Civinnovate

Discover, Learn, and Innovate in Civil Engineering

PART-I: Concrete Technology.

Chapter 1

Date
Page

Introduction to concrete and concrete materials [6]

1.1 Use of Concrete in structure and types of concrete.

→ concrete is defined as a "composite material consisting of binding medium and aggregate particles". This binding medium is the product of reaction between hydraulic cement and water so the main constituent of concrete are cement, sand, aggregate and water.

→ Concrete is worlds most used construction material because of:-

a) It's Simplicity
b) Durability
c) High strength
d) Excellent resistant to water
e) versality in shape and forms.
f) cheapest and most readily available material on the job.
g) consideration of energy and resource conservation.

(~~Record~~ - maximum use of concrete in the world in china-Dam is 16 million cubic meter is used for 17 years)

Types of Concrete:

① plain cement concrete (P.C.C)
→ The concrete in which no reinforcement provided called P.C.C
→ It is strong in compression but weak in tension.

② Reinforcement cement concrete (R.C.C)
→ It is the concrete in which reinforcement is embeded for taking tensile, excessive compression and shear stress.

Ⓒ pre-stressed cement concrete
→ It is the concrete in which the stresses are artificially induced before its actual use.
→ This type of concrete can take high tensile and compressive stress.

Special types of concrete:
i) Light weight concrete
ii) Fibre reinforced concrete
iii) concrete containing polymer.
iv) Non-cracking concrete.
v) Chemical resistance concrete.
vi) Temperature resistance concrete.
vii) High strength concrete.
viii) Heavy weight concrete.
ix) Fire resistance concrete.
→ These special types of concrete are made by:-
a) new construction technique.
b) use of various types of aggregate.
c) use of admixture
d) use of polymer etc.

1.2 Concrete materials – Role of different materials (Aggregates, cement, water and admixtures)

1.2.1 Aggregates – properties of aggregates and their gradation:-
→ Aggregate is an inert, inexpensive material dispersed throughout the cement paste so as to produce a large volume of concrete.
Normally, 3 quarters of volume of concrete is occupied by aggregate.

Normally,
Aggregate → 70-75%
water → 15-20%
cement → 10-15%

Role of aggregate:
→ It provides a mass of particles, which resist the action of applied loads, abrasion, percolation of moisture and the action of weather.
→ It reduces the volume changes resulting from setting and hardening process and from moisture changes in the cement paste.
→ It provides a relatively cheap filler for the cementing material.

Types of aggregate.
Ⓐ According to source
ⓐ Natural aggregate
i) Igneous rock
ii) Sedimentary rock
iii) Metamorphic rock
ⓑ Artificial aggregate.
→ For some special purpose, due to the influence of manufacturing method, artificial aggregate are made.

Ⓑ According to size
ⓐ coarse aggregate (4.75mm >)
→ Aggregate retained is 4.75mm IS sieve is called coarse aggregate.
ⓑ fine aggregate (4.75mm <)
→ Aggregate passed through 4.75mm IS sieve is known as fine aggregate.

Ⓒ According to density.

ⓐ Light weight aggregate ($\approx 1120\ kg/m^3$)

ⓑ Medium wt. aggregate ($\approx 1520\ kg/m^3$)

ⓒ Heavy wt. aggregate ($\approx$ more or 2080 kg/m^3)

Properties of aggregate:

Ⓐ Physical properties.

① Shape

ⓐ Rounded

ⓑ Irr-regular

ⓒ Flaky particles

→ A particles is said to be flaky if its least dimension is less than 0.6 times the mean sieve size.

mean size → If particle pass through 40mm seive and retain in 20mm sieve then mean sieve size is $\frac{40+20}{2} = 30mm$

ⓓ Elongated particles

→ If the largest dimension of particle is 1.8 times the mean sieve size, then the particles is called elongated.

ⓔ Angular particle

→ If the dimension of particle possesing well defined edge is called angular particle.

ⓕ Flaky and elongated particle.

② Texture

→ Smooth, rough, granular, crystalline and honeycombed

↳ visible pores and cavity.

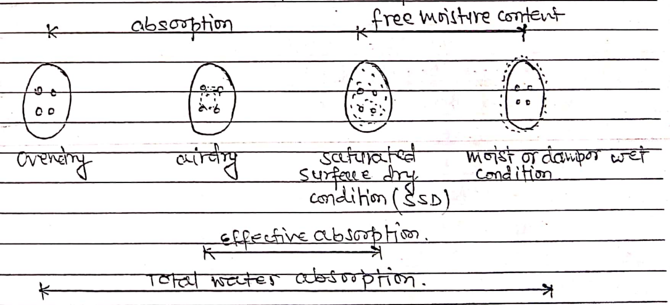
③ specific gravity = $\frac{\text{wt. of material}}{\text{same vol. of wt. of water}}$

④ Apparent specific gravity.

→ It is the ratio of mass of aggregate dried in oven to the mass of water occuping the volume equal to that of the solid including intermediate pores.

⑤ Bulk density.

→ Actual mass of aggregate that would fill a contain a unit volume. The volume is considered as the occupied by both aggregates and voids. So Bulk density depends on the how densly the aggregates are compacted of the size distribution and shape of the particles.

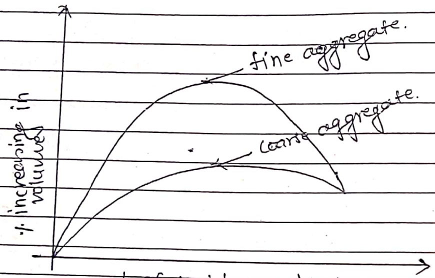


⑥ porosity.

⑦ Bulking of sand

→ Increasing volume of a given mass of sand caused by the films of water pushing the sand particle apart. If the mixed is design by volume of batching (set), bulking of sand effect the mixed proportion. Bulking of sand depend upon the amount of moisture presents and its finess.

eg: mixed proportion : 1:2:4
means: 1kg : 2kg : 4kg
→ 1vol. : 2vol. : 4vol.



(B) Mechanical properties

① Bond and Bond strength:-

→ Bond is the interlocking capacity of a aggregate and adhesion between aggregates and cement paste.

→ Bond strength is the resistance develope to split the aggregate particle from hardned cement paste.

② Hardness (abrasion strength):-

→ Resistance of aggregate to wear is called hardness. It can be determined by loss Angeles abrasion test and express as a percentage loss of weight.

③ Toughness

→ It is defined as the resistance of aggregate to failure by impact or ability of aggregate to with stand repeated blows.

④ Crushing strength

→ It is the strength under the compressive loads. The compressive strength of concrete can not exceed that of the aggregate used there in.

(C) Chemical properties

① Alkali aggregates reaction:

→ Aggregate contain silicius material, alkaline hydroxide derived from alkalies in the cement. As a result alkali silicate gel is formed around the aggregate particles. This gel ~~sils by~~ swells by absorbing water and internal pressure is created as a result expansion and cracking occures.

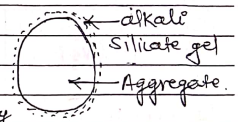
→ The reaction of aggregate is affected by –

i) particle size and porosity

ii) Quality of cement and quantity of alkali present in cement.

iii) permeability of paste.

iv) Availability of water in paste.



Gradation of Aggregates:-

Sieve analysis:-

→ The process of dividing a Sample of aggregate into fraction of same particle size is known as sieve analysis. and its purpose is to determine the grading or size distribution of the aggregate.

→ By the use of well graded aggregates reduction of air voids in the concrete decrease strength and durability increases.

→ Grading of aggregate also affect the workability.

Fineness modulus of aggregate:-(FM)

→ It is an index number used to indicate the average size of particles in the aggregate. Also defined as the sum of the cumulative percentages retained on the

Sieves of the standard series, divided by 100. The standard series consists of sieves, each twice the size of the preceding one viz: 150, 300, 600 μm, 1.18, 2.36, 5.00 mm (ASTM No. 100, 50, 30, 16, 8, 4) and up to the largest sieve size present.

→ How to obtain fineness modulus (FM):-

Soln:- Let.

Sieve I.S	wt. retained (kg)	Cumulative wt. retained	cumulative %. wt. retained	% passing
80 mm	0	0	0	100
40 mm	0	0	0	100
20 mm	3.5	3.5	35	65
10 mm	3.0	6.5	65	35
4.7 mm	2.8	9.3	93	7
2.36 mm	0.7	10.0	100	0
1.18 mm	0	10.0	100	0
600 μm	0	10.0	100	0
300 μm	0	10.0	100	0
150 μm	0	10.0	100	0
Total cumulative %. wt. retained			= Σ 693	

Hence,

Fineness modulus (FM) = $\frac{693}{100}$ = 6.93 means mean size of sample.

[Note.

Total mass retained = 10 kg.

Now find cumulative %. wt. retained for 20mm IS sieve. = $\frac{3.5 \times 100}{10}$ %

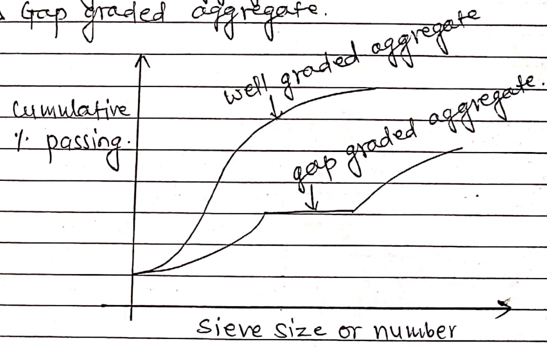
= 35 %.]

Grading curve (G.C)

→ A curve is plotted cumulative %. passing in ordinates Vs sieve size or number are plotted to lograthmic scale in abscissa is called grading curve.

i) well graded aggregate

ii) Gap graded aggregate.



1.2.2 Cement:

→ Cement is a binding material with adhesive and cohesive properties. Portland cement is a cement obtained by mixing together calcareous and argillaceous or silica, alumina and iron oxide bearing materials burning them at a clinkering temperature and grinding the resulting clinker.

→ Portland cement is the no material other then gypsum, water and grinding aids may be added after burning.

Date ___
Page ___

Manufacture of portland cement:

→ Just line diagram for remember.

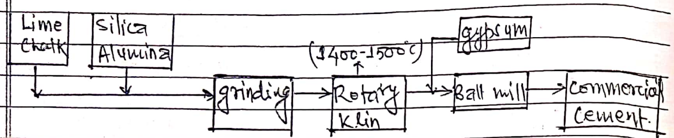


fig: Algorith of manufacture of portland cement.

→ It can be seen that it is made primerily from combination of calcareous material such as lime stone or chalk and silica or alumina found as clay or shale. The process of manufacture consists essentially of grinding the raw material into a very fine powder mixing them in predetermined the proportion and burning in a large Rotary Klin at a temperature of about 1400°C. When the material partially fuses into Ball is known as Klinker. The clinker is cooled and grinding to a fine powder with some gypsum added and resulting product is the commercial portland cement.

→ The gypsum is added to retard down the initial setting action of cement.

Chemical reaction in the Rotary Klin:

lime → $CaO + CO_2$

clay → $SiO_2 + Al_2O_3 + Fe_2O_3 + H_2O$

1450°C →

→ $3CaO \cdot SiO_2$ (tricalcium silicate)

→ $2CaO \cdot SiO_2$ (dicalcium silicate)

→ $3CaO \cdot Al_2O_3$ (trycalcium aluminate)

→ $4CaO \cdot Al_2O_3 \cdot Fe_2O_3$ (tetra calcium alumino ferite)

Date ___
Page ___

Compound composition of portland cement:

→ The raw materials used for the manufacture of consist mainly of lime, silica, alumina and Iron oxide. To oxide interact with one another in the klin at high temperature form more complex compounds.

Compound	composition in %
CaO	60-67
SiO_2	17-25
Al_2O_3	3-8
Fe_2O_3	0.5-6
MgO	0.1-4
Alkalies	0.4-1.3

Lime saturation factor = $\dfrac{CaO - 0.7SO_3}{2.8SiO_2 + 1.2Al_2O_3 + 0.65Fe_2O_3}$

→ For 33 grade, it should be between 0.66.

→ The formed complex compounds are the major consitituent of cements. These are also called "Bogue's compounds".

Formula	Symbol.
$3CaO \cdot SiO_2$	C_3S
$2CaO \cdot SiO_2$	C_2S
$3CaO \cdot Al_2O_3$	C_3A
$4CaO \cdot Al_2O_3 \cdot Fe_2O_3$	C_4AF
CaO	C
SiO_2	S
Al_2O_3	A
Fe_2O_3	F
H_2O	H
SO_3	$\bar{S}$

Bogue's equation:

$C_3S = 4.07C - 7.6S - 6.72A - 1.43F - 2.85\bar{S}$

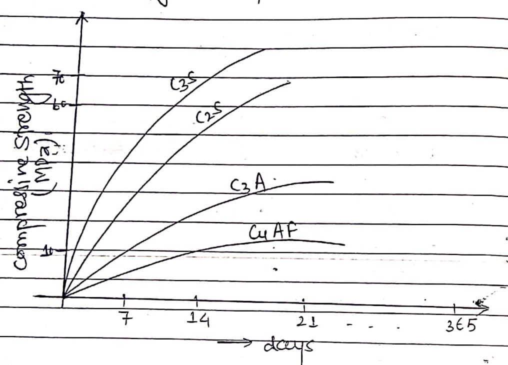
$C_2S = 2.87S - 0.754\,C_3S$

$C_3A = 2.65A - 1.69F$

$C_4AF = 3.04F$

→ In these equation impurities are not considered.

Structure and reactivity of compounds:



C_3S and C_2S:

→ C_3S and C_2S are two compounds responsible for strength for hydrated cement paste.

→ C_3S gives early high strength and C_2S later gain strength.

→ impure → C_3S – Alite

→ impure → C_2S – Belite

→ $\alpha\,C_2S \xrightarrow{1400°c} \beta\,C_2S \xrightarrow{670°c} \gamma\,C_2S \longrightarrow C_3S \longrightarrow$ unstable

Calcium Aluminate (C_3A) and Ferroaluminate (C_4AF):

→ C_3A contributes little or nothing to the strength of cement.

→ C_4AF present in small quantities, it doesn't affect the behaviour of significantly as compare with others compound but its presence may accelerate the hydration of silicates

→ These are cubic crystal structure.

Hydration of Cement:

→ Anhydrous cement doesn't bind fine and coarse aggregate it acquires adhesive property only when mixed with water. The chemical reaction between cement and water is called ~~[illegible]~~ hydration of cement.

For C_3S:

$2C_3S + 6H \longrightarrow C_3S_2H_3 + 3Ca(OH)_2 + 502\ (J/gm)$

[100] [24] [75] [49] (120 cal/gm)

For C_2S:

$2C_2S + 4H \longrightarrow C_3S_2H_3 + Ca(OH)_2 + 260\ (J/gm)$

[100] [21] [99] [22] (62 cal/gm).

→ Approximately same amount of water for hydration is required but C_3S produce more $Ca(OH)_2$ due to lesser amount of $Ca(OH)_2$. cement containing higher C_2S is more suitable in acidic environment.

→ Adhesive property of the cement paste is due to formation of calcium silicate hydrate ($C_3S_2H_3$).

For C_3A:

$C_3A + 6H \longrightarrow C_3AH_6 + 867\ (J/gm)$

(207 cal/gm)

For C_4AF:

$C_4AF + 2CH + 10H \rightarrow C_3AH_6 + C_3FH_6 + 416$ (J/gm)
(100 cal/gm)

→ The reaction of C_3A with water is more rapid and would lead to the flash set so gypsum is added for prevention.

Heat of Hydration:

→ Quantity of heat develope upon the complete hydration at the given temperature is called heat of hydration.

$C_3A, C_3S, C_4AF, C_2S \rightarrow$ decrease in heat.

Properties of cement.

① Chemical composition.
→ same above & reaction of hydration of cement.

② Fineness.
→ Finner the cement, more surface area of aggregate cover and gain strength in weak in short time.

③ Soundness.
→ once, the cement sets doesn't undergo a large change in volume. The large change in volume is due to free lime, magnesium and calcium sulphate, so presence of these material is called unsound.

④ Setting time.
→ i) Initial setting time → plastic cement change into solid mass
→ ii) Final setting time. → Hydration complete and develop strength.

⑤ Heat of Hydration.

⑥ Strength.

1.2.3 Introduction to special types of cement:

① Ordinary portland cement (OPC).
→ Most common cement in use
→ Medium rate of strength development and heat evolution.
→ It is suitable in general concrete construction, when there is no exposure sulphate in the soil or in the water.

② Rapid hardening portland cement
→ High early strength cement.
→ The strength of cement develop rapidly due to higher contain of C_3S.
→ Use in emergency work.

③ Low heat portland cement.
→ This cement has the low heat of hydration due to the lower contain of C_3S and C_3A.
→ slower development of strength but ultimate strength unaffected.

④ Sulphate Resistance cement.

Sulphate attack:
→ When C_3A reacts with gypsum calcium sulphoaluminate is formed which is expansive in nature i.e volume of product is more than C_3A and gypsum and cause crack and disintegration of concrete. This expansion is called sulphate attack.
→ This cement has low C_3A contain to avoid sulphate attack.

⑤ portland pozzolana cement.
→ It contains the Silica and alumina materials.
→ strength is high and heat of hydration less than ordinary portland cement.

⑥ portland blast-furnace slag cement.
→ corrosion resisting cement.
⑦ white cement (china clay)
⑧ coloured cement.
→ color pigment in white cement.
⑨ Hydrophobic cement.

1.2.4 Use of water in concrete:
→ water is the most important consitituent in concrete. It plays voilet role on the strength of concrete. water in concrete construction is used for following purposes.
ⓐ water for mixing concrete.
ⓑ water for washing aggregate.
ⓒ water for concrete curing.

(a,b) water for mixing concrete and washing aggregate.

IS456 Matter	max(%)	Remarks
i) organic	0.02	
ii) Inorganic	3.00	
iii) Sulphate	0.04	
iv) Chloride	2.00	
v) Suspended matters.	→ 1.00 → 2.00	For plain Reinforced work For RCC

→ Drinking water may not be suitable as mixing concrete. If water as a high concentration of sodium or potasium because their is danger alkali

aggregate reaction.
→ The presence of algea in mixing water, results in loss of strength.
→ The pH of water between 6 to 8 is suitable for used.
→ sea water increase the risk of corrosion in the Reinforcement cement (RCC).
→ water having sugar retards the setting time of concrete.

(c) water for concrete curing.
→ If the water consist of various harmful elements of a compounds they can easily be taken to the pores and capillaries of the concrete and deterioration process of concrete may be accelerated. In order to avoid such ~~si~~ situation a good quality of water shall be used. water suitable for concrete mixing is quite suitable for curing of concrete.

1.2.5 Admixtures
→ A suitable material added to concrete just before or during the mixing to modified one or more properties of concrete is known as admixture.

Classification of admixtures:
① Chemical Admixture.
→ Chemical admixture are materials in the form of powder or fluids that are added to the concrete to give it certain characteristics not obtainable with plain concrete mixes. In normal use admixture doses are less than 5% by mass and added to the

concrete at the time of mixing.

(a) Accelerator admixture

→ Speed of the hydration of the concrete eg. NaCl, $CaCl_2$ etc. However use a chlorides may causes corrosion in steel Reinforcement and is prohibited in some countries.

(b) Retarders admixture

→ slow down the hydration of concrete eg. Sucrose, glucose. sugar. critic acid etc.

(c) plasticizers and superplasticizers.

→ These are used to increase workability of fresh concrete. plasticizers can be used to reduce the water contained of concrete while maintaining workability and it sometimes called water reducer. Such treatment improves the strength and durability characteristics eg. sulfonatedmelamineformaldehyde, Acetoneformaldehyde condensate etc. superplasticizer is high range water reducer, it use to permit the reduction of water to the extent upto 30% without reducing workability. (15% For plasticizers). It produces flowing, self levelling, self compacting and high strength and high performance concrete.

(d) pigment admixture.

→ It is used to change the colour of concrete for asthetic purpose.

(e) Corrosion inhabitors

(f) Bonding Agent

(2) Mineral admixture:

→ These are inorganic materials that also have cementing properties or latent hydraulic properties. These are very fine grain materials are added to the concrete mix to improve the properties of concrete is called mineral admixture. Relatively large amount of mineral admixture is used in concrete compare to chemical admixture.

(a) Natural minerals

(b) By product materials.

(a) Natural minerals.

→ All natural pozzolanas

→ clays and shales.

→ Diatomaceous Earth.

→ Minerals

→ volcanic gases

→ volcanic tuffs and pumicites

(b) By product materials.

→ Ground granulated blast furnace slag.

→ Flyash ↓ commonly used to controls environmental pollution.

→ silica flume

→ surkhi

→ Metakaoline etc.

Water proofing agent:

→ The concrete should be impervious to water under the following two conditions.

① When concrete surface is subjected to water-pressure on end side and.

② The concrete should be impervious to the absorption of surface water by capillary action.

→ Water proofing admixtures may be obtained in powder, paste or liquid and may consist of pore filling or water repellent materials.

The chief materials in pore filling class are:

i) Silicates of soda.

ii) Aluminium and zinc sulphates.

iii) Aluminium and calcium chlorides.

Use of Mineral admixtures in concrete

→ To increase the rate of strength development at early age.

→ To increase the water tightness

→ To retard the initial setting time.

→ To improve resistance to attack by sulphate soils and sea water.

→ To increase workability.

→ To improve the extensibility.

→ To decrease heat evolution.

→ To improve the lower susceptibility to dissolution and leaching.

→ To increase resistance to freezing and thawing etc.

Extra

① What do you understand by good concrete?

→ A concrete mix which posses the high workability, high strength, high modulus of elasticity, high density, low permeability and low resistance to chemical attack and high dimensional stability is called a good concrete.

Probable questions in this chapter:

2068 ① What is the basic ingredients of concrete? Mention different types of admixtures used in concreting works.

② What is the role of superplasticizer in concrete? Explain how superplasticizer works in concrete.

③ Describe different natural characteristics of aggregates and their effects on concrete behaviour.

④ Explain the role of water in concrete.

⑤ Explain the Alkali-Silica reaction.

2069 ⑥ Explain Bougue's compound of cement.

⑦ List out the special type of cement.

⑧ What is bulking of sand?

⑨ Define the fineness modulus of aggregate?

⑩ What is good concrete?

⑪ Explain about the manufacture of portland cement.

⑫ Describe in brief water proofing agent.

⑬ List out the special types of concrete.

⑭ What do you understand by sulphate attack in concrete.

⑮ Described the hydration of cement.

⑯ List out the use of mineral admixtures in concrete.

CHAPTERE→2

"STRUCTURE OF CONCRETE" [6]

2.1 Concrete as three phase system:

→ Type, amount, size, shape and distribution of phases present in the solid stage is called structure of concrete.

Macrostructure → Gross structure is visible to human being.

Microstructure → Microscopically magnified portion of a macrostructure.

Three phases of concrete:

i) Aggregate phase
→ Aggregate of different size.

ii) Binding medium phase
→ cement paste phase.

iii) Transition zone phase.
→ Interaltral regions.
→ Each of three phase in multiphase in nature.

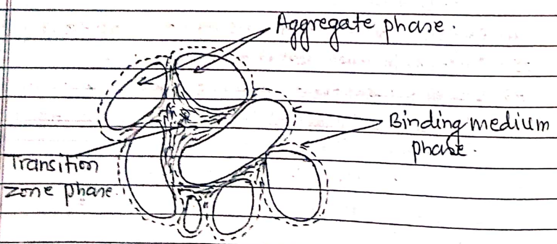


fig: phases of concrete.

2.2 Structure of aggregate phase:

→ Responsible for unit weight, elastic modulus, dimensional stability of concrete because these properties are based on physical characteristics.
→ 60-70% of the volume of solids in most of the concrete.
→ The chemical or minerlogical composition of the solid phase in aggregate is usually less important than the physical charateristics such as the volume, size and distribution of pores.
→ Stronger than other two phases.
→ Size, shape affect the strength of concrete in indirect way.
→ Larger the size of aggregate in concrete and higher the proportion of elongated and flaky particle the greater will be the tendency for water films to accumulate next to the aggregate surface thus weakning cement paste aggregate in transition zone.

