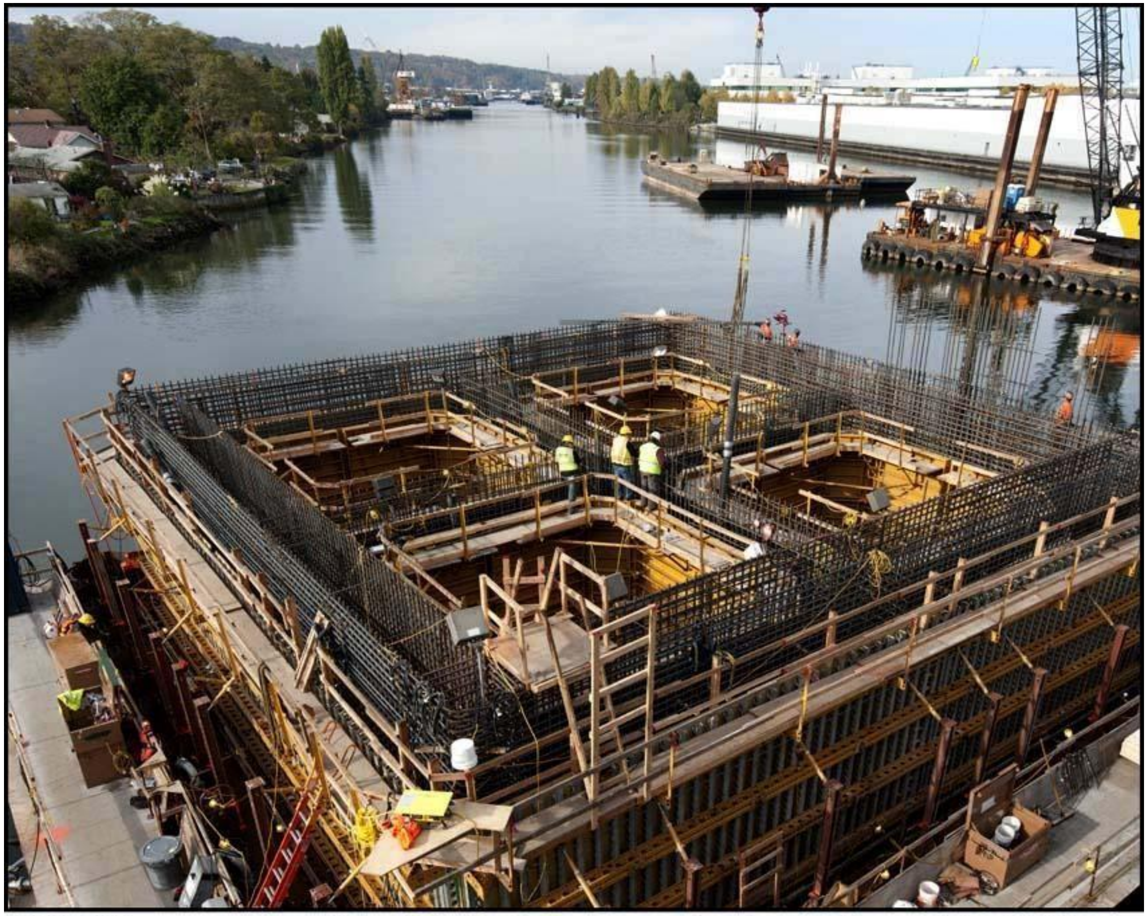
Caisson is a water tight structure made of wood, steel, R.C.C i.e. reinforced cement constructed in connection with excavation for the foundation of bridges, piers in rivers, dock structures etc.

TYPES OF CAISSON

There are three types of caisson as follows:

- Open Caisson.
- Box Caisson.
- Pneumatic Caisson.

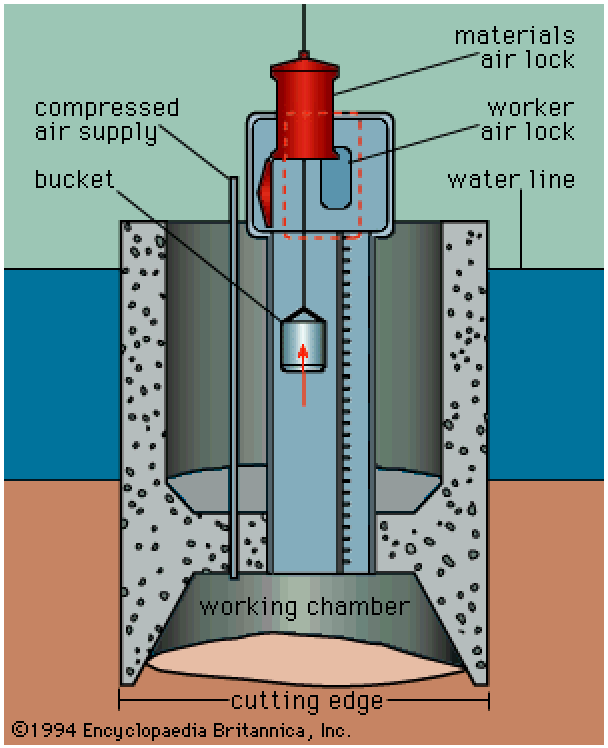
Open Caisson



2011–11.2_0310+311 First walls in Pier 3 caisson are ready to be poured. photo: John Stamets

Box caisson

Pneumatic Caisson



SHAPES OF CAISSON

Basic shapes

- Circular Rectangular
- Square
- Octagonal

Combination of basic shapes

- Double Circular
- Double Rectangular
- Double hexagonal
- Double-D
- Double Octagonal

USES OF CAISSON

- Caissons are more suitable for the deep foundation under water where the foundation should be extended up to or below the river bed so as to obtain the proper stability.
- Caissons as type of well foundation is constructed in rivers and lake, bridges, break water dock structures for the point of view of shore protection.
- When depth of water in river, lake, or sea etc. are more, then caisson structure is used.
- It is also used for pump house which are subjected to huge vertical as well as horizontal forces.
- It is also occasionally used for large and multi-storey building and other structures.