2.3 Structure of the hydrated cement paste phase (hcp):

① Solids in hcp:
→ Four principles solid phases generally present in hcp.
i) calciumsilicate hydrate phase (CSH phase)
→ Most important in determining the properties of the paste.
→ It is often known as C-S-H gel or tobermorite gel
→ Exact structures not known.
→ 50-60% of completely hydrated portland cement paste.
→ powers - Brynaures model → Layer structures.
→ with very high surface area (100-700 m²/gm)
→ strength is due to mainly vendervaall force.
→ Size of gel pores is almost 18 A°.

(iv) Water in hcp.

→ Capillary water.
 → water present in a void larger than 2·5nm
 → In voids > 50nm
 → Effect on strength.
 → In voids < 50nm
 → Effect on shrinkage.

→ Adsorbed water.
 → close to solid surface.
 → Influence of attractive forces.
 → physically held by hydrogen bonding upto six molecular layers of water.
 → Bond energy decreases with distances from solid surface.
 → Major portion loss by drying hcp to 30% RH (Relative humidity).
 → Less responsible for shrinkage of hcp on drying.

→ Interlayer water.
 → Associated with C-S-H structure.
 → Monomolecular water layer between C-S-H layers is strongly held by hydrogen bonding.
 → ~~Kont~~ Lost on strong drying.
 → C-S-H structure shrinks under water lost.

→ Chemically combined water.
 → Integral part of structure.
 → Not lost on drying.

③ Strength of hcp.
→ principle source of strength is venderwall force.
→ Degree of adhesion depends upon surface involved.
→ strength is inversly proportional to porosity.
→ Small crystals of C-S-H, calcium ~~sulphate~~ sulphoaluminate hydrates and hexagonal calciumaluminate hydrates posses enormous surface area and adhesion property.
→ voids in hcp – function of amount mixed and degree of hydration.

④ hcp dimentionally not stable.

⑤ Durability of hcp.
→ hcp is alkaline so, exposure to acedic to acidic environment cause deteriomental.
→ permeability ~~respe~~ (water tightness) is the primary factor for durability.
→ porosities represented by the C-S-H interlayer space and small capillaries do not contributes to permeability of hcp.

2·4 Transition zone in concrete:
→ Transition zone in concrete study under the three major principles.

1) Significance of transition zone.
→ concrete is brittle in tension and relatively tough in compression.
→ compressive strength of concrete is greater than tensile strength by an order of magnitude.
→ At given cement content and water cement ratio, age of hydration strength of concrete decreases as the coarse aggregate size increases.
→ The permeability of a concrete containing even a very dense aggregate will be higher by an order of magnitude than the permeability of the corresponding cement paste.

(ii) Calcium hydroxides.
→ It is also called portlandite.
→ 20-25% of solid hcp.
→ Tends to form large crystals with distinctive hexagonal prism
→ Lower surface area so less the vanderwall force than C-S-H phase.
→ Higher percentage has adverse effect on chemical durability to acidic soln due to high solubility than C-S-H phase.

(iii) Calcium sulfoaluminates.
→ Nearly 15-20% solid volume of hcp.
→ At early stage sulphate to aluminate ratio (in ionic form) favours the formation of trisulphate hydrate also called ettringite which forms needle shaped prismatic crystals.
→ In paste of ordinary portland cement ettringite eventually transforms to the monosulphate hydrate which forms hexagonal plate crystals.
Ettringite – $C_6A\bar{S}_3H_{32}$
Monosulphate hydrate – $C_4A\bar{S}H_{18}$.
→ presence of monosulphate hydrate in concrete is vulnerable (weak) to sulphate attack.

(iv) Unhydrated clinker grains:
→ Depends on the particle size of cement and degree of hydration.
→ clinker particles size range 1 to 50 μm.
→ Smaller particles hydrates first and larger become smaller due to hydration.
→ At later stage due to lack of available space formation of very dense hydration product.

② Voids in hcp:
(i) Interlayer space in C-S-H
→ size of it is 18 A°, porosity - 28%.
→ size are too small to have adverse effect on strength and permeability of hcp.
→ water in these voids held by hydrogen bonding, removal of water under certain condition leads drying shrinkage.
void > 50nm → macropores (strength and permeability)
void < 50nm → micropores (dry shrinkage).

(ii) capillary voids.
→ space not filled by solid component of hcp.
→ Total volume of water cement mixture remains unchanged during the hydration process.
→ size and amount of capillary voids is related to water-cement ratio and degree of hydration.

(iii) Air voids.
→ Entrained air
→ size 50 to 200 μm
→ spherical shape.
→ effect strength.
→ Entrapped air.
→ Irregular in shape
→ larger than entrained air.
→ Both are larger than capillary voids so, adversly effect the strength and permeability.

2) Structure of transition zone:

→ Experimentally difficult to obtain structure of transition zone.

→ "Maso giles" → water films around the large aggregate particles.

→ Higher water cement ratio around larger aggregate particles.

→ More porous structure in the vicinity of coares aggregate.

3) Strength of transition zone.

→ Adhesion force is due to venderwall's forces.

→ In early age volume and size of voids in transition zone are larger so, less strength of transition zone.

→ However with increase in age of strength of transition zone may become greater than the initial strength or early strength.

→ Larger the aggregate size, thicker the water film around aggregate and develop crack under tension.

→ Micro cracks in transition zone even before loading.

Probable questions of in this chapter

2069 ① Describe, in brief, concrete as three phase construction material.

② Explain the voids and water in hydrated cement paste.

③ Explain the significance of transition zone and strength of transition zone in concrete.

④ What is transition zone in concrete?

⑤ What are the different phases of concrete? Describe briefly their role in hardened concrete.

CHAPTER-3

'MIX DESIGN OF CONCRETE AND PROPERTY OF GREEN CONCRETE' [12]

3.1 Workability and it's test

→ As per IS code, the workability of the freshly mixed concrete is the property which determines the ease and homogenity with which it can be mixed, placed, compacted and finished.

→ According to road Research Laboratory, UK, the strict definition of workability is the amount of useful internal work neccessary to provide full compaction. This work i.e in compaction energy is required to

* overcome the internal friction between individual particles in the concrete.
* overcome the surface friction between concrete and form work or reinforcement.

→ The factors involving workability are:

(i) Amount of water.
(ii) Type and grading of aggregate.
(iii) Ratio of fine and coarse aggregate.
(iv) Surface texture of aggregate.
(v) water cement ratio.
(vi) Aggregate cement ratio.
(vii) Use of admixtures.

Test of workability:

→ There are different test are available to check/measure workability. These are.

① slump test
② compaction factor test.
③ Flow table test
④ Vee-Bee (consistometer) Test.
⑤ Kelly Ball test.

→ Among these, the slump and compaction factor test is commonly used due to easyness in laboratory or in field.

① Slump test:

→ Most common method.

→ Not suitable for very wet and very dry concrete.

→ used conveniently as a control test and gives an indication of the uniformity of concrete from batch to batch.

→ The apparatus For conducting the slump test consists of a metallic mould in the form of frustum of cone having the dimension as -

Base diameter (ϕ_b) = 203mm

Top diameter (ϕ_T) = 102mm

height = 305mm.

→ The thickness of metallic sheet for mould shouldn't be thinner than 1.6mm.

→ For tamping the concrete a steel tamped rod having 16mm diameter and 0.6m long with bullet end is used.

Procedure:

→ The mould is placed on a smooth, horizontal, rigid and non- absorbent surface, inner surface of mould should be throughly clean and free from adherence of any old set concrete.

→ fill the mould with fresh concrete in 3 equal height layers.

→ Each layer is tamped 25 times by the tamping rod. After the top layer has been rodded the concrete is struck off level with the trowel.

→ The mould is removed immediately by raising it slowly and carefully in a vertical direction to allow the concrete to subside.

→ This subsidence is refered as slump of concrete (in mm) or The decrease in the height of the centre of the slumped concrete is called slump.

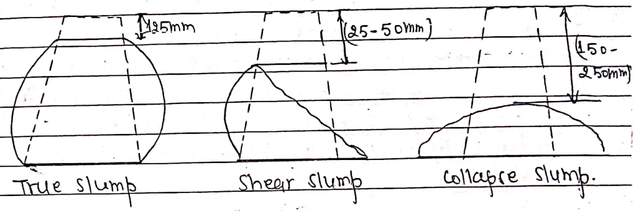


fig: Different types of Slump.

② Compaction factor test:

→ It is more precise and sensitive than the slump test and is particularly useful for concrete mixes of very low workability. It is developed in Road Research lab in UK.

→ This test works on the principles of determining the degree of compaction acheived by a standard amount of workdone by allowing the concrete to fall through a standard height.

→ The degree of compaction called the compacting factor is measured by the density ratio.

$$\text{compacting factor (CF)} = \frac{\text{wt. of partially compacted concrete.}}{\text{wt. of fully compacted concrete for equal volume}}$$

Procedure:

- Take full concrete in upperhopper.
- Open the trap door of upperhopper to fall concrete into lower hopper.
- Level the cylinder with plane blades supplied with the apparatus and clean the cylinder outside and take wt. of this concrete in the cylinder which is called weight of partially compacted concrete.
- Empty the cylinder and then refilled with concrete the same sample in 5 cm layers each with heavy vibration or ramming for full compaction then take weight which is called weight of fully compacted concrete.

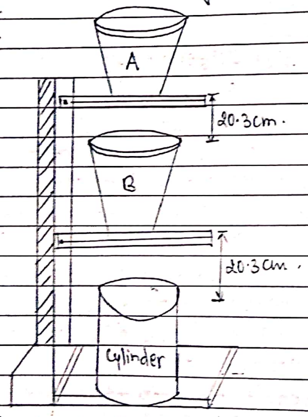


fig: compacting factor apparatus

Dimensions.

Hopper A and B. Top internal dia(D) = 25.4cm, 22.9cm
Bottom " " = 12.7cm, 12.7cm
Internal height = 27.9cm, 22.9cm.

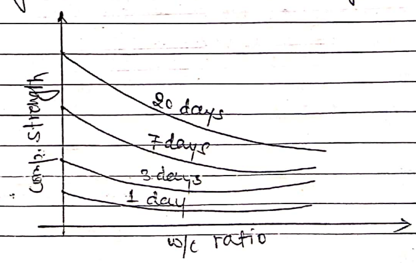
Cylinder, Internal dia(D) = 15.2cm. Internal height = 30.5cm.

3.2 W/c ratio in concrete:

- Water cement ratio (w/c) can be expressed in terms of weight and volume. In terms of volume, water used in litres per bag of cement (50kg) is taken as water cement ratio by volume. In terms of weight, quantity of water to be used per unit weight of cement is known as w/c ratio.

eg. ⓐ 500gm of water is used per kg of cement then.
w/c = 0.5 (by weight)
ⓑ If 25 litre of water is used per bag of cement (50kg) then
w/c = 0.5 (by volume).

- The minimum w/c ratio is 0.38, required for concrete for complete reaction (hydration) with gel pores.
- w/c ratio can be reduced by using plasticizers and superplasticizers.
- w/c ratio has great influence on green concrete against as hardend of concrete.
- It will have significant influences on workability. The higher w/c ratio, higher will be the fluidity of concrete which increases workability for uncontrolled concrete.
- On increasing w/c ratio decreases strength of concrete.



3.3 **Introduction to nominal mix:**

→ A mix having fixed cement aggregate ratio which insure adequate strength is called nominal mix design. It is used for M20 or less strength of concrete.

→ Mix design is defined as the process of selecting suitable ingredients of concrete and determining the relative quantities with the purpose of producing a economical concrete which has certain minimum properties (durability, strength, workability).

Information required for mix design:

→ Grade of concrete.
→ Types of cement to be used.
→ Type and maximum size of Aggregate.
→ Minimum cement content.
→ Maximum water cement ratio (w/c) by weight.
→ Degree of workability.
→ Exposure condition.
→ Types of admixture if used.
→ Maximum temperature of fresh concrete.
→ Method of placing.
→ Method of supervision etc.

proportion of nominal mix:

M10 → 1:3:6 → For mass concrete works.
M15 → 1:2:4 → for RCC, beam, columns etc.
M20 → 1:1½:3 → for hydraulic structure, piles
M25 → 1:1:2 and foundation.

3.4 **probabilistic concept in mix design approach:**

→ With the given materials, the four variables to be considered in connection with specifying a concrete mix are:
ⓐ water-cement (w/c) ratio.
ⓑ cement content or cement-aggregate ratio.
ⓒ Gradation of Aggregate.
ⓓ consistency.

→ The strength of concrete determined through the cube specimens varies with the size of the cube. The strength of the specimen increases with decreases in its size. The below table gives approximate distribution of strength of concrete based on the size of the cube.

Cube Size (mm)	100	150	200	300
Relative strength with 150mm cube	1.05	1.00	0.95	0.87

→ If these value plotted in histogram, results are found so follows a bell shaped curve known as "Normal distribution curve"

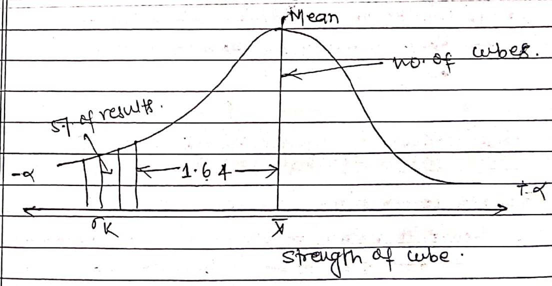


fig: Normal distribution curve.

Terms.

i) Mean strength $(\bar{X}) = \frac{\Sigma X}{n}$

ii) Standard deviation (σ).

$$\sigma = \sqrt{\frac{\Sigma (X-\bar{X})^2}{n-1}}$$

(iii) Coefficient of variation (CV).

$$CV = \frac{\sigma}{\bar{X}} \times 100.$$

→ The value of standard deviation or coefficient of variation could be used to determine the average design strength of the mixes. The following relationship can be used.

→ Target strength or avg. design strength

$$f(t) = f_{ck} + 1.64\sigma \text{ or } (K\sigma)$$

where,

$f(t)$ - Target strength.
f_{ck} - Characteristics strength.
σ - standard deviation.

Characteristics Strength:-

→ Volume of strength of materials below which not more than 5% of the result are expected to fall.

→ For example, If 100 specimens are made for the design of M20 concrete then more than 95 samples have more than 20 MPa compressive strength.

→ $f_{ck} = f_t - 1.64\sigma$

→ f_{ck} is called characteristics strength.

Grading of concrete (M20):

→ The number 20 refers to characteristics compressive strength of 15×15×15 cm³ cubes at 28 days expressed in N/mm² or MPa.

3.5 Concrete mix design by DOE, ACI and IS method:

(A) ~~Method~~

→ The main purpose of concrete mix proportioning are.
(a) To ensure workability in green state.
(b) To ensure required strength, durability and surface finish in the hardned state.

[A] Concrete mix design by DOE or BRE or British mix design method:

DOE → Department of environment
BRE → Building research establishment.

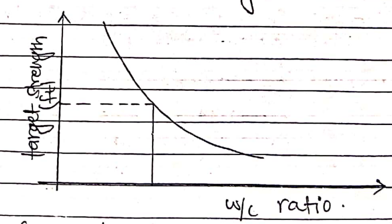
PROCEDURE:

(1) Determine the target strength from characteristics strength.

$$f_t = f_{ck} + 1.64\sigma$$

where, f_t - target strength.
f_{ck} - characteristics strength.
σ - standard deviation
= 4 for M20 or M25
= 5 for > M30.

(2) Determine minimum water cement ratio from target strength and check for durability condition.



(3) Calculate the free water content depending upon maximum size of aggregate and types of aggregate to get required workability.

④ Calculate the cement content from water cement ratio and water content comparing the cement content obtained with durability and other conditions.

⑤ Determine weight density of concrete depending upon the free water content and relative density of the combine aggregate based on saturated and saturated surface dry condition (SSD) condition.

⑥ Calculate the total aggregate content as,

Total aggregate content = $\gamma_\omega - W_c - W_w$

Where,

γ_ω – wet density of concrete.

W_c – wt. of cement.

W_w – wt. of water.

⑦ Determine proportion of fine aggregate depending upon water cement ratio, maximum size of aggregate, grading zone of fine aggregate and workability.

Then,

Fine aggregate = Total aggregate contain * proportion of fine aggregate.

Coarse aggregate = Total aggregate – fine aggregate.

⑧ Find the proportion of ingradients.

C : S : CA / (w/c).

Where, C – cement.

S – Sand (fine aggregate).

CA – coarse aggregate.

w/c – water cement ratio.

[B] Concrete mix design by ACI method.

Procedure.

① Determine the target strength from characteristics compressive strength.

$$f_t = f_{ck} + 1.64\sigma$$

② Determine the minimum water cement ratio from the target strength and check for durability condition.

③ Determine the amount of mixing water for the given slump and maximum size of Aggregate.

④ Determine the cement content from water cement ratio and water content and check for durability condition.

⑤ Determine the amount of coarse aggregate required for the unit volume and the value this obtained is multiplied by dry rodded unit wt. of aggregate to the required dry weight.

⑥ Determine the amount of fine aggregate from.

$$W_m = 10\rho_A(100-A) + \gamma_c\left(1-\frac{\rho_A}{\rho_c}\right) - \gamma_w(\rho_A - 1)$$

Where,

W_m – wt. of fresh concrete.

ρ_A – Avg. sp. gravity of combine fine and coarse aggregate.

ρ_c – sp. gravity of cement.

γ_c – cement content.

γ_w – water content.

A – Air content.

⑦ Then fine aggregate = Wm - all other constituents.

⑧ Adjust the mixing water quantity based on moisture content on aggregate.

⑨ Find the proportion of all ingradients.

C : S : CA / (w/c).

[C] Concrete mix design by IS method:

PROCEDURE:

① Determine target strength based on 28 days characteristics strength.

$$f_t = f_{ck} + 1.64\sigma$$

② Determine the minimum water cement ratio based on target strength and check for durability condition.

③ Determine the amount of entrapped air for the maximum size of Aggregate.

④ The water content and percentage of sand in total aggregate are selected based on water cement ratio, workability and maximum size of Aggregate.

⑤ Make the adjustment in water content and percentage of fine aggregate for other conditions.

⑥ Determine the cement content from water cement ratio and final water content after adjustment.

⑦ Determine the quantity of coarse and fine aggregate from these equations.

$$V = \left[W_w + \frac{W_c}{S_c} + \frac{1}{p} \cdot \frac{W_s}{S_s}\right] * \frac{1}{1000}$$

$$V = \left[W_w + \frac{W_c}{S_c} + \frac{1}{1-p} \frac{W_{CA}}{S_{CA}}\right] * \frac{1}{1000}$$

Where,

V – Net volume of fresh concrete.
S_c – Sp. gravity of cement.
S_s – Sp. gravity of sand.
S_{CA} – Sp. gravity of coarse aggregate.
W_c – wt. of cement.
W_{CA} – wt. of coarse aggregate.
W_s – wt. of sand.
W_w – wt. of water.
p – ratio of fine aggregate to total aggregate.

⑧ Obtained the actual amount of water to be added after making correction for water absorption by aggregate and free moisture present.

⑨ Find the required proportion of all consitituents or gradients.

C : S : CA / (w/c).

3.6 **Segregation and bleeding**

→ The stability of concrete mix requires that, it should not segregate and bleed during transportation and placing.

Segregation:

→ Segregation is defined as the separation of the constituents of a heterogeneous mixture so that their distribution is no longer uniform.

→ The main cause of segregation is difference in size of aggregate (specific gravity of aggregate) and it controlled by proper grading and care handling.

→ There are two forms of segregation.

(i) The coarser particles tend to separate out since they travel along a slope or settle more than finer particle. This type of segregation occurs if mix is too dry.

(ii) separation of grout (cement + water) from the mix. This type of segregation occurs if mix is too wet.

~~[illegible]~~ → when correct method of handling, transporting and placing the likelyhood of the segregation can be greatly reduced.

→ The tendency to segregate can be determined by reducing the height of drop of concrete.

Bleeding:

→ Bleeding is a form of segregation in which some of the water in the mix tends to rise to the surface of the freshly placed concrete. In other words, the separation of cement paste from the mix is termed as bleeding.

→ This is caused by inability of solid constituent of the mix to hold all of the mixing water when they are settle downwards. As a result of bleeding the top layer of concrete placed may become too wet and porous weak and not durable concrete will result.

→ In some cases bleeding may create capillary channels which increase the permeability of concrete.

→ ASTM standard for the measurement of rate of bleeding.

(i) A sample of concrete is placed in a container of 250mm diameter and 280mm height.

(ii) The bleeded water accumulated on the surface is withdrawn at 10 minute interval during 1st 40 min and at 30 min interval. Bleeding is expressed in terms of amount of accumulated water as the percentage of net mixing water in the sample.

Hence,

$$\text{Bleeding} = \frac{\text{Total sum of bleed water}}{\text{Net mixing water}}$$

3.7 **Quality control in site: Mixing, handling, placing, compaction and curing:**

→ **Quality control in site:**

→ Concrete is generally produced in batches at the site with the locally available matter of variable characteristics. It is therefore likely to vary from one batch to another. The variation depends upon several factors such as.

* variation in quality of constituents
* variation in mix proportion.
* variation in mixing.
* quality of overall workmanship and supervisional.

→ The aim of quality control is to reduce the different variations and produce uniform material providing the characteristic desirable for the job.

→ It is necessary to define the quality of concrete in terms of desired performance characteristic, economics safety and other factors.

Advantages of quality control:-

(1) Quality control is the rational use of available resources after testing their characteristics resulting in the reduction of material cost.

(2) Quality control reduces the maintenance cost

(3) In the absence of quality control of site the designer is attempt to over design so as to minimize the risk. These adds to the overall cost.

Mixing:

→ The mixing operation consists essentially of rotation or stirring, the objective being to coat the surface of all the aggregate particles with cement paste, and to blend all the ingredients of concrete into a uniform mass; this

uniformity must not be disturbed by the process of discharging from the mixer.

→ The usual type of mixer is a batch mixer, which means that one batch of concrete is mixed and discharged before any more materials are put into the mixer. There are four types of batch mixers.

(i) A tilting drum mixer
(ii) A non-tilting drum mixer
(iii) A pan type mixer
(iv) A dual drum mixer.

Handling:-

→ There are many methods of transporting concrete from the mixer to the site and, in fact one such method was discussed. The choice of method obviously depends on economic consideration and on the quantity of concrete to be transported. There are many possibilities, ranging from wheelbarrows, buckets, skips and belt conveyors to special trucks and to pumping but, in all cases, the important requirement is that the mix should be suitable for the particular method chosen. i.e it should remain cohesive and should not segregate.

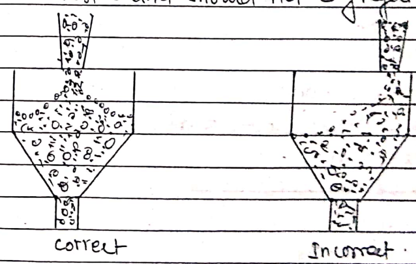


fig: Control of segregation on filling concrete buckets

placing and compacting: (Refer Neville and Brooks page 133)

→ The operations of placing and of compacting are interdependent and are carried out almost simultaneously. They are most important for the purpose of ensuring the requirements of strength, impermeability, and durability of the hardened concrete in the actual structure. As far as placing is concerned the main objective is to deposit the concrete as close as possible to its final position so that segregation is avoided and the concrete can be fully compacted.

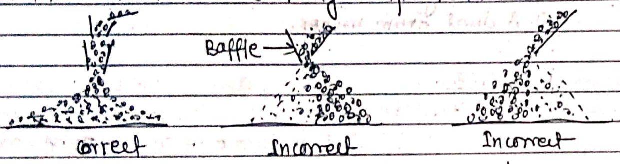


fig: control of segregation for fully compaction.

Curing:

→ curing is defined as the process of maintaining a satisfactory moisture content and favorable temperature in concrete during the period immediately after the placement of concrete, so that the hydration of cement may continue till the desired properties are developed sufficiently to meet the requirement of service.

Objective of curing:

→ keep concrete saturated.
→ Reduce shrinkage.
→ preserve properties of concrete.
→ To prevent loss of water by evaporation and maintain the hydration.

Methods of curing:

(1) Sprinkle of water (2) Steam curing
(3) pounding of concrete (4) Member curing.
(5) Shading of concrete work.

3.8 Concrete in extreme temperatures

Concreting in hot weather:

→ Any operation of concreting done at atmospheric tempr above 40°c or where the tempr of concrete at the time of placement is expected to beyond 40°c may be categorized in hot weather concreting.
→ A higher tempr a fresh concrete results in more rapid hydration of cement and therefore accelerated setting time and may reduce long term strength.
→ The rapid hardening of concrete before compaction accelerated chemical activity result in rapid setting and rapid evaporation of mixing water.
→ Due to rapid evaporation, plastic shrinkage in concrete occures and would cause cracking.
→ While concreting in hot weather following prevention may be adopted.
(i) The tempr of concrete may kept as known as possible by shading aggregate and mixture.
(ii) The tempr of aggregate may be lowered by sprinking water over them.
(iii) Tempr is lowered by shading pipelines or water tanks.
(iv) The crushed ice may be used.
(v) Rapid hardening is also reduced by working at night.
(vi) For curing moisture retaining materials are used.
→ The effects of hot weather concreting may be as.
(i) accelerated setting.
(ii) reduction in strength.
(iii) increase tendency to cracking.
(iv) rapid evaporation during curing.

Concreting in cold weather:

→ Any concreting operation done at a tempʳ below 6°C is termed as cold weather concreting.

→ If concreting is done at freezing tempʳ it will have harmful effect on the properties of concrete.

The effects are:-

(i) Setting is suspended if concrete freezes immediately after it has be placed.

(ii) If concrete freezes after it has set but before it has attain sufficient strength, the expansion due to formation of ice causes disruption and irrepairable loss of strength.

(iii) If concrete has required a sufficient strength before freezing it can withstand the internal pressure generated by formation of ice form the remaining mixture of water but its quality is small because at this stage sum of the mixing water will have combine with the cement and sum will be located in a small gel pores and this not be able to freeze.

→ While concreting in cold weather following precaution are adopted.

(i) To increase the tempʳ of fresh concrete, mixing water may be heated but not above 80°C as more hot water will cause flash set of the cement.

(ii) If heating of water does not rise the tempʳ of concrete to the desired value aggregate also be heated not above 25°C.

(iii) Aggregate should be heated uniformly.

(iv) Use of cement with high C_3S and C_3A content.

(v) By using accelerator agent.

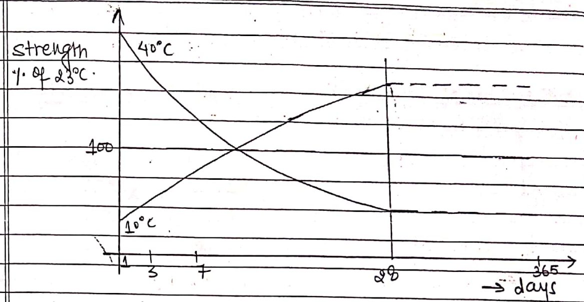


fig: Variance of strength of concreting in hot and cold weathers.

Probable questions of in this chapter:

2068 ① Describe the stepwise process of mix-design of concrete by ACI method.

2068 ② What measures do you recommend for quality control to concrete at site? Explain briefly.

③ Explain the BS method of mix design.

④ What is characteristic strength and target strength of concrete? How the cube size affect the strength of concrete?

⑤ Detail out stepwise process of mix design using IS method.

⑥ What do you understand by workability of concrete? Explain the slump test with the types of slump.

⑦ What do you understand by nominal mix?

⑧ List out the difference between segregation and bleeding.

⑨ Describe the concrete in extreme temperature, briefly?

CHAPTER-4

PROPERTIES OF HARDENED CONCRETE [6]

4.1 Deformation of hardened concrete, Moduli of elasticity:

Deformation of hardened concrete:

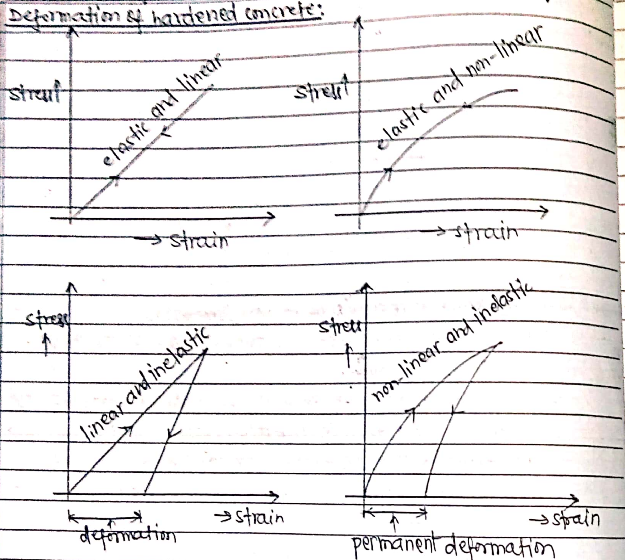


fig: strain-stress relationship of concrete.

→ To be able to calculate the deformation and deflection of the structural member we have to know the relation between stress and strain. The concrete behaves nearly elastically when load is first applied however under sustain loading the strain increases with time under a constant stress (i.e creep). At very low stress these behaviour of concrete is not very pronounced but at moderate and high stress the behaviour of concrete is non-linear and inelastic.

Modulus of elasticity:

→ slope of the Relationship between stress and strain is called modulus of elasticity. but youngs modulus is applied strictly for linear catagorized only.

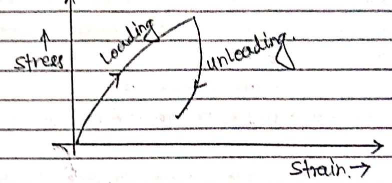


fig: stress-strain behaviour of concrete.

→ If the load is applied extremely rapidly (less than 0.01sec) the strain are greatly reduced and the curvature of stress-strain curve become very small.

→ An increase in loading time from 5sec to 2min. can be increase the strain by up to 15%.

→ An increase in loading within the range of 2min to 10min, the increase in strain behaves as non-linear.

→ Modulus of elasticity is the measurement of stiffness or resistance to deformation.

Types of modulus of elasticity:

1. Static modulus of elasticity.
   (a) Initial tangent modulus of elasticity.
   (b) Tangent modulus of elasticity.
   (c) Secant modulus " "
   (d) Chord " "
2. Dynamic modulus of elasticity.
3. Flexural modulus of elasticity.

① Static modulus of elasticity:

→ The value of modulus of elasticity determined by actual loading of concrete is static modulus of elasticity. It is given by the slope of stress-strain curve for concrete under uniaxial loading since the curve for concrete is non-linear, the four methods used to determine the modulus of elasticity.

(a) Initial tangent modulus.

→ The tangent of curve at the origin.

(b) Tangent Modulus.

→ The slope of the line drawn tangent to the stress-strain curve at any point.

(c) secant modulus.

→ slope of line drawn from the origin to the point on the curve corresponding to 40% stress of the failure load.

(d) Chord modulus.

→ The slope of line drawn between the two points of the stress-strain curve is called chord modulus.

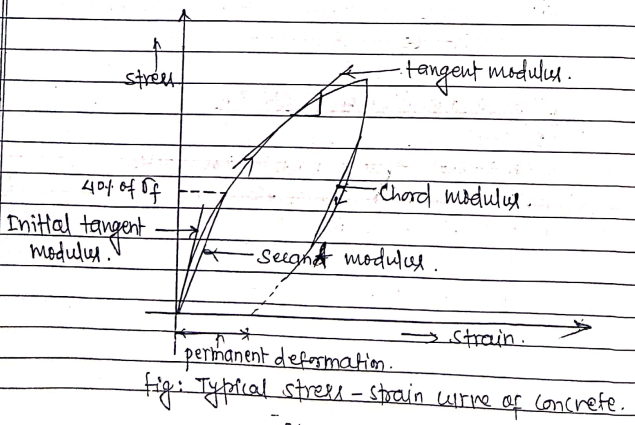


fig: Typical stress-strain curve of concrete.

② Dynamic modulus of elasticity:

→ Due to several cycles of loading and unloading reduce the subsequent creeps show that the stress-strain curve on subsequent loading exhibites only a small curvature, this slope of curve is dynamic modulus. The dynamic modulus of elasticity corresponding to a very small instantanous strain approximately given by the initial tangent modulus and it is also calculated by.

$$E_d = K n^2 L^2 \cdot \rho$$

where, E_d - dynamic modulus of elasticity.

K - constant.

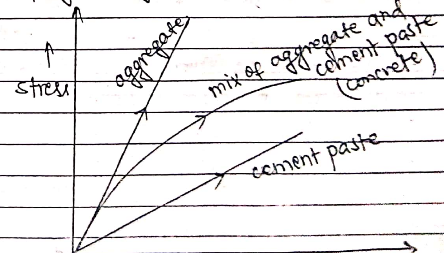
n - resonant frequency.

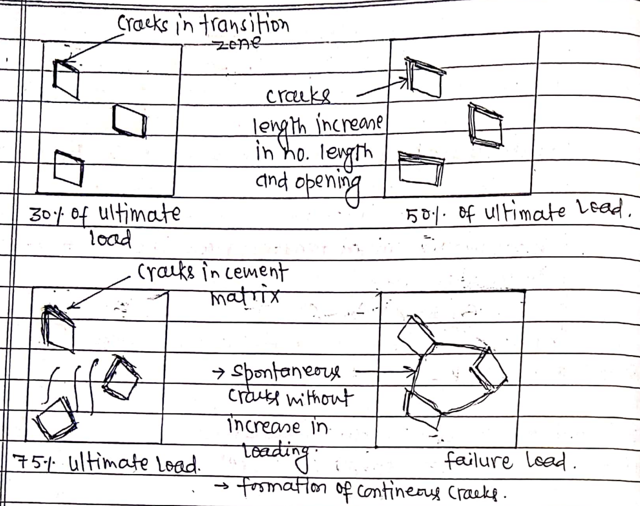
L - Length of specimen.

ρ - density of concrete.

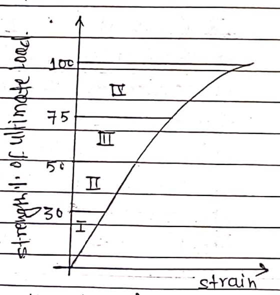
③ Flexural modulus of elasticity:

→ The stress-strain curve of concrete become immediatly apparent relative to aggregate and cement paste. concrete is neither a elastic material nor liner. The cause of non-linearity in the stress-strain relationship has been explained from the studies on the process of progressing micro cracks in concrete under load.





→ As we know micro cracks are already exist in transition zone even before loading. Below 30% of ultimate load the transition zone cracks remain stable. Therefore the stress-strain curve remains linear.



→ Above 30% of the ultimate load, as the stress increases the transition zone micro cracks are begin to increase in length width and number and upto 50% a stable system of microcracks may be assumed.

→ If the load still increases the microcracks begin to form in the cement matrix and the curve is bent considerably towards horizontal.

→ And the load is further increase more than 75% of ultimate load the cracks seems to reach the critical level.

→ Modulus of elasticity of concrete.

$$E = 5700\sqrt{f_{ck}}$$

→ poisson's ratio of concrete – (0.16–0.2).

4.2 Shrinkage and Creep:

→ Increase in strain of the concrete element with time under sustained load is known as creep.

→ The cause of creep in concrete is more complex. In addition to moisture movement their are other causes that contribute to creep phenomenon. When a stress level greater than 30% to 40% of ultimate stress. The contribution of the transition zone microcracks to creep.

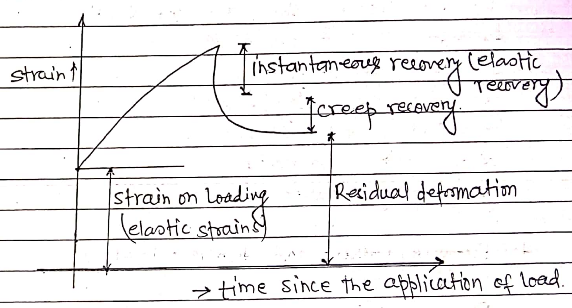


fig: creep.

→ Withdrawal of water from harden concrete stored in unsaturated air causes drying shrinkage. The shrinkage strain is due to a differential relative humidity between the concrete and the environment a loss of physically adsorbed water from C-S-H structure while the creep strain due to the sustain applied loading.

→ When the concrete is exposed to drying condition the creep strain is caused by the additional micro cracking in the transition zone due to drying shrinkage.

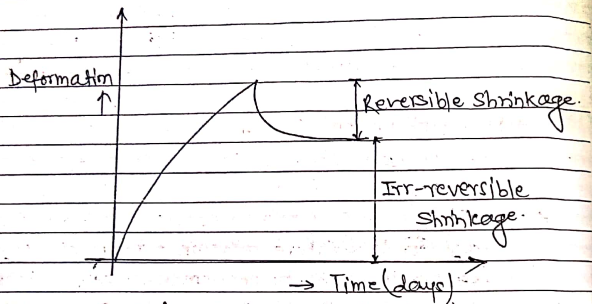
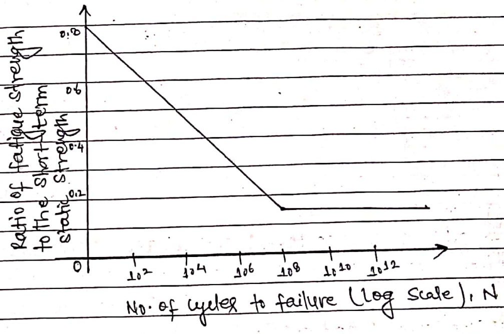


fig: Shrinkage.

Factors affecting shrinkage and creep:
→ Aggregate contained.
→ water cement ratio
→ Age of application of load.
→ Humidity and curing.
→ Modulus of elasticity.
→ size of member.

4.3 Fatigue, impact and dynamic Loading:

Effect of fatigue:
→ Two types of failure in fatigue can takes place in concrete.
① failure occures in sustained load (or a slowly increasing load) near but below, the strength under an increasing load, Known as static fatigue or creep rupture failure.
② Failure occurs under cyclic or repeated loading known as fatigue failure.



→ The repeated loading is applied in concrete structure like road pavement, airport pavement etc when the material is fail under a no. of repeated loads failure in fatigue is said to takeplace.
→ For the constant load of alternating stress the fatigue strength decrease as the no. of cycle increases, this is also given by Aas - Jacobsen.

$$\log N = \frac{1 - \frac{f_{max}}{f_c}}{\beta\left[1 - \frac{f_{min}}{f_c}\right]}$$

where,
$\beta = 0.085$
f_{max} = maximum stress in a cycle.
f_{min} = minimum stress in a cycle.
f_c = strength under static loading.

Effects of impact or dynamic loading:-

→ If the breaking of a structure or specimen takes place under a very short time of loading (i.e fraction of the number of cycles), strength under dynamic loading is observed. This is basically the interaction of the $\frac{f_{max}}{f_c}$ versus log N curve with the $\frac{f_{max}}{f_c}$ axis. This can be termed as strength under impact load. This is complicated experiment to be carried out.

→ The CEB-FIP Model code (1990) recommends that the increases in compressive strength due to impact with rates of loading less than 106 MPa/sec, can be computed using the relationship.

$$\frac{f_{c,imp}}{f_c} = \left(\frac{\sigma}{\sigma_0}\right)^{\alpha_s}$$

where

$f_{c,imp}$ = impact compressive strength.

f_c = compressive strength of concrete.

σ = impact strength rate.

σ_0 = 1 MPa

$$\alpha_s = \frac{1}{5 + 9\frac{f_c}{f_{co}}}$$

f_{co} = 10 MPa.

→ The ultimate strength is also effected by the rate of loading. Due to progressive microcracks at sustained loads a concrete will fail at a lower stress than that induced short term loading.

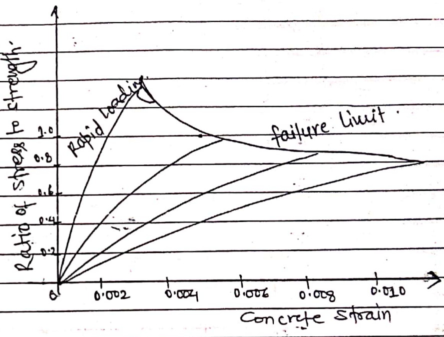


fig: Relationship between the short-term and long-term loading strength of concrete.

4.4 Effect of porosity, water cement ratio and aggregate size:

Effect of porosity: [Refer Neville and Brooks page 101]

→ It is the primary factor that governs the strength of concrete.

→ The water cement ratio is major influencing factor on porosity.

→ It is the depends on the degree of hydration and water cement ratio.

→ The decrease in porosity with an increase in the degree of hydration.

→ The magnitude of porosity is such that, for the usual range of w/c ratios, the cement paste is only about 'half solid'. For instance, at a w/c ratio of 0.6, the total volume of pores is between 47 and 60 percent of the total volume of the cement paste, depending on the degree of hydration.

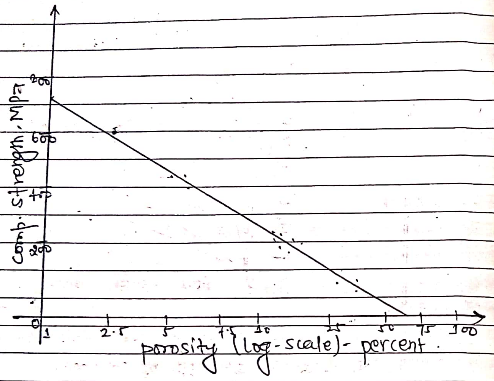


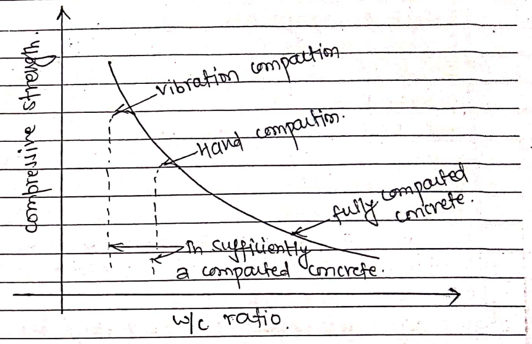
fig: Relation between compressive strength and Logarithm of porosity.

Effect water cement ratio:

→ w/c ratio is an index of strength of concrete. The strength of cement paste increases with increasing cement content and decreases with water content and air content.

→ For the fully compacted concrete at a given age and thermal temperature. Its strength is taken to be inversely proportional to w/c ratio.

→ In the below figure, strength Vs w/c ratio shown that lower w/c ratio could be used when the concrete is vibrated to get higher strength where as comparetively higher w/c ratio is required when concrete is hand compacted or manually compacted.



→ In both cases, when the w/c ratio is below the practical limit the strength of concrete fall rapidly due to introduction of air voids.

→ The ~~Relation~~ Relation between compressive strength of concrete and w/c ratio is introduced by Abram's Law.

He state that "with given materials and condition of test of the quantity of mixing water to the quantity of cement alone determines the strength of concrete and it is independent of aggregate cement ratio, so long as the mix is of workable plasticity".

Mathematically,

$$C = \frac{A}{B^x}$$

where,

C - compressive strength of concrete.

A and B - Emperical constants.

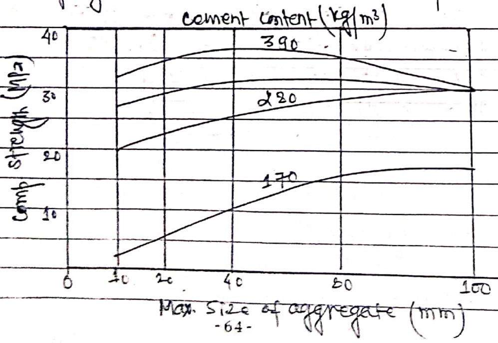
x - water cement ratio.

Effect of aggregate size:-

→ Second to the w/c ratio, aggregate is an important factor affecting the concrete strength. The most important properties of concrete are shape, texture and size of aggregate.

→ Maximum size affects the strength in several ways. Larger particle reduce the specific surface area of aggregate which leads a reduction in bond. Also larger particles tend to more strain volume change in cement paste and therefore induced more internal stress. which will weaken the concrete. Aggregate effects can be offset by reducing the water content. In general at a constant w/c ratio, higher strength can be obtained by using linear mixture. If constant workability ~~at~~ is maintained, strength will increase with cement content.

→ High strength concrete or rich mix concrete is adversely affected by the use of larger sized aggregate when large sized aggregate is used, due to internal bleeding in the transition zone will become much weaker by developing microcracks with lower comp. strength.



4.5 Effect of gel/space ratio:-

→ Gel/space ratio is defined as the ratio of hydrated cement paste to the sum of volume of hydrated cement and capillary voids.

If, c be the mass of cement
V_c be the sp. volume of cement
w_o be the volume of mixing water
α be the rate of hydration.

Power's equipment Assumption.

1 cm³ (volume of cement) = 2.06 cm³ (volume of cement after hydration)

→ 1 cm³ of cement produced 2.06 cm³ of hydrated product.

Gel/space ratio = $\dfrac{\text{hydrated cement paste}}{\text{volume of cement + volume of water}}$

$$G_{SR} = \frac{2 \cdot c \cdot V_c \cdot \alpha}{c \cdot V_c \cdot \alpha + w_o} = \frac{2}{\left(1 + \frac{w_o}{c \cdot V_c \cdot \alpha}\right)} \quad \text{——— (1)}$$

If, $V_c = 0.319$ ml/g.

$$G_{SR}(r) = \frac{0.657\alpha}{0.319\alpha + \frac{w_o}{c}} \quad \text{——— (2)}$$

$$\text{porosity } (p) = \frac{V_v}{V} \quad \text{——— (3)}$$

or, $1 - p = 1 - \dfrac{V_v}{V} = \dfrac{V - V_v}{V}$

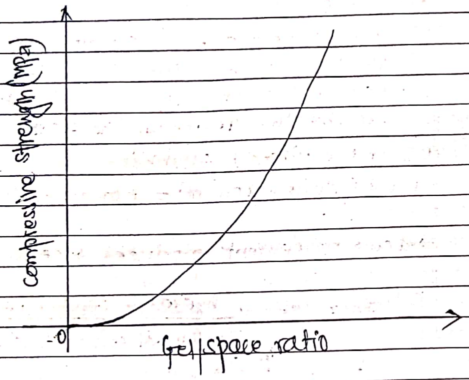
or, $1 - p = \dfrac{V_s}{V} = r$

$$\therefore \boxed{1 - p = r} \quad \text{——— (4)}$$

where,
r - Gel/space ratio
p - porosity.

Compressive strength = $234\, r^3$ (MPa) ——— (5)

from the relation eqn ⑤ shows the increases the gel/space ratio, also increase the the compressive strength of concrete.



probable questions in this Chapter:

2068 ① What is elastic deformation of concrete? Explain Shrinkage and creep of concrete.

② What are the factors affecting Shrinkage? Describe the performance of concrete under repetitive loading.

Imp ③ Explain the stress-strain behaviour of concrete in relation with progress of micro-cracks.

④ How water-cement ratio affects strength of concrete? Describe in brief.

⑤ How does size and grading of aggregate affect strength of concrete? Explain.

⑥ Describes briefly effect of gel/space ratio on the strength of concrete?

⑦ What ~~are~~ is the type of modulus of elasticity? described brief in each type.