ADVANTAGES

- The caisson can be extended up to large depths.
- Caissons are more suitable for the deep foundation under water where the foundation should be extended up to or below the river bed so as to obtain the proper stability.
- Cost of Construction is relatively less on bed level or lower side.
- Quality control of pneumatic caisson is good because work is done in dry conditions. Concrete gain more strength due to dry conditions.
- In-situ soil tests are possible to determine the bearing capacity of pneumatic caisson.
- There is direct and easy passage to reach the bottom of caisson, hence any obstruction can easily be removed.

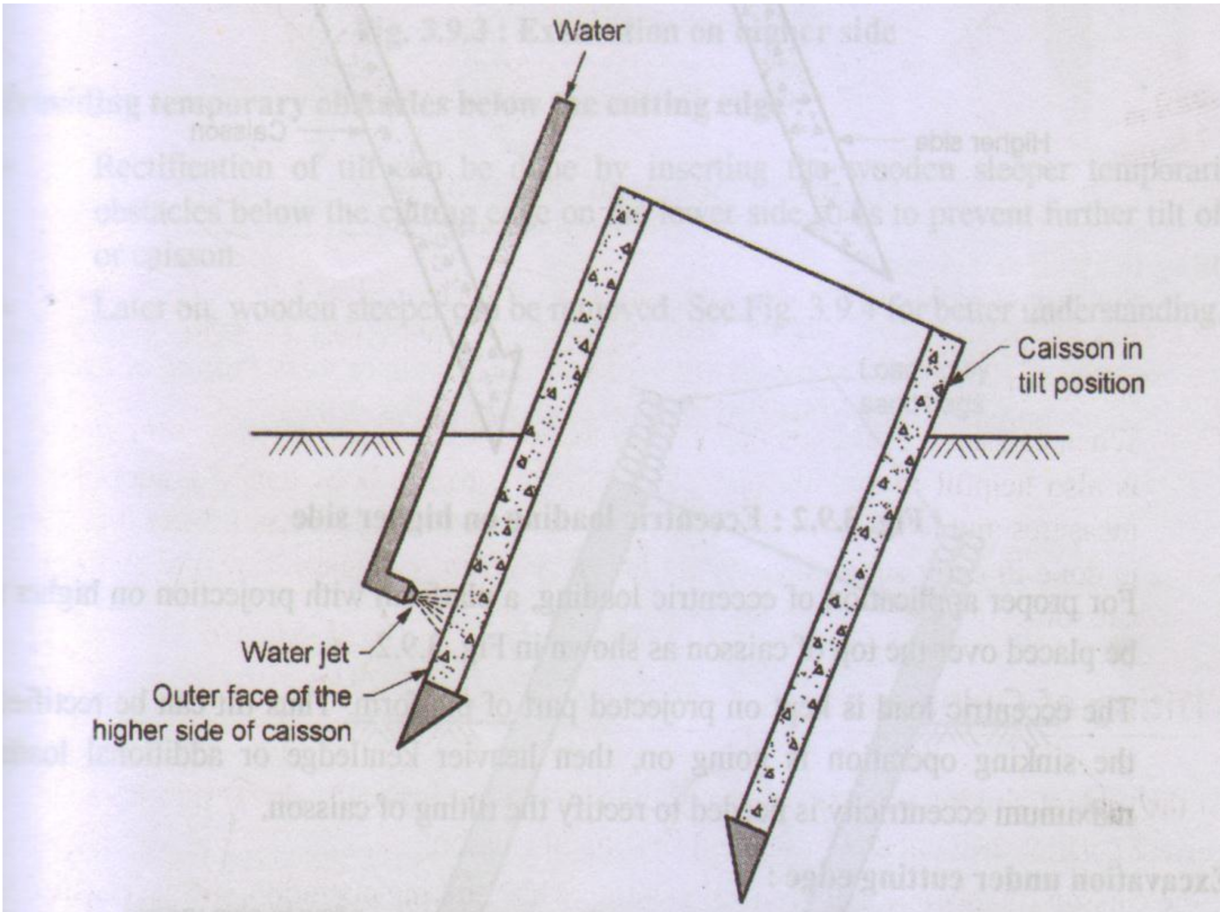
DISADVANTAGES

- In box caisson the placing of concrete is done for concrete seal under water, it may not be satisfactory.
- If any obstruction of boulders or logs are encountered, then progress of work becomes slow.
- The help of divers may be required for excavation near haunches at the cutting edges.
- Construction of pneumatic caissons is much expensive than open caissons.
- During working the various constructional activities, a proper care has to be taken, otherwise it may lead to fatal accidents.
- Labor cost is high.

DIFFICULTIES

Sinking of caisson:

Tilting of caisson:



PREVENTIONS

- Control of tilting.
- Eccentric loading.
- Pushing the caisson.
- Pulling the caisson.
- Strutting the caisson.

CAISSON DISEASES

- Workmen may suffer from giddiness.
- There is pains in ears of workmen.
- There is breaking of ear drums of workmen.
- There is bursting of blood vessels in the nose or ears of workmen.
- It may cause paralytic death.
- If the bubbles are developed in spinal cord, it causes paralysis and if the bubble are developed in heart, it causes heart attack.
- Caisson diseases can be controlled by recompression followed by slow decompression.

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- Image courtesy, http://www.google.com


Thank you!