CHAPTER-5

TESTING OF CONCRETE AND QUALITY CONTROL [12]

5.1 Various strength of concrete: Tensile, compressive, shear and Bond

→ In concrete design and quality control, strength is the property, which is generally specified, although in practical cases other factors may be of importance like durability, impermeability and volume stability.

→ This is because (strength of concrete means compressive strength in general).

(i) It is easy to quantify — The major cause of reducing strength is porosity, microcracking of hcp are difficult to measure/quantify in useful manner.

(ii) In most of the cases concrete is employed primarily to resist compressive stress.

(iii) compressive strength can be used to related many properties of concrete such as elastic modulus, water tightness or impermeability, wear resistance, fire resistance etc.

→ Although, in practice, most concrete is subjected simultaneously to a combination of compressive, shearing and tensile stresses in two or more directions. The uniaxial compression tests are the easiest to perform in the laboratory and the 28-day compressive strength of concrete determined by a standard uniaxial compression test is accepted universally as a general index of concrete strength.

→ Strength of concrete is its resistance to rupture or failure ~~Ngt~~ may be measured in a number of ways, such as strength in compression, in tension, in shear or in flexure etc. All these indicate strength with reference to particular method of testing.

5.2 Compressive strength test:

→ In order to determine the compressive strength of concrete two types of speimen can be used.

1. Cube test
2. Cylinder test.

① Cube test:

→ Specimen are cast in steel or cast iron moulds of 150 mm × 150 mm × 150 mm dimension cube. The cube is filled in three layers and well compacted. After compaction the top surface is made smooth. The finished surface is left undisturbed for 24 hrs. at room temperature. Then the mould is removed and speimen is stored in water for further curing. usually the specimen's are cured for 28 days.

→ The testing of specimen are taken place at 1, 3, 7, 14, 28, 90 and 365 days. The test is performed by standard, uniaxial compression test and at each time minimum three samples are tested.

→ compressive strength of concrete.

$$= \frac{\text{force at failure}}{\text{Area}}$$

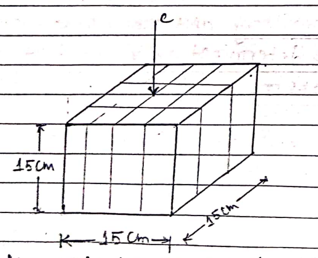


fig: uniaxial compression.

② Cylinder test:

→ According to British standard strength of cylinder
$$f_{cy} = \frac{4}{5} \times \text{cube}.$$

→ The standard cylinder mould of 150mm diameter and 300mm in height made of cast iron or steel. The cylinders are also filled in three layers and well compacted cylindrical specimen's are also used for determination of compressive strength of concrete varies with $\frac{h}{d}$ ratio. $[\frac{h}{d} = 2]$.

→ Experiment show that there is no relationship betn cube strength and cylindrical strength but the cube strength is always higher than cylindrical strength.

→ L' Hermite suggested that the ratio of strength of a cylinder and a cube be taken as.

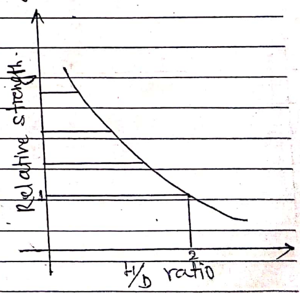
$$\text{Cylinder strength} = 0.76 + 0.2 \log_{10}\left(\frac{f_{cu}}{2840}\right)$$

where,

f_{cu} - strength of cube in pounds per square inch (lb/in^2).

→ But the actual relation between the strength of these two types of speimen depends on the level of strength and the moisture condition of concrete at the time of testing beside these the ratio of height and diameter also effect the strength of cylinder.

H/D ratio	Strength correction factor: American standard	British std.
2.00	1.00	1.00
1.75	0.98	0.98
1.50	0.96	0.96
1.25	0.93	0.94
1.00	0.87	0.92



5.3 Tensile strength test:

→ Since the concrete is very weak in tension and is not expected to resist tensile forces from external loading condition but due to the restrained condition of shrinkage and temperature effect, tensile stress are developed in concrete.

(a) Direct tension:

→ It is very difficult to apply uniaxial tension to a concrete specimen so it is rearly used.

$$f_{ct} = 0.35\sqrt{f_{ck}}$$

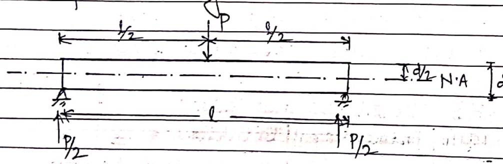
where,

f_{ct} – direct tension strength.
f_{ck} – Characteristic comp. strength.

(b) Modulus of rupture (flexural test)

→ This is the tensile strength under bending.
→ In the flexural test the maximum tensile stress is reached in the bottom fibre of test beam is known as modulus of rupture.

i) central point Loading.

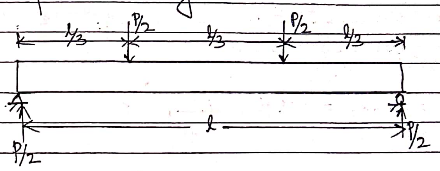


$$\frac{M}{I} = \frac{f}{y}$$

or, $f = \frac{M}{I} \times y = \frac{PL}{4I} \cdot d/2$

$\therefore f_{cr} = \frac{(PL/4)}{\left(\frac{bd^3}{12}\right)} \times d/2 = \frac{3}{2} \cdot \frac{PL}{bd^2}$ ——— (1)

ii) Third point Loading.



$\therefore f_{cr} = \frac{(PL/6)}{\frac{bd^3}{12}} \cdot d/2 = \frac{PL}{bd^2}$ ——— (2).

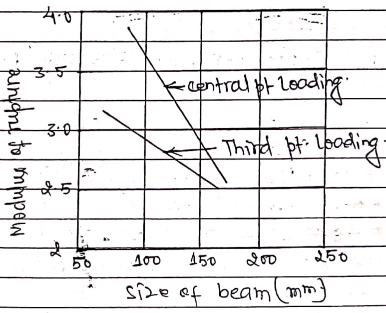


fig: Modulus of rupture of pcc beams of different sizes subjected to central point and third-point loading.

Then,

Modulus of rupture $(f_{cr}) = 0.7\sqrt{f_{ck}}$

→ The CEB – FIP model code given the relationship betn direct tensile strength and modulus of rupture of concrete.

$$f_{ct} = f_{ck}\left[\frac{2\left(\frac{h}{h_0}\right)^{0.7}}{1 + 2\left(\frac{h}{h_0}\right)^{0.7}}\right]$$

Where, h – depth of beam in mm
h_0 – 100 mm.

→ The CB-FIP model code also gives the upper and lower bound values of the tensile strength with compressive strength.

$$f_{ct,max} = 0.95 \left(\frac{f_{ck}}{f_{cko}}\right)^{2/3}$$

$$f_{ct,min} = 1.85 \left(\frac{f_{ck}}{f_{cko}}\right)^{2/3}$$

$$\cancel{f_{ct,mean} = 1.4} \left(\frac{f_{ck}}{f_{cko}}\right)^{2/3}$$

where,

f_{ck} – Characteristics strength (MPa)

f_{cko} – 10 MPa.

(c) Splitting test:

→ This test is also called Brazian test because it was developed from Brazil.

→ In this test a concrete cube or cylinder of the type used in compression strength testing is placed with its axis horizontal. The load is increase until the failure takesplace by splitting in the plane containing vertical diameter of the specimen.

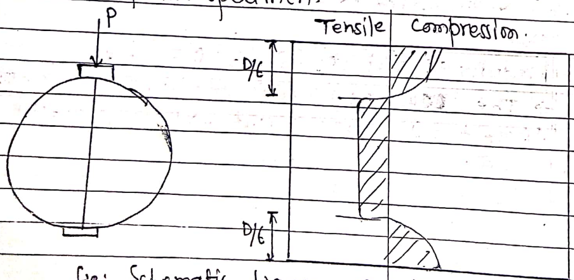


fig: Schematic diagram of splitting test.

The horizontal tensile strength is obtained by.

$$T = \frac{2P}{\pi L D}$$

where,

P – applied load

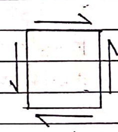
D – Diameter of cylinder

L – Length of cylinder.

③ Shear test

→ Shear is the action of two equal and opposite forces applied in planes a short distance apart. In general shear stress exist with tensile and compressive stresses.

→ pure shear can be applied through torsion of a cylindrical specimen in which case the stresses are equal in primary shear.



Secondary tension and secondary compression develop in a beam maximum at 45° due to shear. As the concrete is weak in tension than in shear failure in tension invariably occurs in diagonal tension.

→ Shear strength of concrete is about 12% of the compressive strength of concrete.

→ As per IS 456:2000, Higher the grade and percentage of steel, higher will be the permissible shear strength of concrete.

Concrete grade	M15	M20	M25	M30	M35	M40
Comp. Strength (N/mm²)	15	20	25	30	35	40
Max. Shear Strength τ_{max} (N/mm²)	1.6	1.8	1.9	2.2	2.3	2.5

④ Bond strength:

→ Bond strength is defined as the resistance to slip of the steel reinforcement bars which are embedded in concrete.

→ Bond strength considerably depends on type of cement, admixture and water cement ratio.

→ cement with higher C_2S gives higher bond strength. cement with high C_3S gives less bond strength.

→ Bond strength can be increased by 60% for deformed bar.

→ For compressive steel, bond strength can be increased by 25%.

Concrete grade	M20	M25	M30	M35
Comp. strength (MPa)	20	25	30	35
Bond strength (MPa)	1.2	1.4	1.6	1.8

5.4 Variability of concrete strength and acceptance criteria:

Variability of concrete strength.

→ Since strength is a variable quantity, when designing a concrete mix, we must aim at a mean strength higher than the minimum required from the structural standpoint so that we can expect every part of the structure to be made of concrete of adequate strength.

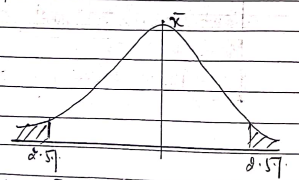
Let us suppose the mean strength ($\bar{x}$) = 40 N/mm2 and standard deviation (σ) = 4 N/mm2 then determine the 95% confidence interval.

Here,

$f_{ck} = \bar{x} \pm k\sigma$

$= 40 \pm 1.96 * 4$

$= 40 \pm 7.84$



95% confidence intervals

[32.16 N/mm2 to 47.84 N/mm2]

Acceptance criteria: [Refer IS 456: 2000 code]

[Clause 17.1.2, page 30]

ⓐ compressive strength for M15.

$f_{mean} \geq f_{ck} + 0.825 *$ established standard deviation.

or

$f_{mean} \geq f_{ck} + 3$ N/mm2 whichever is greater.

For M20 or above.

$f_{mean} \geq f_{ck} + 0.825 *$ established standard deviation.

or

$f_{mean} \geq f_{ck} + 4$ N/mm2. whichever is greater.

ⓑ For single test.

For M15, $f_i \geq f_{ck} - 3$ N/mm2

For M20 or above, $f_i \geq f_{ck} - 4$ N/mm2

where, f_i - strength of individual sample.

5.5 Non-destructing testing of concrete:

→ The tests performed without destruction of sample known as non-destructive test. These are used to estimation of the properties of concrete in the structure. The methods adopted included ultrasonic pulse velocity and rebound hammer (Schmidt hammer).

→ These methods are based on measuring a concrete property that bears some relationship to strength. The accuracy of these methods, in parts, is determined the degree of correlation between strength and the physical quality measured by the non-destruction.

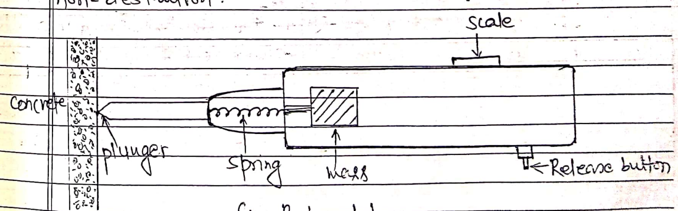


fig: Rebound hammer.

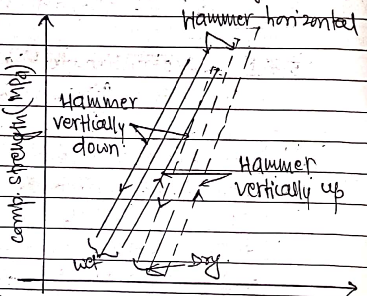
→ The test is preformed on the principle that, the rebound of elastic mass depends on the hardness of surface against which the mass strikes. figure shows the rebound hammer in which the spring loaded mass has a fixed amount of energy imparted to it by extending a spring to a fixed position, this is achieved by pressing the plunger against a smooth surface of concrete which has to be firmly supported. upon release, the mass rebounds from the plunger and the distance travelled by the mass, expressed as a percentage of the initial extension of the spring, is called the rebound number; it is indicated by a graduated scale.

→ The rebound number is related to compressive strength and individual hammer as.

→ The test is sensitive to presence of aggregates or voids so that it is necessary to take many reading over the area to be tested.

→ The plunger must always be normal to the concrete surface.

→ smooth surface is required.



Ultrasonic pulse velocity test:

→ The principles of this test is that the velocity of sound in a solid material, u, is a function of the square root of the ratio of its modulus of elasticity, E, to its density ρ, viz

$V = f\left(\frac{gE}{\rho}\right)^{1/2}$ where g is the acceleration due to gravity.

This relation can be used for the determination of the modulus of elasticity of concrete if poisson's ratio is known and hence as a means of checking the quality of concrete.

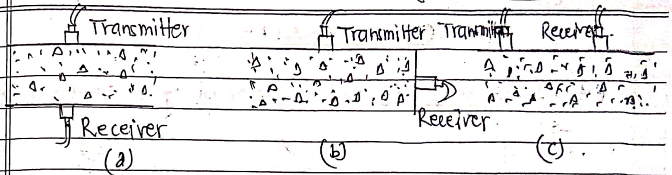


fig: Methods of propagating and receiving ultrasonic pulses (a) direct transmission (b) semi-direct transmission and (c) indirect or surface transmission.

→ A pulse of longitudinal vibration is produced by a transducer (transmitter + receiver) which is held in contact with one surface of the concrete under test. when the pulse generated is transmitted into the concrete from the transducer, it undergoes multiple reflections at the boundries of the different materials phases within the concrete, a complex system of waves developed. The first wave to reach the transit time (T) of the pulse to be measured Then velocity is given by $u = d/t$

probable questions in this chapter:

2068 ① Explain non-destructive testing process of concrete and its features.

2068 ② Explain the reasons for popularity of compressive strength test of concrete. Describe defferent methods of obtaining tensile strength of concrete.

③ Explain splitting test to obtain tensile strength of concrete.

④ Write short notes on ultrasonic pulse velocity test.

⑤ Explain importances of non-destructive testing of concrete.

⑥ What is Tensile strength test of concrete?

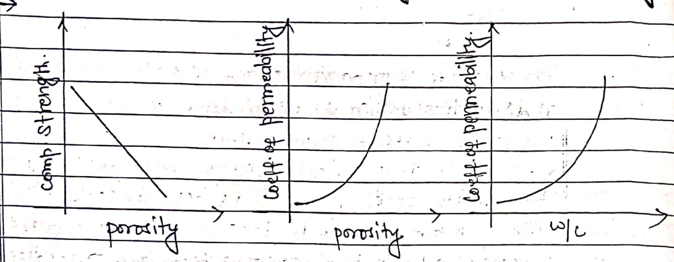
⑦ Explain the schmidt Hammer test.

⑤ what is non destructive test?

CONCRETE DURABILITY [6]

6.1. Effect of water and permeability on concrete durability.

→



→ For concrete made with usual normal weight aggregate 'K' is goverme by porosity of the cement paste. since the capillary 'n' is govern by the w/c ratio and degree of hydration. For the given degree of hydration permeability is lower for the paste with lower w/c. permeability also increase if more porous aggregates are used. From the durability point of view it may be important to achive low permeability as quickly as possible to reduce permeability.

i) A mix with low w/c ratio is advantages.

ii) concrete must be dense so better to use well graded aggregates.

6.2 physical and chemical causes of concrete deterioration:

1) sulphate attack
2) Attack by sea water.
3) Alkali aggregate reaction. (See 1.2)
4) Acid Attack.

① sulphate attack.

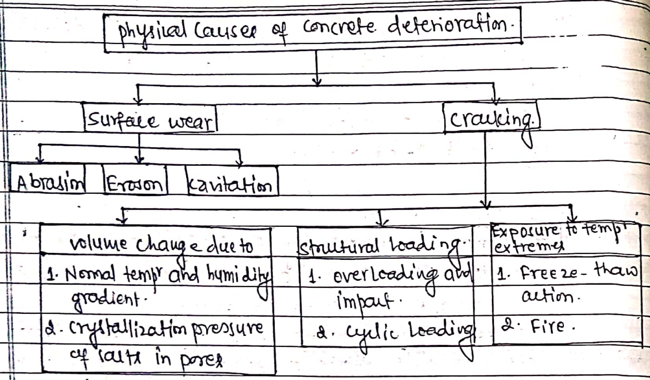
→ Concrete attack by sulphate, has a characteristics whitist appearence, damage usually starting at the edge and corners is followed by cracking and spalling of concrete. The region for this appearence is that the essence of sulphate attack is the formation of calcium sulphate and calcium sulphate aluminate both products occupying the greater volume then the compound which they replaced so that the expansion and destrution of harden concrete takesplace.

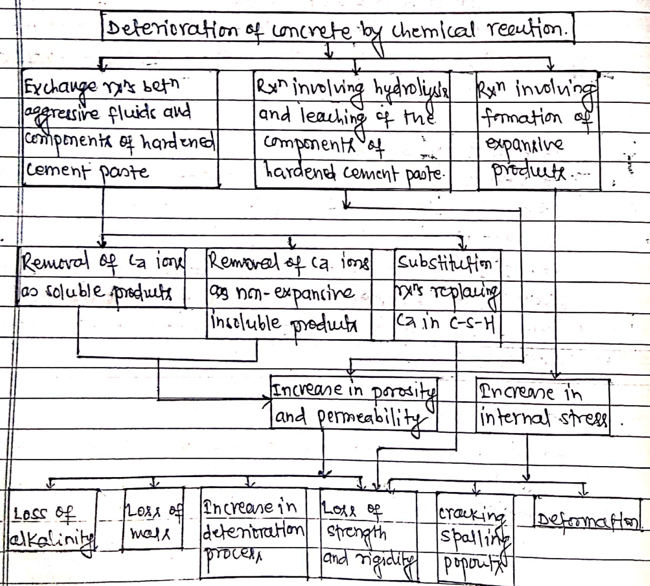
② Attack by sea water.

→ Sea water contains sulphate which caused the sulphate attack but because chlorides are also present is sea water attack does not generally cause expansion of concrete this is due to gypsum is more soluble in chlorides so their is no destruption but only a slow decrease in strength due to the corrosion in reinforcedment.

③ Acid Attack.

→ Sulphur dioxide and carbondioxide and other fumes in atmosphere form acid which attak concrete by dissolving and removing a part of the hydrated cement paste and a leave a very soft and weak mass. This acid attack is generally incounter in various industrial area.





6.3 Carbonation:

→ The reaction of carbondioxide with the products of cement hydration is known as carbonation.

→ The rate of carbonation is mainly influence by the permeability and calcium content of the concrete as well as the ambient atmospheric condition i.e amount of CO_2, relative humidity and temperature.

→ Carbonation is more rapidly in hot climate than in a moderate climate.

→ Carbonation has an adverse effect unability to protect of unreinforcement and cause cracking.

→ It reduce the permeability is main advantage of carbonation.

→ Also carbonation of concrete is a process by which carbondioxide from the air penetrates into concrete and reacts with calcium hydroxide to from calcium carbonates.

$$CO_2 + Ca(OH)_2 \longrightarrow CaCO_3$$

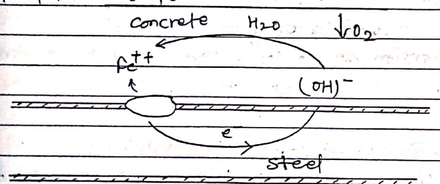
→ Conversion of $Ca(OH)_2$ into $CaCO_3$ by the action of CO_2 results in small shrinkage (increase in volume).

factors affecting rate of carbonation.

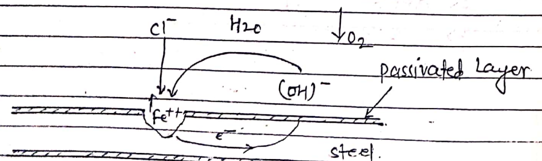
→ The level of pure water i.e relative humidity.
→ Grade of concrete.
→ permeability of concrete.
→ Depth of cover
→ Age of concrete.
→ wheather, The concrete is protected or not i.e exposure condition.

6.4 Corrosion of steel in concrete:

→ The strongly alkaline nature of $Ca(OH)_2$ (pH of about 13) prevents the corrosion of the steel reinforcement by the formation of a thin protective film of iron oxide on the metal surface; this protection is known as passivity. However, if the concrete is permeable to the extent that carbonation reaches the concrete in contact with the steel or soluble chlorides can penetrate right up to the reinforcement, and water and oxygen are present, then corrosion of reinforcement will take place. The passive iron oxide layer is destroyed when the pH falls below about 11.0 and carbonation lowers the pH to about 9. The formation of rust results in an increase in volume compared with the original steel so that swelling pressures will causes cracking and spalling of the concrete.
In the below figure, shows the process of corrosion of steel in concrete.



(a) fig: Electro-chemical process.



(b) fig: Electro-chemical corrosion in the presence of chlorides.

$Fe \longrightarrow Fe^{++} + 2e^-$ (anodic rxn)

$4e^- + O_2 + 2H_2O \longrightarrow 4OH^-$ (cathodic rxn)

$Fe^{++} + 2OH^- \longrightarrow Fe(OH)_2$ (Ferrous hydroxide)

$4Fe(OH)_2 + 2H_2O + O_2 \longrightarrow 4Fe(OH)_3$ (Ferric hydroxide)

Impact of corrosion:-

→ The corrosion products are more voluminious then the parent metal and their formation and deposition on the bar leads to surrounding concrete being subjected to excessive pressure and since concrete is relatively week in tension, longitudinal cracks along the bars are initiated such cracks also accelerates further corrosion by allowing the passing of chloride oxygen and water through concrete and change the structural behaviour of the members.

Effects of corrosion:-

- → Development of longitudinal cracks along the bar.
- → By corrosion, change the structural behaviour.
- → spalling of concrete cover.
- → Appearence of rust strains at the surface of concrete.
- → Reduction c/s of bars.

preventive measures of against corrosion:-

- → First and formest precaution is to provide dense concrete.
- → provide adequate cover to the reinforcing bar.
- → Use a slage cement or portland pozzalona cement.
- → By controlling the permissible crack width.

Extra
Factors affecting cracking of concrete:-

① Before hardening
- → Drying
- → construction practice
- → Early setting.

② After hardening.
- → Unsound material
- → shrinkage
- → Thermal cause
- → Moisture movement
- → Transition zone crack.
- → corrosion
- → chemical cause.

Effects of Air content in concrete:
- → Air entrainment increase workability.
- → Air entrainment reduce durability.
- → Air entrainment may increase by better hydration.
- → Air entrainment needed to resist freezing and thawing.

probable questions of this chapter:
2068 ① Explain, in brief, physical and chemical causes of concrete deterioration.
② Explain the mechanism of corrosion in reinforcement of concrete. What are the impacts of corrosion?
③ Explain electro-chemical process of rusting of concrete.
④ Explain the effects of permeability on concrete durability.
⑤ What is carbonation? List out the factors affecting rate of carbonation.

NUMERICALS OF
PART-I
CONCRETE TECHNOLOGY

prob① A sieve analysis was carried out for 5kg aggregates in the laboratory with available sieves. The weights retained in sieves of 40, 32, 20, 16, 10, 4.75 are 0, 1.2, 1.8, 0.5, 1.5 and 0 kg respectively. obtain Fineness Modulus for the sample.

Soln:-

IS Sieve	wt. retained (kg)	cumulative wt retained	cumulative % wt. retained.
40	0	0	0
32	1.2	1.2	24
20	1.8	3.0	60
16	0.5	3.5	70
10	1.5	5.0	100
4.75	0	5.0	100
		Total	354

Note:

$$\text{cumulative \% wt retained} = \frac{\text{cumulative wt. retained}}{5} \times 100$$

$$\text{fineness modulus (F.M)} = \frac{\text{Total cumulative \% wt. retained}}{100}$$

$$= \frac{354}{100} = 3.54 \text{ Ans}$$

prob② The oxide composition of typical portland cement is given below. Find the percentage of Bogue's compositions

$CaO - 63\%$ (C)
$SiO_2 - 20\%$ (S)
$Al_2O_3 - 6\%$ (A)
$Fe_2O_3 - 3\%$ (F)
$SO_3 - 2\%$ ($\bar{S}$)

Soln:- We have know, The Bogue's composition eqn are-

$C_3S = 4.07C - 7.6S - 6.72A - 1.43F - 2.85\bar{S}$

$C_3S = 4.07*63 - 7.6*20 - 6.72*6 - 1.43*3 - 2.85*2$
$= 54.1\%$

$C_2S = 2.87S - 0.754\,C_3S$
$= 2.87*20 - 0.754 \times 54.1$
$= 16.6\%$

$C_3A = 2.65A - 1.69F$
$= 2.65*6 - 1.69*3$
$= 10.83\%$

$C_4AF = 3.04F$
$= 3.04*3$
$= 9.12\%$

Hence the Bogue's compositions are.

$$\left.\begin{array}{l} C_3S - 54.1\% \\ C_2S - 16.6\% \\ C_3A - 10.83\% \\ C_4AF - 9.12\% \end{array}\right\} \text{Ans.}$$

prob③ In ACI method of Mix Design of concrete, it was obtained that for $1m^3$ of concrete 400 kg of cement, 200 kg of water and 100 kg of coarse aggregate is required. If entrapped air is 2 percent, what will be total weight of fine aggregates required for $1m^3$ concrete? Assume specific gravity of cement, coarse aggregate and fine aggregate as 3.15, 2.7 and 2.6 respectively. Make other necessary assumptions.

Soln:- Given,

For $1m^3$ of concrete,

wt. of cement = 400 kg
wt. of water = 200 kg
wt. of coarse aggregate = 100kg.
Air entrapped V = 2%
Sp. gravity of cement (G_c) = 3.15

Sp. gravity of coarse aggregate (ρ_{CA}) = 2.7
Sp. gravity of fine aggregate (ρ_{FA}) = 2.6

we have, from ACI method,
wt. of fresh concrete.

$$W_m = 10\rho_A(100-A) + \gamma_c\left(1-\frac{\rho_A}{\rho_c}\right) - \gamma_w(\rho_A - 1) \quad (1)$$

$$\rho_A = \frac{\rho_{CA}+\rho_{FA}}{2}$$

$$= \frac{2.7+2.6}{2} = 2.65$$

$$W_m = 10\times2.65\,(100-2) + 400\left(1-\frac{2.65}{3.15}\right) - 200\,(2.65-1)$$

$$= 2330.49 \text{ kg/m}^3$$

Then,

Total fine aggregate = W_m − all other constituents
$= 2330.49 - (400+200+1000)$
$= 730.49$ kg/m³.

Hence, total fine Aggregate required for 1 m³ concrete is 730.49 kg. Ans.

Prob (4) Design a concrete mix according to DOE method from the following data for a RCC structure.
Characteristic strength = 30 MPa
Exposure condition = severe
Type of cement = OPC
Slump required = 30-60 mm
Nominal max. size of aggregate = 20mm (crushed)
Sand = 25% passing in 600 micron (i.e IS zone I)
Sp. gravities of coarse and fine aggregate = 2.70 and 2.65 respectively.

Is that type of question asked in exam the required table and graph are given.

Soln: We follow the steps according to DOE method.

Step 1,
Target strength (f_t) = $f_{ck} + 1.64S$
$= 30 + 1.64\times5$
$= 38.2$ MPa

Step 2,
Slump = 30-60 mm
Max. size of aggregate = 20 mm (crushed)
from table (5).
Water content (w) = 210 kg/m³

Step 3,
from fig (1),
$w/c = 0.48$
Cement content (C) = $\frac{210}{0.48} = 437.5$ kg/m³.

Step 4. Check for durability.
from table (4),
Exposure condition = severe.
Max. $w/c = 0.45$
Minimum cement = 320 kg/m³.
Calculation of cement content:
$C = \frac{210}{0.45} = 467$ kg/m³ > 320 kg/m³ (OK).

Step 5,
Let relative density of combined aggregates = 2.68
from fig (2).
Wet density of concrete = 2400 kg/m³
Total aggregate content = 2400 − 467 − 210 = 1723 kg/m³

Step 6,
proportion of fine aggregate.

Grading zone I (i.e 25% passing in 600 micron) from fig. ③.

proportion of fine aggregate = 43%

fine aggregate = 43% of 1723

= 741 kg/m3

coarse aggregate = 1723 − 741

= 982 kg/m3.

Step 7.

proportion of ingradients

C : S : CA / w/c

⇒ 467 : 741 : 982 / 0.45

⇒ [1 : 1.58 : 2.10 : 0.45]

→ If surface moisture of Aggregates (fine/coarse) considered, also the moisture correction be applied.

generally,

free moisture fine aggregate = 2%

coarse aggregate = 1.5%

Correction for moisture content.

water to be deducted from

ⓐ fine aggregate = 0.02 * 741 = 14.82 kg/m3

ⓑ coarse " = 0.015 * 982 = 14.73 kg/m3

Actual quantities.

water = 210 − 14.82 − 14.73 = 180.45 kg/m3

Sand = 741 + 14.82 = 755.82 kg/m3

Coarse aggregate = 982 + 14.73 = 996.73 kg/m3

cement = 467 kg/m3

The ratio is,

[1 : 1.62 : 2.13 : 0.39] Ans

→ self do. other two designs. method m'th follow their steps.

prob ⑤ Calculate the percentage porosity, gel-space ratio and compressive strength for w/c ratio 0.55. Assume degree of hydration as 80% and 90%.

Soln: Given,

w/c ratio = 0.55

ⓐ Degree of hydration = 80%

Assume,

volume of cement (V_c) = 100 cm3

Total volume of cement paste (V) = $V_c + V_w$

= 100 + w/c * V_c * Sp. gravity of cement

= 100 + 0.55 * 100 * 3.14

V = 272.7 cm3

volume of hydrated products (V_p) = 2h cm3

= 2 × 80

= 160 cm3

volume of unhydrated cement (V_{uc}) = $\frac{100-h}{100} * V_c$

= $\frac{(100-h)}{100} * 100 = (100-h)$

= 100 − 80.00

= 20 cm3

volume of voids (V_v) = V − V_{uc} − V_p

= 272.7 − 20 − 160

= 92.7 cm3

percentage porosity (p) = $\frac{V_v}{V}$ = $\frac{92.7}{272.7} * 100$

= 33.99% ≈ 34%

Gel-space ratio (r) = (1 − 0.34) = 66% = 0.66

compressive strength = 234 r^3

= 234 $(0.66)^3$

= 67.27 MPa Ans

(b) Degree of hydration (h) = 90%
preffered from 1st case.

Total volume of cement paste (V) = 272.7 cm³

Volume of unhydrated cement (V_{uc}) = 100 − h
= 100 − 90
= 10 cm³

Volume of hydrated products (V_p) = 2h
= 2 × 90
= 180 cm³

Volume of voids (V_v) = $V - V_{uc} - V_p$
= 272.7 − 10 − 180
= 82.7 cm³

Percentage porosity (p) = $\frac{V_v}{V} \times 100$
= $\frac{82.7}{272.7} \times 100$
= 30.33%

Gel-space ratio (γ) = 1 − p
= 1 − 0.3033
= 0.697

Compressive strength = $234\,\gamma^3$
= $234 (0.697)^3$
= 79.145 MPa

Ans.

prob ⑥ Calculate the percentage porosity and gel-space ratio for w/c ratio 0.4 and 0.5. Assume degree of hydration for both cases as 80%.

Soln:- Assume,
Volume of cement (V_c) = 100 cm³

Total volume of cement paste (V) = $V_c + V_w$

$V = V_c + w/c \times V_c \times$ Sp. gravity of cement.

$V = 100 + w/c \times 100 \times 3.14$

$V = 100(1 + 3.14 \times w/c)$ ——— ①

Volume of unhydrated cement (V_{uc}) = 100 − h ——— ②

Volume of hydrated products (V_p) = 2h ——— ③

Volume of voids (V_v) = $V - V_{uc} - V_p$
= $100(1 + 3.14 \times w/c) - (100 - h) - 2h$ ——— ④

Percentage porosity (p) = $\frac{V_v}{V} \times 100$ ——— ⑤

Gel-space ratio (γ) = 1 − p ——— ⑥.

Further calculation in tabulation form, using above equations,

w/c	V (cm³)	h (%)	V_{uc}	V_p	V_v	p (%)	γ
0.4	225.6	80	20	160	45.6	20.21	0.798
0.5	257	80	20	160	77	29.96	0.700

Ans.

w/c x Vc x sp. gr

prob ⑦ Calculate the theoretical strength of a moist cured concrete containing 500 gm cement and 0.5 w/c ratio at the age of 28 days. Assume 80% hydration in 28 days.

Soln:- Given,

wt. of cement (C) = 500 gm

w/c ratio = 0.5

hydration (α) = 80% = 0.80

Now,

wt. of water (W) = 0.5×500

= 250 gm

volume of mixing water (w_o) = 250 ml

$$\text{Gel-space ratio } (r) = \frac{0.657\alpha}{0.319\alpha + \frac{w_o}{c}}$$

$$= \frac{0.657 \times 0.8}{0.319 \times 0.8 + \frac{250}{500}}$$

$\therefore$ Gel-space ratio (r) = 0.696

Compressive strength of a moist cured concrete

$= 234 r^3$

$= 234 \times (0.696)^3$

= 78.89 MPa Ans

prob ⑧ What will be the 28 days strength of a concrete having w/c ratio 0.4 as per Abram's equation. Assume K_1 = 1400 lbs/sq in ft, K_2 = 7 lbs/sq in ft.

Soln:- Given,

w/c ratio = 0.4

K_1 = 1400 lbs/sq in ft.

K_2 = 7 lbs/sq in ft.

we have, Abram's equation,

$$\text{strength of concrete} = \frac{K_1}{(K_2)^{w/c}}$$

$$= \frac{1400}{(7)^{0.4}}$$

= 642.82 MPa Ans

prob ⑨ Calculate the tensile strengths of a concrete cylinder (300mm × 150mm) and cube (150mm × 150mm) in size under standard splitting test, if load shown by the testing machine is 400kN in both cases.

Soln:- Given,

Load (P) = 400 kN

① Tensile strength under splitting test of cylinder

$$T = \frac{2P}{\pi LD} = \frac{2 \times 400 \times 1000}{\pi \times 300 \times 150}$$

$\therefore$ T = 5.66 N/mm²

$\therefore$ T = 5.66 MPa.

② Tensile strength of cube.

$$(T) = \frac{2P}{\pi a^2}$$

$$= \frac{2 \times 400 \times 1000}{\pi \times 150 \times 150}$$

= 11.32 N/mm² Ans

Prob 10) The test results of a compressive strength test is given as follows: 30, 28, 25, 27, 23, 29, 31, 30, 30, 32 (MPa). What will be the characteristic strength of the concrete? Make necessary assumption.

Soln:-

Comp. strength of tested specimen (MPa) x	x^2
30	900
28	784
25	625
27	729
23	529
29	841
31	961
30	900
30	900
32	1024
$\Sigma x = 285$, $N = 10$	$\Sigma x^2 = 8193$

Standard deviation (σ) = $\sqrt{\frac{\Sigma x^2}{N} - \left(\frac{\Sigma x}{N}\right)^2}$

$= \sqrt{\frac{8193}{10} - \left(\frac{285}{10}\right)^2} = 2.66$ MPa

Mean strength or target strength (f_t) = $\frac{\Sigma x}{N}$

$= \frac{285}{10} = 28.5$ MPa

Assume, 95% confident level.

for 95% confident level, $K = 1.64$

we have, $f_t = f_{ct} + K \cdot \sigma$

$f_{ct} = 28.5 - 1.64 * 2.66 = 24.14$ MPa

Hence, characteristic strength of concrete = 24.14 MPa Ans

Prob 11) If target strength is 20 N/mm² and standard deviation is 4 N/mm², determine the:
(a) characteristic strength
(b) 95% confidence limit.

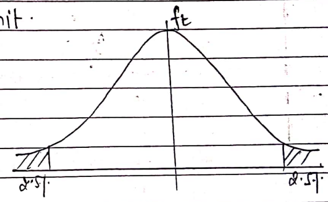
Soln: Given,
Target strength (f_t) = 20 N/mm²
Standard deviation (σ) = 4 N/mm².

(a) characteristic strength.

$f_{ck} = f_t - 1.64 * \sigma$
$= 20 - 1.64 * 4$
$= 13.44$ MPa.

Hence, characteristic strength of concrete (f_{ck}) = 13.44 MPa Ans

(b) 95% confidence limit.



→ for 2.5% = 0.025
K = 1.96

$f_{ck} = f_t \pm K\sigma$
$= 20 \pm 1.96 * 4$
$= 20 \pm 7.84$

taking (+) sign,
$f_{ck} = 27.84$ MPa.

taking (−) sign,
$f_{ck} = 12.16$ MPa.

Hence,
95% confidence interval is,
[12.16 MPa to 27.84 MPa] Ans.

prob 12 Following are the test results of 28-days compressive strength (N/mm²) of concrete for 22 sample. What is the characteristic strength of the concrete?

22, 17, 30, 18, 25, 30.5, 27, 28, 27, 20, 30, 32, 14, 26, 25, 24, 24, 32.5, 23, 15, 20, 20.

Soln:

comp. strength of tested sample (MPa) (x)	x^2
22	484
17	289
30	900
18	324
25	625
30.5	930.25
27	729
28	784
27	729
20	400
30	900
32	1024
14	196
26	676
25	625
24	576
24	576
32.5	1056.25
23	529
15	225
20	400
20	400
$\Sigma x = 530$	$\Sigma x^2 = 13377.50$

Sample no. (N) = 22

Standard deviation (σ) = $\sqrt{\frac{\Sigma x^2}{N} - \left(\frac{\Sigma x}{N}\right)^2}$

$= \sqrt{\frac{13377.5}{22} - \left(\frac{530}{22}\right)^2}$

$= 5.26$ MPa

Target strength or mean strength (f_t) = $\frac{\Sigma x}{N}$

$= \frac{530}{22}$

$= 24.1$ MPa.

Hence,

Characteristic strength of concrete (f_{ck}) = $f_t - 1.64 \times \sigma$

$= 24.1 - 1.64 \times 5.26$

$= 15.46$ MPa Ans.

prob 13 When 15 samples of cubes are tested in laboratory, following results in N/mm² are obtained. 19, 20, 18, 22, 20, 20, 17, 21, 19, 21, 18, 21, 20, 22, 19.

Then determine,

(i) characteristic strength.

(ii) 95% confidence limit.

Soln: Given,

sample no. (N) = 15

standard deviation (σ) and target strength or mean strength (f_t) is calculated the same process of above problem 11.

Hence,

Standard deviation (σ) = 1.42 N/mm²

Target strength (f_t) = 19.8 N/mm²

Then,

(i) Characteristic strength (f_{ck}) = $f_t - 1.64 \times \sigma$

$= 19.8 - 1.64 \times 1.42$

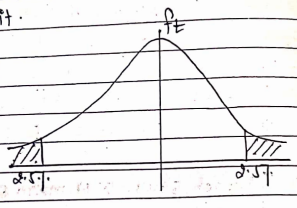
$= 17.47$ N/mm² Ans.

(ii) 95% confidence limit.

→ for 2.5% = 0.025

K = 1.96.

Then,



$f_{ck} = f_t \pm 1.96 * \sigma$

$= 19.8 \pm 1.96 * 1.42$

$= 19.8 \pm 2.78$

taking (+) sign,

$f_{ck} = 19.8 + 2.78$

$= 22.58 \ N/mm^2$

taking (−) sign,

$f_{ck} = 19.8 - 2.78$

$= 17.02 \ N/mm^2$

Hence.

95% confidence interval is.

[17.02 N/mm² to 22.58 N/mm²] Ans.

prob (14) find out the mean strength of the concrete from the following set of data:-

The characteristic strength of concrete = 18 MPa

Standard deviation (σ) = 3.6

probability factor (K) = 2.33

Air entrainment (a) = 4%.

Soln. Given,

Characteristic strength (f_{ct}) = 18 MPa

Std deviation (σ) = 3.6

probability factor (K) = 2.33

Air entrainment (a) = 4%

mean strength (f_m) = ?

we have,

$$f_m = \frac{f_{ct} + K.\sigma}{1 - 0.055 * a} \quad \text{——— (1)}$$

where,

f_m - mean strength of concrete.

f_{ct} - characteristic strength of concrete.

K - ~~permeability~~ probability factor.

σ - standard deviation.

a - air entrained.

from eqn (1).

$$f_m = \frac{18 + 2.33 * 3.6}{1 - 0.055 * 4}$$

∴ $f_m = 33.83$ MPa

Hence,

Mean strength of the concrete (f_m) = 33.83 MPa Ans.

"Best of Luck" (G.guess)

PART-II

MASONARY STRUCTURES

- Birendra Prakash Gupta

CHAPTER-7 Introduction to Masonary structures.

Masonary structures are those structures which are made built from individual units laid in and bound together by mortar. The term masonary can also refer to the units themselves. The common materials of masonary construction are brick, stone, marble, granite, travertine, limestone, cast stone, concrete block, glass block, stucco and tile. Constructing with building stones is the simplest and one of the oldest building method in the world. Today masonary is still the most used building material. The earliest masonary structures were constructed using primitive form of raw materials such as stone units, rammed earth and adobe. The structures often were built by placing the blocks together without any bonding. The evolution of masonary structures has resulted in development of not only more robust materials over the year but also more robust techenology. The masonary structures are well known for their simplicity in construction and economy compared to steel and reinforced concrete structures.

Masonary structure started being constructed using mortar which provided both better stability and better performance. The mortar varied from materials such as mud, lime and cement to mortar from lime- surkhi (brick dust). Masonary is generally a highly durable form of construction, however, the materials used, the quality of the mortar and workman ship and the pattern in which the units are assembled can significantly affect the durability of the overall masonary construction.

7.1 Use of masonary structures.

Masonary is commonly used building material for structural and non-structural purpose. Additionally, there is growing interest for masonary structures because of its 3-phase efficiency - in production phase, in construction phase and in operation phase. Masonary structures provide more ~~effi~~ comfortable living environment inside which will ultimately reduce the amount of energy spent to improve comfort condition of houses built with other material such as steel. The masonary structures are again gaining currency owing to the growing environmental concern. More over, the apealing of masonary structures for their user comfortability, aesthetic beauty and closeness to the nature have attract many for masonary building.

→ Other uses of masonary structures are in Arches, partition walls, retaining walls, dams, coffer dam (coffer dam is a temporary dam) etc.

→ Masonary are also used for finishing works in buildings and also for cladding and Roofing.

→ The Hanging garden, one of the seven wonders of the world is a typical example of masonary structure.

→ The Great wall of china, the largest man-made object on earth; the Hagia Sophia, one of the most beautiful churches ever built; The great medieval castle of Malbork, Portland, which is the size of a small town; The structure of the Taj mahal, India; and the 1200 miles of sewers which the victorians builts under the city of london are some other examples of masonary structures.

→ The oldest surviving ~~arches~~ stone masonary structure is said to be an arch bridge over Meles River at Smyrna, Turkey.

Advantages of Masonary structures

i). The use of material such as bricks and stones can increases the thermal mass of the building.

ii) Most types of masonary typically will not requires painting and so can provide a structure with reduced life-cycle costs.

iii) Masonary structures are heat resistante and thus provides good fire protection.

iv) Masonary walls are more resistant to projectiles, such as debris from hurricanes or tornadoes.

v) Masonary structures built in compression preferably with lime mortar can have a useful life of more than 500 years as compared to 30 to 100 for structures of steel or reinforced concrete.

Disadvantages of Masonary structures

i) Extreme weather causes degradation of masonary wall surfaces due to frost damage. This type of damage is ~~cornage~~ common with certain types of brick, though rare with concrete blocks.

ii). Masonary tends to be heavy and must be built upon a strong foundation, such as reinforced concrete, to avoid settling and cracking.

iii). save for concrete, masonary construction does not lend itself well to mechanization, and requires more skilled labor than stick-framing.

Structural limitations.

Masonary boasts an impressive compressive strength (vertical loads) but is much lower in tensile strength (twisting or stretching) unless reinforced. The tensile strength of masonary walls can be strengthened by thickening the wall, or by building masonary piers (vertical column or ribs) at intervals. Where practical, steel reinforcement can be added.

7.2 Construction techenology-English bond, Flemish bond, Rat-trap bond.

7.2.1 Size and weight of bricks.

The bricks are prepared in various sizes. The custom in the locality is the governing factor for deciding the size of a brick. Such bricks which are not standardised are called known as the traditional bricks.

If the bricks are large, it is difficult to ~~burn~~ them properly and they become too heavy to be placed with a single hand. On the other hand, if bricks are small, more quantity of mortar is required. Hence BIS (Bureau of Indian Standard) has recommended the bricks of uniform size. Such bricks are known as the modular bricks and the actual size of a modular brick is 190mm x 90mm x 90mm. Whith mortar thickness, size of a such bricks become 200 mm x 100mm x 100mm and it is known as nominal size of the modular brick. Thus the nominal size of brick includes the mortar thickness.

It is found that the weight of 1 m^3 of brick earth is about 1800 kg. Hence the average weight of a brick will be about 3 to 3.5kg.

NBC (Nepal building code) has recommended

the standard size of brick as ~~240x115x57 240mmx~~ 240mm x 115mm x 57mm which 10mm morter in length side.
i.e, length = 2*breadth + Thickness of mortar.

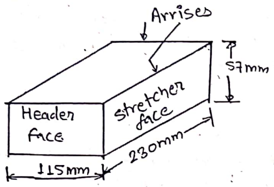


Fig. 7.1

7.2.2 Some Definitions.

1). **Stretcher:** This is a brick laid with its length parallel to the face or front or direction of wall. The course containing stretchers is called a stretcher course (see fig. 7.1).

2). **Header:** This is a brick laid with its ~~direction~~ breadth or width parallel to the face or front or direction of a wall. The course containing headers is called head course. (see. fig. 7.1 and 7.2).

3). **Arrises:** The edges formed by the intersection of plane surfaces of brick are called the arrises and they should be sharp, square and free from damage (see fig. 7.1).

4). **Bed:** The lower surface of the brick when laid flat is known as the bed.

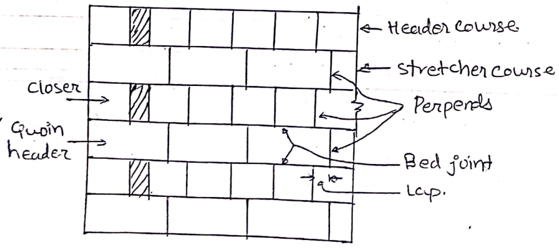


Fig. 7.2 Definitions

5). **Bed joint:** The horizontal layer of mortar upon which the bricks are laid is known as a bed joint. see fig. 7.2.

6). **Perpends:** The vertical joints separating the bricks in either length or cross directions are known as the perpends and for a good bond, the perpends in alternate courses should vertically one above the other. see fig. 7.2.

7). **Lap:** The horizontal distance between the vertical joints in successive courses is termed as a lap and for a good bond, it should be one fourth of the length of a brick. see fig. 7.2.

8). **Closer:** A piece of brick which is used to close up the bond at the end of brick courses is known as a closer and it helps in preventing the joints of successive courses to come in vertical line. Generally the closer is not specially moulded. But it is prepared by the mason with the edge of the trowel.

9). **Frog:** A frog is a mark of depth about 10mm to 20mm which is placed on the face of a brick to form a key for holding the mortar. The wire cut bricks are not provided with frogs. A pressed brick as a rule has frogs on both the faces. A hand made brick has only one ~~[illegible]~~ frog.

7.2.3 Bonds in Brickwork

The bricks being uniform size can be arranged conveniently in a variety of forms. The various types of bonds with their patented names have been constructed. some of the bonds in brickwork are listed below.

i). Stretcher bond.	vii). Dutch bond
ii). Header bond	viii) Brick-on-edge bond
iii). English bond	ix) English cross bond
iv) Flemish bond	x). facing bond.
v) Garden-wall bond	xi) Rat-trap Bond
vi) Raking bond	

Bonding means the arrangements of bricks in such a way that no vertical joint of one course is exactly over the one below. This means that the brick is laid in such a way that it overlaps and breaks the joint below. An un-bonded wall, with its continuous vertical joints has little strength and stability and such joints must be avoided.

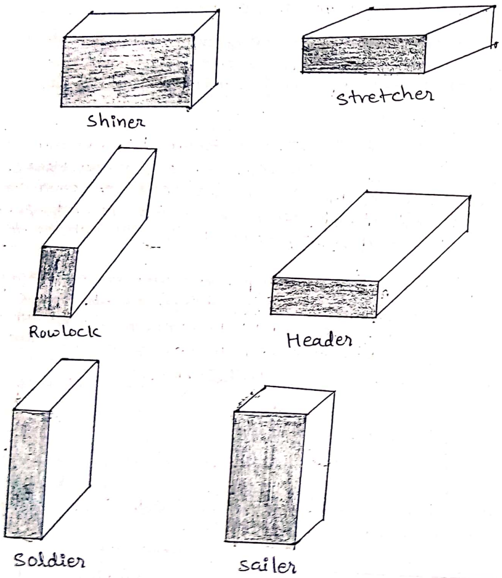


Fig.7.3 Position of Bricks in Brick wall construction.

7.2.3.1 English bond.

This type of bond is generally used in practice. It is considered as the strongest bond in brick work.

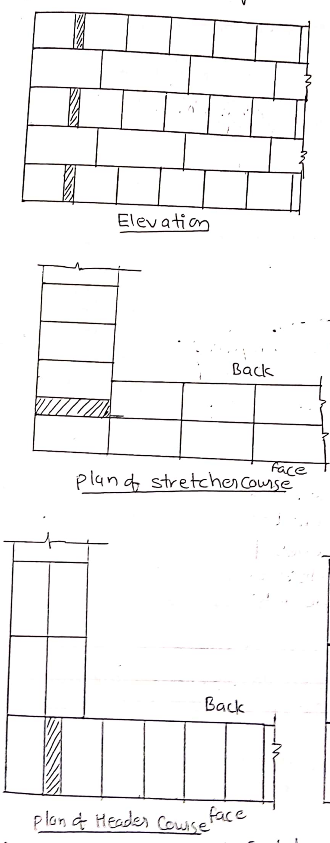


Fig.7.4 1-brick wall with English bond.

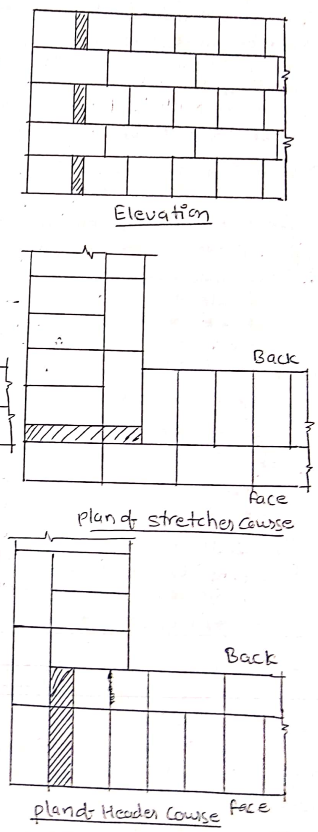


Fig.7.5 $1\frac{1}{2}$ bricks wall with English bond.

Following are the features of an English bond.

i). The alternate courses consist of stretchers and headers.

ii). The queen closer is put next to the quoin header to develop the face lap.

iii). Each alternate header is centrally supported over a stretcher.

iv). If the wall thickness is an even multiple of half-brick, the same course shows headers or stretchers in both the front and the back elevations. But if the wall thickness is an uneven multiple of half brick, a course showing stretcher on the face shows header on the back and viceversa.

v). The bricks in the same course do not break joints with eachother. The joints are straight

vi). In this bond, the continuous vertical joints are not formed except at certain stopped ends.

vii). The number of mortar joints in the header course is nearly double than that in the stretcher course. Hence care should be taken to make the header joints thinner; otherwise the face lap disappears quickly.

viii). A Header course should never start with a queen closer as it is liable to get displaced in this position.

ix). The queen closer are not required in the stretcher courses.

x). In the stretcher course, the stretchers have a minimum lap of one fourth of their length over the headers.

xi). For walls having thickness of two bricks or more, the bricks are laid as stretchers or headers only on the face courses of the wall. The interior filling is done entirely with the headers.

Fig. 7.4 shows the plans of alternate courses and elevation of brick wall meeting at the corner with thickness of each wall as one brick.

Fig. 7.5 shows the plans of alternate courses and elevation of brick wall meeting at the corner with thickness of each wall as $1\frac{1}{2}$ bricks.

7.2.3.2 Flemish bond.

In this type of bond, the headers are distributed evenly and hence, it creates a better appearance than the English bond. Following are the features of Flemish bond.

i). In every course, the headers and stretchers are placed alternatively.

ii) The queen closer is put next to the quoin header in alternate courses to develop the face lap.

iii) Every header is centrallay supported over a stretcher below it.

iv) The bricks in the same course do not break joints with each other. The joints are straight.

v). In this bond, the short continuous vertical joints are formed.

vi). The brickbats are to be used for walls having a thickness equal to uneven number of half brick.

Flemish bond may be divided into two type.

a). Double Flemish bond. b). Single Flemish bond

a). Double Flemish bond,

In double Flemish bond (see fig. 7.6 and fig 7.7), the headers and stretchers are placed alternatively in front as well as the back elevations.

For this type of bond, the half bats and three-quarter bats will have to be used for walls having thickness equal to odd number of half bricks. For walls of thickness equal to even number of half bricks, no bat will be required and a stretcher or a header will come out as a stretcher or a header in the same course in front as well as back elevations. This bond gives better appearance than the English bond. But it is not so strong as the English bond as it contains more number of stretcher.

b) Single Flemish bond

In Single Flemish bond (see fig 7.8 and fig 7.9), the face elevation is of Flemish bond and the filling as well as backing are of the English bond. Thus in this type of bond, an attempt is made to combine the strength of the English bond with the appearance of the flemish bond. This type of bond is used when expensive bricks are used for the face work (facing). But inorder to construct this bond, a wall of minimum thickness $1\frac{1}{2}$ bricks is required.

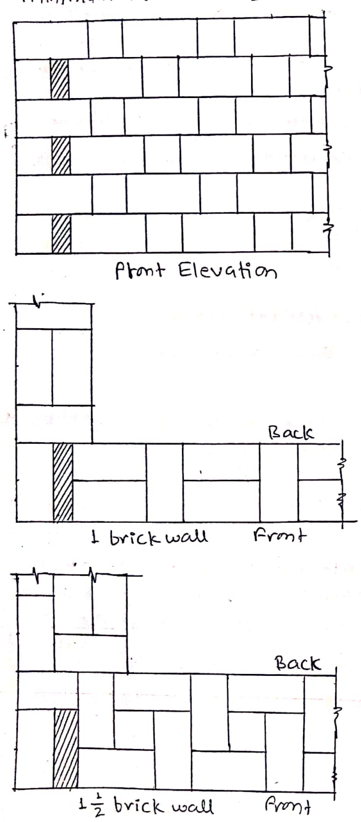


Fig. 7.6 Courses 1,3,5, etc of Double Flemish bond.

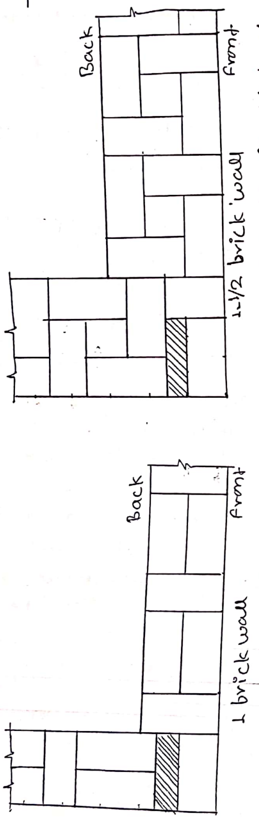


Fig. 7.7 2,4,6 etc Courses of Double Flemish bond.

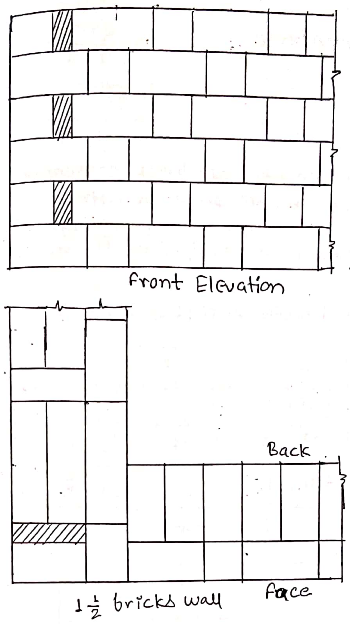


Fig. 7.8 Courses 1,3,5 etc of single Flemish bond.

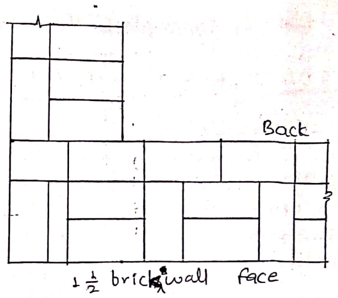


Fig. 7.9 Courses 2,4,6 etc of single Flemish bond.

The comparison of English bond and Flemish bond can be made with respect to following Aspects.

i). The English bond is found to possess more strength than the Flemish bond. for walls having thickness greater than $1\frac{1}{2}$ bricks

ii) The Flemish bond grands more pleasing appearance than the English bond.

iii). It is possible to make use of broken bricks in the form of brickbats in case of the Flemish bond. However more mortar will be required.

iv) The construction with the Flemish bond requires greater skill as compared to the English bond.

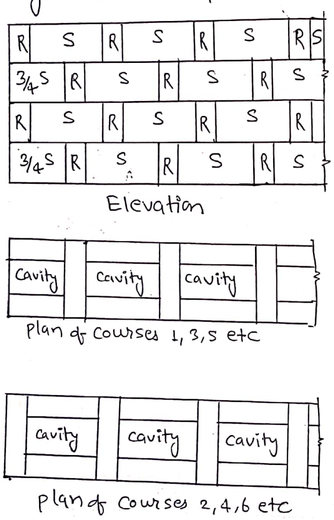
7.2.3.3 Rat Trap Bond.

Introduction

A "Rat Trap Bond" is a type of wall brick masonary bond in which bricks are laid on edge such that the shinner and Row lock are visible on the face of masonary. This gives the wall with an internal cavity bridged by the row lock.

There are two possible layout of Rat Trap bond.

a). Each alternate course begins with a three-quarter (3/4 S), followed by a Rowlock, the intermediate course begins with a Rowlock (R) followed by a shiner (S). see fig. 7.10



b). Each alternate course begin with a shiner (S), followed by a Rowlock (R). The intermediate course begins with two Rowlocks followed by a shiner. see fig. 7.11.

For walls without corners such as in the case of non-load bearing walls between column in framed structure, both methods are correct and can be applied according to the preferences of the masons. However for walls with corners method b must be applied.

Fig. 7.10 Rat Trap Bond type-a.

History.

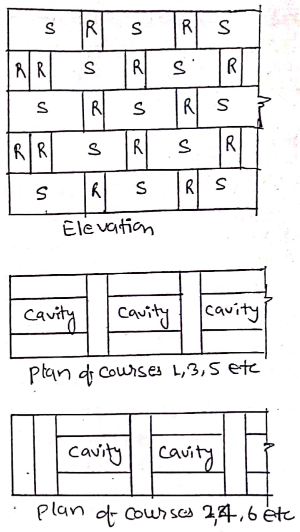


Fig. 7.11 Rat Trap Bond type-b

Rat trap bond was commonly used in England for building houses of fewer than 3-stories up to the turn of the 20th century. However, the brick industry successfully opposed the Rat Trap bond declaring it a non load bearing wall bond and promoted the traditional English and Flemish bond that uses approximately 35% more bricks. The Rat Trap bond then entirely disappeared, from the construction sector for decades. Influenced by Mahatma Gandhi, Mr. Laurie Baker, a British architect started the trend of cost efficient housing in india by re-introducing the Rat Trap Bond. Today the Rat Trap Bond is widely used in India and is proven economical walling bond with good insulation properties due to the air cavities. The origin of the peculiar Rat Trap Bond name is not known, but is probably connected to the trap formed inside the wall due to the bric arrangement.

Applicability.

The Rat trap Bond is a variation of normal walling masonary bonds. Therefore the Rat Trap Bond can be used for all 9" (approx. 23cm) thick walls. The Rat Trap Bond uses a high proportion of shiners and hence requires less facing bricks than normal bonds. This makes the Rat Trap Bond somehow less sturdy (strong) and therefore, it is recommended to use the Rat Trap Bond only for load bearing walling up to 3-stories. If the Rat Trap

Bond is used as filler wall between concrete pillars, then there is no limitation of heights. However, the main used of applying Rat Trap Bond is where cost and energy saving is an issue.

Advantages.

1). The main ~~objective~~ advantage of the Rat Trap Bond is the economic use of bricks. Using this bond, a wall of one brick thickness (9") can be constructed with fewer bricks as compared to a solid wall in English or flemish bond.

- By adopting this method of bonding, it is possible to use approx 35% less bricks and 50% less cement mortar. This reduces the cost of a 9" wall by 30%. (based on brick dimensions 230 x 110 x 55 mm).
- In kathmandu, 550 bricks are required per m^3 of ~~Elem~~ English Bond masonary. for RTB masonary, 360 bricks per m^3 are used.

2). It increases thermal comfort due to the presence of cavity. The interiors remains cooler in summer and warmer in winters.

3). The walls have approximately 20% less dead weight and hence the foundations depending on the bearing capacity of the soil, can suitably be redesigned to save bricks, steel and cement.

4). I.OE. Pulchowk campus Lab test report confirms the load bearing capacity of RTB for H_2 mortar is 10.52 kg/cm²

5). By adopting RTB, one can create aesthetically pleasing wall surface and plastering can be avoided.

6). Vertical wiring and plumbing can easily be made during the wall construction and even after since cavities allow inserting the fittings. for horizontal installations of fittings, planning must be made before constructing walls.

7). Rat Trap Bond is a modular masonary system which can reduce wastage of bricks by un-necessary cutting.

Disadvantages

The main drawback of the Rat Trap Bond is that fired clay bricks are used. The firing of bricks is in general highly energy consuming and air polluting. However, these negative points can be reduced if:

- RTB is constructed with bricks that are fired is VSBK (Vertical shaft brick klin). which uses 40-50% less energy than the traditional brick firing techenologies of Nepal.
- RTB is constructed in concrete blocks which uses less energy than traditionally burnt bricks.

7.3 Hollow block and Compressed earth block

7.3.1 Hollow Blocks.

Howllow blocks are precast concrete units made of appropriate mixture of cement and aggregates such as sand, river bed gravel, crushed stone etc. It can be produced in different shape and sizes for wall construction to fit different construction needs and designs. Most of them are made in full and half length units as per modular design. These Howllow Blocks are prepared by concrete block Techenology.

Concrete Block Technology.

It is based on the principle of densification of a lean concrete mix to make a regular shaped, uniform, high performance masonary units. Concrete block Techenology can be easily adopted to suit special needs of users by modifying design parameters such as mix proportion, water / cement ratio and type of production system. It is an effective means of utilizing wastes generated by stone crushers, quarrying and stone processing units. The techenology has high performance in areas where raw materials are easily available.

Advantages of Hollow concrete Blocks.

1). Reduces cost investment at least 30% compared to fired clay brick masonary.
2). Reduces more than 50% energy (MJ/m²) compared to fired clay brick masonary.
3). Easy and speedy construction
4). Have good thermal and sound insulation.
5). It can be produced in different shapes and sizes to fit different construction needs and designs.
6). National and International standards are available and hollow concrete block technology is accepted by the building code.
7). Reduces dead load
8). Reduces maintenance cost as there is No salt peter or leaching.
9). Reduces the thickness of plaster due to size accuracy & less cement consumption due to fewer joints
10). makes environment friendly, as fly ash used as one of the raw materials.
11). These are durable and maintenance free.
12). It act as damp-proof as it has low water absorption

Technical Specifications of Concrete Blocks

i) Typical size = 300 x 200 x 150 mm
ii) Average compressive strength at 28 days : 50-110 kg/cm²
iii) Mix proportion : 1:12-14 (1 part cement : 12-14 parts sum graded aggregates)
iv) Water absorption in 24 hours : < 10% by weight of Block.

7.3.2 Compressed Earth Blocks.

The Compressed Earth Blocks are masonary unit of cuboidal shape and are manufactured by compacting raw material earth mixed with a stabilizer such as cement or lime under a pressure of 20-40 kg/cm² using manual soil press such as Balram. It is also termed as Stabilized compressed Earth Block. A number of manual and hydraulic machines are available in India for the manufacture of CEB. The basic principle of all the machines is the compaction of raw earth to attain dense, even sized masonary. Some of the hydraulic machines can even manufacture interlocking blocks. These interlocking blocks are highly suitable for speedy and mortar less construction.

Advantages of Compressed Earth Blocks.

1). A local material.
↳ The production is made on the site itself or in the nearby areas. Thus it will save the transportation, fuel, time and money
2). An adapted material.
↳ Being produced locally it is easily adapted to the various needs: technical, social, cultural habits.
3). A job creation opportunity.
↳ CEB allow unskilled and unemployed people to learn a skill, get a job and rise in the social value
4). Limiting Deforestation.
↳ Firewood is not needed to produced CEB, so it will save the forests.
5). It has high strength and durability and needs minimum maintenance.
6). A transferable technology.
↳ It is a simple technology requiring semi skills, easy to get. Simple villagers will be able to learn how to prepare it in a few weeks. Efficient training centre will transfer the techenology in a week time.

7). Market opportunity.
↳ According to local context (material, Labour, equipment etc) the final price will vary, but in most of the cases it will be cheaper than fired bricks.

8). Reducing Imports.
↳ As it is locally produced, there is no need to import heavy and expensive building materials from ~~far away~~ away.

9). Energy efficiency and ecofriendliness.
↳ It requires less energy than fired brick
↳ The pollution emission will be less than that of fired bricks.

10). Fire resistant, provides thermal and sound insulation.

Disadvantages of Compressed Earth Block

1). Proper soil identification is required or lack of soil.
2). Ignorance of the basics of production and use.
3). Low technical performances compared to concrete.
4). Untrained teams producing bad quality products.
5). Low social acceptance due to counter examples. (By unskilled people, or bad soil & equipment).
6). Over-stabilization through fear or ignorance, implying outrageous costs.

7.4. Masonary as infill walls.

Infill walls are the walls which are confined on all four sides with reinforced concrete or reinforced masonary as vertical and horizontal confining element. which are not intended to carry either vertical or horizontal loads and are consequently not designed to perform as moment-resisting frames. Masonary is usually used as infill wall, and is also known as confined masonary. In the case of masonary-infilled frames, the R.C (Reinforced concrete) frame structure, which is designed to resist vertical and seismic loads without infill, is constructed first. masonary filler walls are very often constructed as non structural elements after the

completion of the main R.C structure. In the case of confined masonary, however, masonary walls are intended to carry all vertical and seismic loading. The structural walls, which support the floors, are constructed first. Then the floors with horizontal band-beams elements are put in place, and finally R.C ~~confini~~ vertical confining elements are constructed, well connected with horizontal confining elements.

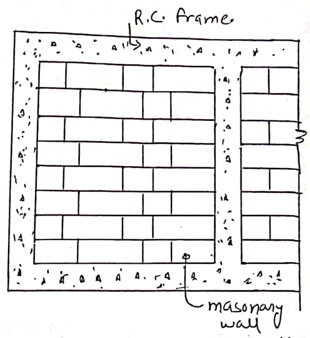


Fig. 7.12 masonary as infill wall.

As the experimental investigations and the experiences obtained after earthquakes have shown, confining the masonary walls with bond-beam and tie column results in

- Improvement in connection between structural walls
- Improvement in the stability of slender structural walls
- Improvement in strength and ductility of masonary panels.
- Reduction in the risk of disintegration of masonary panels damaged by the earthquake

In order to ensure structural integrity, vertical confining elements should be located at all corners and recess of the building and at all joints and wall intersections. In addition, they should be placed at both sides of any wall opening. Vertical confining elements should also be placed at all free ends of structural walls.

7.5 Reinforced and un-reinforced masonary.

7.5.1 Reinforced Masonary.

Reinforced masonary is a construction system, where steel reinforcement in the form of reinforcing bars or mesh is embedded in the mortar or placed in the holes and filled with concrete or grout. Reinforcement increases the tensile as well as compressive strength of wall. before Reinforcement wall has very less tensile strength. By reinforcing the masonary with steel reinforcement, the resistance to seismic loads & energy dissipation capacity may be improved significantly. To achieve this, the reinforcement should be integrated with masonary so that all materials of reinforced masonary system acts monolithically when resisting gravity and seismic loading.

There are various ways in which steel reinforcement can be used in Reinforced masonary structural system. Basically, reinforced masonary system can be classified in to:

i). Reinforced hollow unit masonary.
ii). Reinforced grouted cavity masonary and
iii). Reinforced poket type walls.

i). Reinforced Hollow unit masonary

Reinforced hollow unit masonary represents the basic form of reinforced masonary construction. special shaped units with vertical holes where vertical reinforcement is placed and filled with infill concrete or grout, with or without grooves to accomodate horizontal, bed joint reinforcement, are used for the construction of masonary walls. Before laying the masonary units, vertical reinforcement is placed in position. Then the first course of units is laid in the mortar and horizontal bars or bed joint reinforcement are placed in the grooves or in the mortar joints.

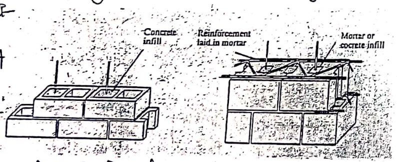


Fig 7.13 Reinforced hollow unit masonary.

The holes containing vertical bars are filled with either concrete or grout, and the grooves containing the horizontal steels are filled with either grout or mortar, as the construction of the wall progresses. In order to improve the resistance and depending on the shape of the units, all holes in the hollow blocks are often grouted or

filled with concrete infill.

ii). Reinforced cavity masonary

Reinforced cavity masonary is different by technology of construction, and consequently, by structural characteristics and behaviour. as seen in fig 7.14, it consists of two leaves (wythes) of masonary units, separated by a cavity into which the vertical and horizontal reinforcement is placed and grouted with either concrete infill or grout. The two leaves of a cavity wall are tied together with either wall ties or connectors, which should be designed to carry lateral loads, induced by earthquake.

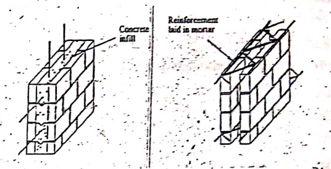


Fig 7.14 Reinforced grouted cavity masonary

The masonary units should be laid in running or stretcher bond: vertical stacked bond is not allowed in seismic zones. The grout can be pocured either as the work progresses or after the masonary units in the whole storey have been laid. In the first case, vertical reinforcing bars are placed first into position. Then the horizontal bars and wall ties are placed and grouted as laying of courses of masonary progresses. In the second case, the mesh of vertical and horizontal is placed first in position. Then, masonary units are laid on each side of the mesh, connected together with wall ties. The ties should be laid in the bed joints along the same vertical line in order to facilitate the vibrating of the grout pours. After the masonary is built to a full storey height, the cavity is filled with grout. Before grouting, all mortar droppings should be removed from the foundation or other bearing surfaces and reinforcement. Cleanout openings should be provided to allow flushing away of mortar droppings and debris at the bottom of each pour.

iii). Reinforced Pocket type walls.

Sometimes, vertical reinforcement is placed in the pockets formed in the wall by spacial bonding arraangement. As in the case of reinforced hollow unit masonary units, depending

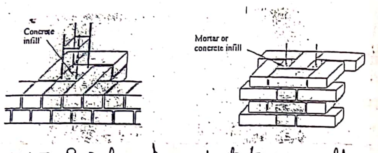


Fig 7.15 Reinforced pocket type walls.

vertical reinforcing bars are placed into position before the laying of masonary units. Dependending on the units used, horizontal bed joint reinforcement is placed in the mortar joints at vertical spacing not exceeding 600 mm. The pockets containing vertical bars are filled with either concrete or grout, as the construction of the wall progresses.

7.5.2 Unreinforced Masonary.

An Unreinforced masonary is a construction system where load bearing walls, non-load bearing walls or other structures, such as dam, Retaining walls etc are made of brick, cinderblock, tiles, adobe or other masonary units that is not braced by reinforced bars or beams. This is the term used in Earthquake engineering as a classification of certain structures for earthquake safety purposes and is subjected to minor variation from place to place.

Unreinforced masonary structures are vulnerable to collapse in earthquake. One problem is that most mortar used to hold bricks together is not strong enough. Additionally, masonary elements may 'peel' from the building, and fall on to occupants outside.

CHAPTER-8 Design of masonary walls for gravity L.

8.1 Introduction to codal provisions.

The most effective use of masonary construction is seen in load bearing structures wherein it performs a variety of functions, namely supporting loads, subdividing space, providing thermal and acousting insulation as well as fire resistance and weather protection which normally in a framed building has to be accounted for separately. Until 1950's there was no engineering methods of designing masonary for buildings and thickness of walls was based on Thumb-rule. As a result walls used to be very thick and masonary structures were found to be very uneconomical beyond 3 or 4 stories. Since 1950's intensive theoritical and experimental research has been conducted on various aspects of masonary in advanced countries. Factors affecting strength, stability and performance of masonary structures have been identified, which need to be considered in design. And code of practice was developed in different countries which specifies all necessary specification and criteria for masonary design. There are various design codes for masonary design.

i) Building code Requirement for masonary structure (ACI 530-02/ASCE 5-02/TMS 402-02).

ii) Internation Building code 2000

iii) New Zealand standard - Code of Practice for the design of concrete and masonary structures (NZS 4230: Part 1: 1990)

iv) Eurocode 6: Design of Masonary structures (DD ENV 1996-1-1: 1996)

v) Indian Standard - Code of practice for structural Used unreinforced masonary (IS: 1905-1987)

vi) Nepal Building code: NBC 109

We are mainly concerned with IS 1905-1987 and NBC 109.

8.2 Design example for gravity loads.

8.2.1 Solid wall

Design steps

i). Assume the thickness of wall (eg. 100mm), mortar Ratio (eg. 1:5 (M1)) and crushing strength (eg. 10 MPa)

ii). Calculate total load on the wall.

a) weight of slab = ------ kN/m

b) weight of surface finish = ----- kN/m

c) Live load = ----- kN/m

d) self weight of wall = ---- kN/m

Total load = (a+b+c+d) kN/m

iii) Find total stress = $\frac{\text{Total load}}{\text{thickness of wall}}$ (kN/m²)

= ---- (N/mm²)

iv). Find permissible stress

$f_{ca} = f_b * k_a * k_s * k_p$

where,

f_b = basic compressive stress

it is function of mortar type and crushing strength and is given in table 8 (IS 1905-1987)

k_a = Area Reduction factor

as per clause 5.4.1.2 (IS 1905-1987)

k_s = stress reduction factor

as per clause 5.4.1.1

k_s = f (slenderness ratio, eccentricity)

and is given in table 9

k_p = shape modification factor

as per clause 5.4.1.3 and is given in table 10.

v). check.

if total stress < f_{ca}

Design is o.k.

Else, Redesign the wall using ~~and~~ another wall thickness and Repeating all above procedure. Mortar type and crushing strength can also be changed.

Example-1

Design the wall of two story building to carry 120mm thick RCC slab with 3m ceilling height. The wall is unstiffened and support a 2.5m wide slab on both sides

Live loads on roof = 1.5 kN/m²

Live loads on floor = 2.0 kN/m²

weight of surface finish = 1.2 kN/m²

Soln

Step-1: Assume, the thickness of wall = 110 mm

mortar ratio = 1:5 (M1)

and crushing strength = 10 MPa

Step-2 Load calculation,

Load from Roof

a) ~~self~~ wt. of slab = unit wt. of RCC * thickness of slab * width of slab

= 25 kN/m³ * 0.12m * 2.5m

= 7.5 kN/m

b) Weight of Surface finish = unit wt. of Surface finish * width of slab

= 1.2 kN/m² * 2.5m

= 3 kN/m

c) Live load = 1.5 kN/m² * width of slab

= 1.5 * 2.5m

= 3.75 kN/m

Total = 1-127-kN/m

Load from 1st floor

a) Wt. of 120mm slab = 7.5 KN/m²

b) Wt. of Surface finish = 3 KN/m

c) Live Load = 2 × 2.5
= 5 KN/m

Total = 15.5 KN/m

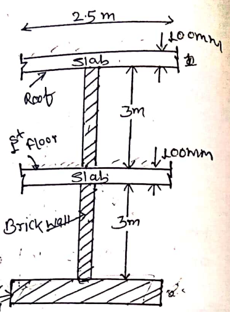


Fig. 8.1

Self wt. of the wall = Unit wt. of the wall * height of the wall * thickness of wall * No. storey

= 20 KN/m³ * 3m * 0.11m * 2

= 13.2 KN/m

Total load = (14.25 + 15.5 + 13.2) KN/m

= 42.95 KN/m

Step 3.

Total stress = $\frac{\text{Total load}}{\text{Thickness of wall}}$

= $\frac{42.95 \text{ KN/m}}{0.11 \text{ m}}$

= 390.45 KN/m²

= 0.39045 N/mm²

Step 4.

Permissible stress (f_{ca}) = $f_b * K_a * K_s * K_p$

where,

f_b = 0.96 N/mm² (from table 8)

Area Reduction factor (K_a)

Area of wall = 0.11 * 2.5
= 0.275 m² (> 0.2 m²)

So, K_a = 1 as per clause 5.4.1.2

Stress reduction factor (k_s)

e = 0 (eccentricity)

Slenderness Ratio (SR) = $\frac{\text{Effective height}}{\text{Effective thickness}}$

Effective height = 0.75H as per clause 4.3.1
= 0.75 * 3000mm
= 2250 mm

Effective thickness = 110mm as per clause 4.5.1

∴ SR = $\frac{2250}{110}$ = 20.45 < 27 O.K
(see clause 4.6.1)

from Table 9, By interpolation,
for e = 0 & SR = 20.45

k_s = 0.6065

Shape modification factor (K_p)

$\frac{\text{Height of brick unit}}{\text{width of brick unit}}$ = $\frac{\cancel{110}\ 55}{\cancel{55}\ 110}$ = 0.5

from table 10
K_p = 1.

∴ f_{ca} = 0.96 * 1 * 0.6065 * 1
= 0.582 N/mm²

Step 6: check

f_{ca} > total stress
So. design is o'k

Hence provide,
Thickness of wall = 110mm
mortar type = M1
Crushing strength = 10 MPa

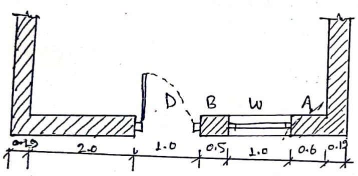
8.2.2 walls with openings

it also follows the same design steps as explained in article 8.2.1.

Example-2

External wall of a single storeyed house is 20cm thick, and has door and window openings as shown in Fig 8.2. Plinth level is 1.2m above the top of foundation footing and floor to ceiling height is 2.8m one way RCC slab of 3m clear span bears on the wall and is 10cm thick. Determine the max^m ~~thickness of wall~~ stress in the wall and calculate strength of bricks and grade of mortar required for the wall. There is a 20cm thick parapet wall of 0.8m height above the roof slab. wall and parapet are plastered on both sides.

Live load = 1.5 kN/m^2



Lintel level = 2.0 m
sill level of window = 0.5m
All dimension are in m.
Fig. 8.2

Sol^n

As thickness of wall is given, we starts from load calculation

Load calculation

Thickness of wall = 19 cm (as 20cm include mortar thickness of 10 mm) known as modular size.

Thickness of wall with both side plaster
= 19+3 = 22 cm.

Load from parapet = Thickness of wall * height of wall * unit wt
= 0.22m * 0.8m * 20 kN/m^3
= 3.52 kN/m

Load from Roof

wt. of 10cm RCC slab = 0.1 * 25 kN/m^3
= 2.5 kN/m^2

Assuming 1.5cm plaster on both side

wt. of plaster = 0.015 * 2 * 20 kN/m^3
= 0.6 kN/m^2

Total = 3.1 kN/m^2

clear span of slab = 3 m
∴ effective span = 3 + 0.1
= 3.1 m (0.1 m bearing width)

Roof load on wall = $\frac{3.1 \text{ kN/m}^2 * 3.1 \text{ m}}{2}$
= 4.81 kN/m (as Roof load distributes on two walls; slab being on one way slab).

self weight of wall up to plint level.
= 0.22 * 2.8 * 20
= 12.32 kN/m

Portion A of wall.

Length of wall = 0.6 + $\frac{0.19}{2}$
= 0.69 cm (up to centre of cross wall)

load on wall = load from parapet + Roof load + self wt
= (3.52 + 4.81 + 12.32)
= 20.65 kN/m * (0.69 + $\frac{1.0}{2}$)
= 24.57 kN

since ~~up to half the length of window~~ loads from the half window length, bears wall A.

Stress Reduction factor (k_s)

Effective ht. of column for the direction perpendicular to the plane of the wall

(Clause 4.3.3) $h_{eff} = 0.75H + 0.25H_1$

$= 0.75\,(1.2+2.8+0.05) + 0.25*2$

$= 3.04+0.5$

$= 3.54\,m.$

Effective height of column for direction parallel to the wall, $h_{eff} = H = 1.2+2.8+0.5$

$= 4.05\,m.$

SR perpendicular to plane of wall $= \frac{3.54}{0.17}$

$= 21.0$

SR, parallel to plane of wall $= \frac{4.05}{0.5}$

$= 8.1$

Thus SR = 21 will govern the design (Clause 4.6.2)

$k_s = 0.59$ from Table 9 for SR=21 & e=0

Area reduction factor (k_a)

$A = 0.5*0.17 = 0.085\,m^2 < 0.2\,m^2$

$\therefore\ k_a = 0.7 + 1.5*0.085$ (clause 5.4.1.2)

$= 0.83$

Shape modification factor (k_p)

For crushing strength 10 MPa and $\frac{h}{t}$ of unit $= 1$

$k_p = 1.1$

$\therefore$ basic compressive stress (f_b) $= \frac{f_a}{k_a.k_s.k_p}$

$f_b = \frac{0.364}{0.83*0.59*1.1}$

$= 0.68\,N/mm^2 > f_b$ for wall portion A

Thus from table 8, M3 mortar can be used with basic compressive stress $0.75\,N/mm^2$, as wall portion B govern the design.

Hence, provide

crushing strength = 10MPa

grade of mortar = M3

8.2.3 Walls with eccentric loading

Calculation of stress for eccentric loading

Consider a masonry section bxd as shown in fig 8.3 subjected to axial load 'W' at point 'P'

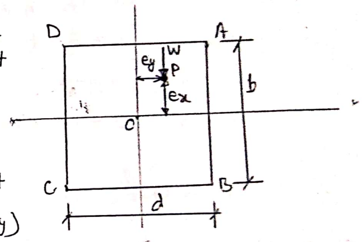


Fig. 8.3

The load equivalent at P

= load W at point O + moment ($W*e_x$) + moment ($W*e_y$)

Area, $A = bd$

$I_{xx} = \frac{db^3}{12}$

$I_{yy} = \frac{bd^3}{12}$

Let F(x,y) be any point on masonry section.

The stress due to load $W = \frac{W}{A} = \frac{W}{bd}$

stress due to moment $W.e_x = \frac{M}{I_{xx}} * y$

$= \frac{W.e_x}{\frac{db^3}{12}} * y$

Since wall is plastered on both sides, it may be assumed to have raked joints on both sides

Thus effective thickness of wall = 19 − 2
= 17 cm ✓

(see clause 5.5.1.1)

$$\text{compressive stress at plinth level} = \frac{\text{load on wall}}{A}$$

$$= \frac{24.57\ kN}{0.17 * 0.69}$$

$$= 209.46\ kN/m^2$$

$$\doteq 0.2095\ N/mm^2$$

Slenderness Ratio

from consideration of height

$$SR = \frac{\text{effective height}}{\text{effective thickness}}$$

$$= \frac{0.75(1.2 + 2.8 + 0.05)}{0.17}$$

$$= 17.87$$

as length of wall A (69 cm) ≥ 4t (4*19 = 76 cm)

So it is not a column. (Clause 2.3.1)

Effective length = 2L (From clause 4.4) (Table-5)

= 2*0.69

= 1.38 m

$$\therefore SR = \frac{1.38}{0.17} = 8.12$$

since, SR, lengthwise < SR, heightwise

SR length govern the design

$$= \frac{12\,W.e_x}{db^3} . y$$

stress due to moment $W.e_y = \frac{12\,W.e_y}{bd^3} . x$

Hence, Total shear

$$\sigma = \frac{W}{bd} + \frac{12\,W.e_x}{db^3} . y + \frac{12\,We_y}{bd^3} . x$$

$$\boxed{\sigma = \frac{W}{bd}\left[1 + \frac{12\,e_x}{b^2} . y + \frac{12\,e_y}{d^2} . x\right]} \quad \text{—(8.1)}$$

For point A, $x = d/2$, $y = \frac{b}{2}$

" " B, $x = \frac{d}{2}$, $y = -\frac{b}{2}$

" " C, $x = -\frac{d}{2}$, $y = -\frac{b}{2}$

" " D, $x = -d/2$, $y = b/2$

∴ minimum stress will be at point C

$$\sigma_c = \sigma_{min} = \frac{W}{bd}\left[1 - \frac{6e_x}{b} - \frac{6.e_y}{d}\right] \quad \text{—(8.2)}$$

max^m stress will be at point A

$$\text{i.e } \sigma_A = \sigma_{max} = \frac{W}{bd}\left[1 + \frac{6\,e_x}{b} + \frac{6\,e_y}{d}\right] \quad \text{—(8.3)}$$

Middle Third Rule

If e_x and e_y are large, the stress at point C becomes negative (i.e tensile). So it is required to keep e_x and e_y limited for stresses to be compressive always.

When, σ_c just becomes zero.

i.e. $\sigma_c = 0$

$$\frac{W}{bd} . \left[1 - \frac{6\,e_x}{b} - \frac{6\,e_y}{d}\right] = 0$$

or $1 - \frac{6e_x}{b} - \frac{6e_y}{d} = 0$

if $e_x = 0 \Rightarrow e_y = \frac{d}{6}$

if $e_y = 0 \Rightarrow e_x = \frac{b}{6}$

Hence the stress at 'c' does not becomes tensile so long as the load remains in the shaded area.

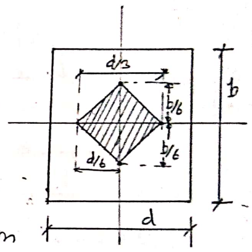


Fig. 8.4.

Definition:

"For no tension on the section of wall the resultant thrust should be within the middle third of the axis."

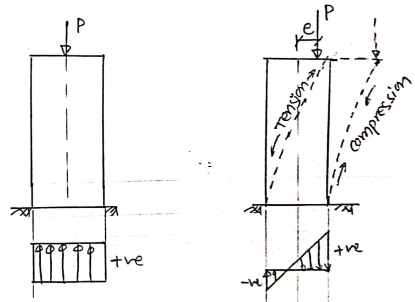


Fig 8.5 stress distribution in axial and eccentric loading

#

In some cases cracking of masonary structures may be allowed on the tension face provided that the compressive stress is within the safe limit. In such cases the effective contact width of the section is reduced. The maximum compressive stress may be determined as follows.

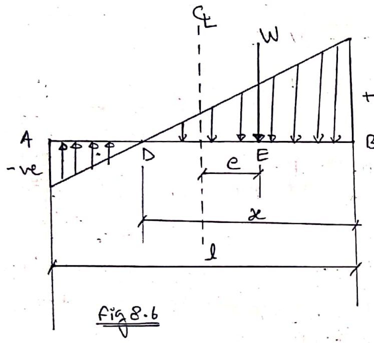


Fig 8.6

Since the tension is induced at A, it will crack and reduced width x will remain in contact.

Since Resultant weight 'W' should lie on the outer third point so that point D should carry zero stress (middle third Rule).

$\therefore$ $BE = x/3$

or $x = 3BE = 3(l/2 - e)$

$$\boxed{x = 3\left(\frac{l}{2} - e\right)} \quad \text{—— (8.4)}$$

So the max^m pressure,

$$\boxed{\sigma_{max} = \frac{2W}{bx}} \quad \text{—— (8.5)}$$

$$\left[\ \frac{1}{2} * \sigma_{max} * (b * x) = W\ \right]$$

Example-3

A 23 cm thick brick masonary wall (see fig 8.7) carries an axial load of 12 kN/m and eccentric load of 27 kN/m acting at a distance of 7.33 cm from the axis of wall. Design the masonary for the wall if its slenderness ratio is 16. assume that joints are not raked.

Sol^n

Effective thickness

$(t) = 23 - 1 = 22\text{cm}$
$= 0.22\text{m}$

(see clause 4.5.1, SP 20)

Resultant eccentricity

$$e = \frac{W_1 * e_1 + W_2 * e_2}{W_1 + W_2}$$

$$= \frac{12 * 0 + 27 * 0.0733}{12 + 27}$$

$= 5.07$ cm

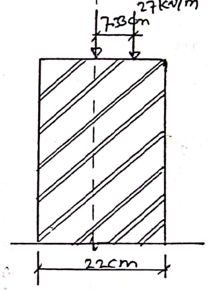


fig 8.7

$$\text{Eccentricity Ratio} = \frac{e}{t}$$

$$= \frac{5.07}{22}$$

$$= 0.23 > \frac{1}{6} -$$

So There will be tension on one face and thickness of wall supporting the load will get reduce

Thickness of wall in compression (Clause 5.4.1.4 (b))

$$x = 3\left(\frac{t}{2} - e\right)$$

$$= 3\left(\frac{22}{2} - 5.07\right)$$

$$= 17.8\text{cm}$$

maximum compressive stress in masonary

$$\sigma_{max} = \frac{2W}{bx}$$

$$\sigma_{max} = \frac{2 * (12+27)\ kN/m}{0.178\ m}$$

$$= 0.44\ N/mm^2$$

Stress Reduction factor (ks)

for $\frac{e}{t} = 0.23$

and SR = 16

ks = 0.59 by interpolation (see Table-9)

Shape modification factor (kp)

$\frac{h}{t}$ of units $= \frac{110}{50}$ $\frac{55}{110} = 0.5 < 0.75$

so, kp = 1 from table 10

Area Reduction factor (ka)

ka = 1 (say) as length of wall is not given assuming A > 0.2m².

As per clause 5.4.1.4(b), 25% increase in permissible stress is allowed.

we have,

permissible stress (fca) = fb * ka * ks * kp * 1.25

$$\Rightarrow f_b = \frac{f_{ca}}{ka * ks * kp * 1.25}$$

$$= \frac{0.44}{1 * 0.59 * 1 * 1.25}$$

$$= 0.6\ N/mm^2$$

from table-8, it can be seen that

crushing strength of brick = 7.5 N/mm²

and mortar type = M1

corresponding to basic compressive stress = 0.74 N/mm²

OR,

crushing strength of brick = 10 N/mm²

and mortar type = L1

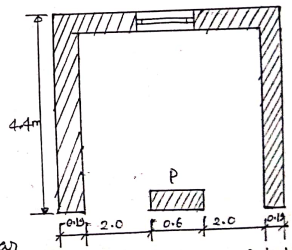
corresponding to basic compressive stress = 0.67 N/mm²

Any of above two results can be adopted.

8.2.4 Walls acting as columns.

Example-4

In a car garrage walls are 20cm thick, height of the floors is 3.0m; plinth is 0.7m above the foundation footing. Roof is constructed with RCC of 12cm thick. If masonary element P in the garrage (see fig 8.8) carries a load of 44 kN at the base inclusive of self load, what should be the strength of bricks and grade of mortar for masonary element 'P'. Assume that joints are not raked.



All dimensions are in 'm'

Fig. 8.8

Soln

Length of masonary element (P) = 0.6m < (4t = 4*19 = 76cm)

so by definition it behaves as a column (See Clause 2.3.1)

t = effective thickness of wall = 20 − 1 = 19 cm load on the wall = 44 KN

stress on base of wall $= \frac{\text{load}}{\text{Area}}$

$= \frac{44\,KN}{0.19 * 0.6}$

$= 385.96\ KN/m^2$

$= 0.386\ N/mm^2$

Area Reduction factor (ka)

Area of plain (A) $= 0.19 * 0.6 = 0.114\,m^2 < 0.2\,m^2$

$\therefore\ ka = 0.7 + 1.5A$

$= 0.7 + 1.5 * 0.114$

$= 0.871$

stress Reduction factor (ks)

eccentricity (e) = 0

effective height (h_{eff}) = H $= 0.7 + 3.0 + 0.06$

$= 3.76\,m$ as per clause 4.3.2

$\therefore\ SR = \frac{h_{eff}}{t_{eff}} = \frac{3.76}{0.19} = 19.8$

($\because$ SR is governed by height and not by length in column)

$\therefore\ ks = 0.625$ from Table-9

By interpolation.

shape modification factor (kp)

$\frac{\text{Height}}{\text{width}}$ of modular brick unit $= \frac{0.9}{0.9} = 1$

~~From table-13~~

as crushing strength of unit is unknown

assuming kp = 1

$\therefore$ Basic compressive stress (f_b) $= \frac{f_{ca}}{Ka * Ks * Kp}$

$= \frac{0.386\ N/mm^2}{0.871 * 0.625 * 1}$

$= 0.71\ N/mm^2$

From table-8, we can find

crushing strength of brick $= 7.5\ N/mm^2$

Now, corrected shape modification factor for crushing strength $= 7.5\ N/mm^2$,

$Kp = 1.1$

$\therefore$ Basic compressive stress should be

$= \frac{0.71}{1.1}$

$= 0.65\ N/mm^2$

Then, Again from table-8

Grade of mortar = M_1

and crushing strength = 7.5 MPa.

should be provided.

Some Problems

Problem-1

Determine the allowable axial load on the column 30cm x 60cm constructed in first class brick work 1:6 mortar using modular brick 200mm x 100mm x 200mm. The Height of the pier between the footing and top of the slab is 5.1m. The strength of units may be assumed as 10MPa.

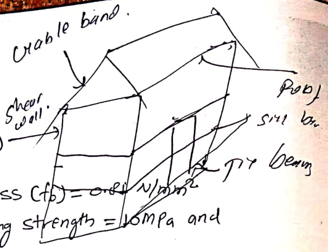
Sol^n Mortar ratio = 1:6. ⇒ Mortar type = M_2

Permissible stress, $f_{ca} = f_b * ka * ks * Kp$.

(clause 5.4.1)

Elements of lateral load resisting masonry system.

Anchorage below . Gable band.



Basic compressive stress (fb)
from Table-8

Basic compressive stress (fb) = 0.81 N/mm²

corresponding to crushing strength = 10 MPa and M_1 mortar

Area reduction factor (ka)

Area of column (A) = 0.3m * 0.6m
= 0.18 m² < 0.2 m²

∴ ka = 0.7 + 1.5A (clause 5.4.1.2)
= 0.7 + 1.5 * 0.18
= 0.97

Element of lateral lo
→ Tie beam or plinth beam
→ Anchorage
+ sill band
+ Shear w
Anchorage between slab & wall

stress reduction factor (ks)

slenderness ratio (SR) = $\frac{h_{eff}}{t_{eff}}$

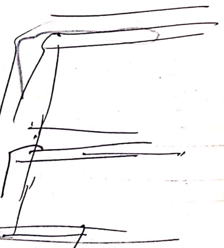
= $\frac{5100}{300}$ = 17

eccentricity (e) = 0

ks = 0.7 from table-9 (by interpolation)

shape modification factor (kp)

$\frac{\text{Height (h)}}{\text{width (w)}}$ of unit = $\frac{200}{100}$ = 2



from table-10

kp = 1.3

∴ fca = 0.81 * 0.97 * 0.7 * 1.3
= 0.715 N/mm²

and Allowable axial load = fca * A
= 0.715 N/mm² * 0.18 * 10⁶ mm²
= 128.7 kN Ans

Problem-2.

A load bearing brick masonary wall of a building is 20cm thick, is laterally supported by RCC slabs at top and bottom, which are 12cm thick each and clear height between slabs is 3.0m. If the wall has an axial load of 71.5 kN/m at the base, inclusive of self weight, what should be the crushing strength of bricks and grade of mortar for the wall. wall is 4m long between cross walls and bricks used are of modular size. Assume that there are no openings in the wall within H/8 of its juction with cross walls and there are no openings in cross walls within H/5 of their junctions with the load bearing wall under consideration. Assume that two ends of wall are discontinuous and joints are not racked.

Soln

Effective height (h) = 0.75 H (clause 4.3.1)
= 0.75 (3 + 0.12) , H = c/c distance between two floors.
= 2.34 m

Effective thickness (teff) = 19 cm (joints are not raked)

SR, heightwise = $\frac{h_{eff}}{t_{eff}} = \frac{2.34}{0.19}$ = 12.3

Since, cross walls are more than 20t = 20 * 0.19 = 3.8m apart, value of stifening coefficient (kn) = 1 (See table-6)

∴ SR, heightwise = $\frac{12.3}{1}$ = 12.3

Effective length (Leff) = L = 4.0m (Table-5)

∴ SR lengthwise = $\frac{L_{eff}}{t_{eff}} = \frac{4}{0.19}$ = 21

since, SR, heightwise < SR, lengthwise

SR = 12.3 will govern the design

Ecentricity = 0

compressive stress in masonary $= \dfrac{\text{Load (kN/m)}}{t \text{ (thickness of wall)}}$

$= \dfrac{71.5 \text{ kN/m}}{0.19 \text{ m}}$

$= 376.32 \text{ kN/m}^2$

$= 0.376 \text{ N/mm}^2$

Area reduction factor. (k_a)

$A = 0.19 * 4.0 = 0.76 \text{ m}^2 > 0.2 \text{ m}^2$

$\therefore k_a = 1$.

Stress Reduction factor. (k_s)

From table - 9 for $SR = 12.3$ and $e = 0$

$k_s = 0.83$ (By interpolation)

shape modification factor (k_p)

For modular bricks, $\dfrac{\text{height}}{\text{width}}$ of unit $= \dfrac{0.9}{0.9} = 1$

But crushing strength of brick is unknown

let, $k_p = 1$.

$\therefore$ basic compressive stress (f_b) $= \dfrac{f_{ca}}{k_a * k_s * k_p}$

$= \dfrac{0.376}{1 * 0.83 * 1}$

$= 0.45 \text{ N/mm}^2$

Refering to table-8, we get

crushing strength of brick $= 5 \text{ N/mm}^2$

Now

$k_p = 1.2$ corresponding to crushing strength $= 5 \text{ N/mm}^2$

$\therefore$ ~~fb~~ $f_b = \dfrac{0.45}{1.2} = 0.38 \text{ N/mm}^2$

Again, Refering to table-8

for crushing strength 5 N/mm^2

M_3 mortar can be used.

Problem-3

A wall 20cm thick, using modular bricks carries at the top a load of 80 kN/m having resultant eccentricity ratio of $\frac{1}{12}$. wall is 5m long between cross walls and is of 3.4m clear height between RCC slabs at the top and bottom. What should be the strength of brick and grade of mortar. Assume that joints are not raked.

Soln

Load on ~~load~~ wall $= 80 \text{ kN/m}$

Effective thickness of wall $= 20 - 1 = 19 \text{ cm} = t$

Effective height (h_{eff}) $=$ ~~H = 3.4m~~ $0.75 H$ (Clause 4.3.1)

$= 0.75 * 3.4$

$= 2.55 \text{ m}$

Effective length (L_{eff}) $= L$ (Table-5)

$= 5 \text{ m}$

since, $h_{eff} < L_{eff}$

SR in the direction of height governs the design

$\therefore SR = \dfrac{h_{eff}}{t} = \dfrac{2.55}{0.19} = 13.42$

self weight of wall $=$ unit wt. $*$ height of wall $*$ thicknes

$= 20 \text{ kN/m}^3 * 3.4 \text{ m} * 0.19 \text{ m}$

$= 12.92 \text{ kN/m}$

Total Load $= 80 + 12.92$

$(W) = 92.92 \text{ kN/m}$

Total stress $= \dfrac{W}{t}\left(1 + \dfrac{6e}{t}\right)$

eccentricity ratio $= \dfrac{e}{t}$

$\therefore$ Total stress $= \dfrac{92.92}{0.19}\left(1 + 6 * \frac{1}{12}\right)$

$= 733.58 \text{ kN/m}^2$

$= 0.734 \text{ N/mm}^2$

Stress reduction factor (k_s)

for $SR = 13.4$

and eccentricity ratio $= \frac{1}{12}$

$k_s = 0.76$ From table-9 By interpolation.

Area reduction factor (k_a)

$A = 5 * 0.19 = 0.95\ m^2 > 0.2\ m^2$

$\therefore k_a = 1$

Shape modification factor (k_p)

for modular brick, $\frac{\text{height}}{\text{width}}$ of unit $= 1$

But, crushing strength of brick is unknown

assuming $k_p = 1$

Basic compressive stress $(f_b) = \dfrac{f_{ca}}{1.25 * k_a * k_s * k_p}$

25% additional stress is allowed for $\frac{1}{24} < \frac{e}{t} < \frac{1}{6}$

as per ~~code~~ clause 5.4.1.4 (a)

$\therefore f_b = \dfrac{0.734}{1.25 * 1 * 0.76 * 1} = 0.773\ N/mm^2$

from table-8 we find that bricks should have ~~compressive~~ crushing strength $= 10\ MPa$

Now, for crushing strength $= 10\ MPa$

$k_p = 1.1$

$\therefore$ basic compressive stress

$f_b = \dfrac{0.773}{1.1} = 0.703\ N/mm^2$

again from Table-8, we can see that

for crushing strength $= 10\ MPa$

mortar type $= M_3$ can be used.

Problem-4

A load bearing brick masonary wall of a building is 20cm thick, is laterally supported by RCC slabs at top ~~and, which~~ and bottom, which are 12cm thick each and clear height between slab and ground is 3.0m. A live load of 3 kN/m² is applied at the top of slab. Determine the crushing strength of brick and grade of mortar, if brick sized standard.

soln

Assume, joints are not raked.

Effective thickness $(t_{eff}) = 19\ cm = 0.19\ m$

Effective height $(h_{eff}) = 0.75H$ clause 4.3.1

$= 0.75(3 + 0.12)$

$= 2.34\ m$

Load calculation.

Load from 12cm RCC slab = unit wt * thickness of slab * width of slab

$= 25\ kN/m^3 * 0.12 * 1m$ (assuming unit width)

$= 3\ kN/m$

Live load $= 3\ kN/m^2 *$ width of slab

$= 3\ kN/m^2 * 1m$

$= 3\ kN/m$

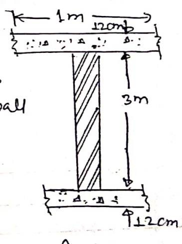


Fig 8-9

Self wt. of wall = unit wt * thickness of wall * height of wall

$= 20\ kN/m^3 * 0.19 * 3.0m$

$= 11.4\ kN/m$

Total load $= 17.4\ kN/m$

Total stress $= \dfrac{17.4\ kN/m}{0.19\ m}$

$= 91.58\ kN/m^2$

$= 0.0916\ N/mm^2$

Area Reduction factor (k_a)

As length of wall is not given, it is assumed that,

$A > 0.2 m^2$

$\therefore k_a = 1$

stress reduction factor (k_s)

eccentricity (e) = 0

slenderness ratio (SR) = $\frac{h_{eff}}{t_{eff}} = \frac{2.34}{0.19} = 12.3$

$\therefore k_s = 0.83$ from table-5 (By interpolation)

shape modification factor (k_p)

For modular brick

$\frac{\text{height}}{\text{width}}$ of brick unit = $\frac{0.9}{0.9} = 1$

But crushing strength of brick is not known

so, Assuming $k_p = 1$

$\therefore$ Basic compressive stress (f_b) = $\frac{f_{ca}}{k_a * k_s * k_p}$

$= \frac{0.0916 \ N/mm^2}{1 * 0.83 * 1}$

$= 0.11 \ N/mm^2$

from table-8, it can be seen that,

crushing strength = 3.5 N/mm^2 for $f_b = 0.25 \ N/mm^2$

as shape modification factor for crushing strength below 5 N/mm^2 is not given in code, it can be assumed that, $k_p = 1$

So, again from table-8,

For crushing strength = 3.5 N/mm^2,

L1 or L2 or M3 mortar can be used.

Problem-5

A column section 400x800mm carries a load 100kN acting at 160mm from the 800mm face and 350mm from the 400mm face. Determine the stress intensities at all four corners.

Soln

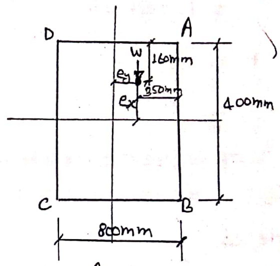


fig. 8.10

$e_x = \frac{400}{2} - 160$

$= 40mm$

$e_y = \frac{800}{2} - 350$

$= 50mm$

$W = 100kN$

$b = 400mm$

$d = 800mm$

we have, stress at any point

$\cancel{\sigma = \frac{W}{bd}\left[1 + \frac{6e_x}{b} + \frac{6e_y}{d}\right]}$

$\sigma = \frac{W}{bd}\left[1 + \frac{12 e_x}{b^2}.y + \frac{12 e_y}{d^2}.x\right]$

For point, A, $x = \frac{800}{2} = 400mm$, $y = \frac{400}{2} = 200mm$

$\therefore \sigma_A = \frac{100kN}{0.4*0.8}\left[1 + \frac{12*0.04}{0.4^2}*0.2 + \frac{12*0.05}{0.8^2}*0.4\right]$

$= 617.19 \ kN/m^2$

For point, B,

$x = \frac{800}{2} = 400mm$, $y = -\frac{400}{2} = -200mm$

$\therefore \sigma_B = \frac{100}{0.4*0.8}\left[1 + \frac{12*0.04}{0.4^2}*0.2 + \frac{12*0.05}{0.8^2}*0.4\right]$

$= 242.19 \ kN/m^2$

For point C,

$x = -0.4m, \quad y = -0.2m$

$\therefore \sigma_C = 312.5\,(1 - 3*0.2 - 0.9375*0.4)$

$= 7.81\ kN/m^2$

For point D,

$x = -0.4m, \quad y = 0.2m$

$\therefore \sigma_D = 312.5\,(1 + 3*0.2 - 0.9375*0.4)$

$= 382.81\ kN/m^2$

Hence

$$\left.\begin{aligned} \sigma_A &= 617.19\ kN/m^2 \\ \sigma_B &= 242.19\ kN/m^2 \\ \sigma_C &= 7.81\ kN/m^2 \\ \sigma_D &= 382.81\ kN/m^2 \end{aligned}\right\}\ \text{Ans.}$$

Problem-6

The bottom width of a retaining wall is 3m. The resultant thrust has the normal component of 150 kN/m Length of wall, and crosses the bottom at a distance of
a) 400mm
b) 600mm from the centre. Determine the intensity of maximum and minimum pressure on the foundation.

Soln

Assuming unit length of wall,

Here, $\frac{d}{6} = \frac{3}{6} = 0.5m$

a). $e = 400mm < \frac{d}{6} = 500mm$

we have,

$\sigma_{\cancel{max/min}} = \frac{P}{bd}\left(1 \pm \frac{12e_y}{d^2}\cdot x\right)$

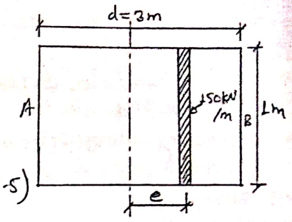


Fig 8.11

$\sigma_{max} = \frac{150kN}{1*3}\left(1 + \frac{12*0.4}{3^2}*1.5\right)$

i.e $\sigma_B = 90\ kN/m^2$

$\sigma_{min} = \sigma_A = \frac{150}{1*3}\left(1 - \frac{12*0.4}{3^2}*1.5\right)$

$= 10\ kN/m^2$

b). $e = 600mm > \left(\frac{d}{6} = 500mm\right)$

Hence, the end A will lift off due to tension from the soil and contact width will be reduced.

So, reduced contact width, $x = 3\left(\frac{d}{2} - e\right)$

$= 3\,(1.5 - 0.6)$

$= 2.7m$

Here, $\sigma_{min} = 0$

and, $\sigma_{max} = \frac{2P}{bx} = \frac{2*150}{1*2.7} = 111.11\ kN/m^2$

Pressure diagram of both the cases are shown in fig 8.12

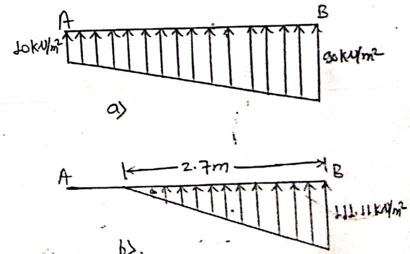


Fig 8.12 Pressure diagram.

Problem-7.

Design a wall of two storey building carrying concrete slab with a storey height of 3m. The wall is stiffened by 110mm thick intersecting wall at 4m center to center spacing. The wall has a door opening of size 900mm * 2000mm at a distance of 300mm from one of the intersecting wall assume loading are as follows.

i) Roof loading = 20 kN/m

ii) Floor loading = 15 kN/m

Take the strength of masonary units is 15 MPa and Mortar M_1.

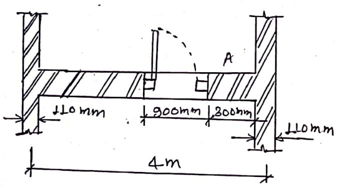
Soln

Assume,

thickness of wall = 230mm

self wt of wall = 2 * 0.23 * 3 * 20 kN/m³

= 27.6 kN/m



Roof load = 20 kN/m

Floor load = 15 kN/m

Total load = 27.6 + 20 + 15

W = 62.6 kN/m

Effective thickness of wall above opening = 23 - 1 = 22cm (assuming joints are not raked)

∴ total stress at base = $\frac{W}{A}$

Effective Area (A) = (4000 - 900) * 220

= 682000 mm²

= 0.682 m²

∴ total stress at base = $\frac{62.6 \text{ kN/m} * 4m}{0.682 \text{ m}^2}$

= 367.16 kN/m²

= 0.367 N/mm²

In opening

effective thickness = $\frac{\text{Effective area}}{\text{length of wall}}$

= $\frac{682000 \text{ mm}^2}{4000 \text{ mm}}$

= 170.5 mm

$t_{eff} = \frac{220 + 170.5}{2} = 195.25$ kn

Effective height = where, kn = stiffening coefficient

For kn,

$\frac{t_p}{t_w} = 3$ Clause 4.5.3

$\frac{S_p}{W_p} = \frac{4000 \text{mm}}{110 \text{ mm}} = 36.36 > 20$

from table-6,

kn = 1

∴ t_{eff} = 195.25mm

$h_{eff} = 0.75 H = 0.75 * 3000$ Clause 4.3.1

= 2250 mm

Area reduction factor (ka)

A = 0.682 m² > 0.2m²

∴ ka = 1

Stress reduction factor (ks)

eccentricity (e) = 0

$SR = \frac{h_{eff}}{t_{eff}} = \frac{2250}{195.25} = 11.52$

from table-9,

ks = 0.852 (By interpolation)

Shape modification factor (kp)

$\frac{\text{Height}}{\text{width}}$ of units = $\frac{57}{110} = 0.52 < 0.75$

and crushing strength = 15 N/mm²

$\therefore k_p = 1$

Basic compressive stress (fb)

From table – 8

for crushing strength = 15 N/mm²

and mortar type = M_1

$f_b = 1.13$ N/mm²

$\therefore$ Allowable stress (fca) = $f_b * k_a * k_s * k_p$

$= 1.13 * 1 * 0.852 * 1$

$= 0.963$ N/mm²

Check,

Here,

fca > total stress at base of wall

So, design is O.K

Hence, provide, wall thickness = 230mm

Crushing strength = 15 N/mm²

grade of mortar = M_1

CHAPTER-9 Masonry Structures under lateral Loads

9.1 Performance of masonry structures in lateral loads

In general, masonry structures are very good in resisting gravity Loads, but do not perform well when subjected to lateral in plane Loading, such as seismic loads caused by an earthquake. As Indonesia is located in a high risk seismic region, many masonry houses experienced severse damage during past earthquakes that caused many injuries and deaths. The houses collapsed gradually in brittle failure without ductility. The performance characteristics of wall material in the house of this region have never been investigated nor studied. Local builders have always followed a traditional way to construct their houses, with very poor knowledge on construction of masonry houses.

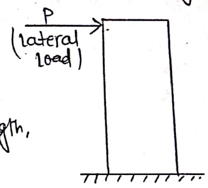


fig: lateral load on masonry structure.

Masonry walls resisting in-plane Loads usually exhibit the following three modes of failure:-

i) **Sliding shear** – a wall with poor shear strength, loaded predominantly with horizontal for exhibits this failure mechanism. Aspect ratio for such walls is usually 1:1 or less (1:1.5).

ii) **Shear** – a wall loaded with significant vertical load as well as horizontal forces can fail in the most common mode of failure. Aspect ratio for such walls is usually about 1:1. Shear also occur for panels with bigger aspect ratio i.e 2:1, in case of big vertical load.

iii) **Bending** – this type of failure can occur if walls are with improved shear resistance. For the ratio i.e 2:1 bending failure can ocure due to small vertical loads, rather than high shear. In this mode of failure the masonry panel can rock like a rigid.

Seismic behaviour of masonry construction has been very frequently unsatisfactory and it is often unfavorably compared with the performance of steel and concrete structures.

9.2 Failure behaviour of masonry structures in lateral Loads.

Masonry buildings, designed and constructed according to requirements of modern seismic codes, behaved adequately. Cases of collapse were rare and were limited to buildings where the requirements of codes, especially those related to the quality of construction, were only partly met. Although the structural typology of masonry buildings varies in different regions, their damage resulting from earthquakes can be classified than uniform way. Structural elements, such as walls, columns and beams are only bearing the weight of the buildings and the Live load under normal conditions, mostly compression forces for the walls and columns and vertical bending for the beams. Under dynamic load, they also have to withstand horizontal bending and shear forces and extra vertical compression forces. The following typical types of failure behaviour of masonry structures is shown in figure below.

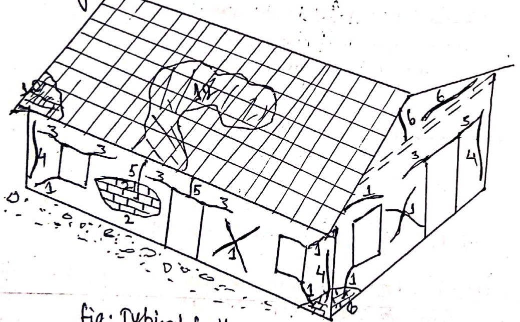


Fig: Typical failures in a masonry structure.

1) Diagonal shear cracks
2) Horizontal shear cracks
3) Bending cracks at lintels and feet
4) Bending cracks at corners
5) Bending cracks at spandrel
6) Bending cracks at gable
7) plaster peeling off

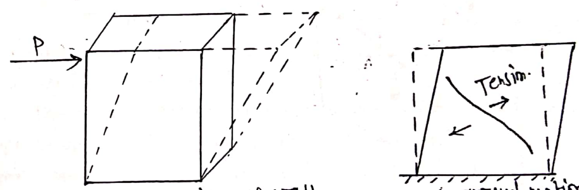
8) crushing of weak masonry under vertical ground motion.
9) Badly anchored roof, pulled out by vertical ground motion.
10) Damage of tiles roof with shear (roof not braced).
11) Falling of tiles from the roof eave.

9.3 In-plane and out of plane behaviour of masonry structures

In plane and out of plane behavior of masonry structures also called the failure mechanism of walls.

① In-plane failure.

→ The failure, ~~in the direction~~ which is pushed in the direction of the plane of wall is called In plane failure.



pushed in the plane of wall.

← ground motion in the wall plane.

Fig: failure patterns In plane failure.

→ A masonry wall joints with the bed in horizontal direction is supported at the lower side, and the upper side loaded in-plane by a horizontal force (P). The failure of wall ~~force~~ is caused by combined effect of normal compressive and shear stresses, which is represented by the principal tensile stress and when it exceeds, the diagonal tensile strength of the masonry failure will take place which is shown in figure above.

② Out-of plane failure.

→ The failure, which is pushed in the direction of perpendicular to the wall is called out-of plane failure. In the out-of plane failure the two types of failure occured which are:-

ⓐ The out-of plane failure ~~facade~~ of the facade, which breaks away from the transversal wall with a vertical crack at the abutment.

ⓑ The rigid body out of plane overturning of the facade with part of the transversal wall, breaking away with a diagonal crack at an angle φ with respect to the vertical.

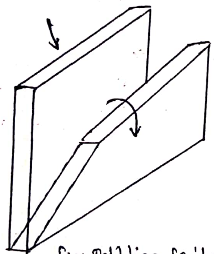


fig: Toppling failure.

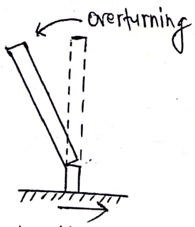


fig: Ground motion perpendicular to the wall.

9.4 Ductile behaviour of reinforced and unreinforced masonry structures:-

As we know that masonary structures are very weak in tension and strong in compression. This is due to the ductile property of masonary. The ductality of masonary structures depends up on the the ductality of masonary units properties of mortar. Saying about masonary (un reinforced) these are brittle in nature instead of ductile. Due to this property unreinforced masonary can not withstand any tensile and cracks are formed in the portion of tension in masonary. The main cause of in-plane and out-of plane failure is also ~~the ductility of~~ due to the ductality of masonary. ~~Th~~

To improve the ductile nature of masonary reinforcements are embeded in the masonary. This type of masonary is called reinforced masonary and have higher ductality than unreinforced masonary. Due to the ductile property of reinforced masonary, it can resist seismic load more than that of unreinforced masonary and performe well in earthqueke. The damage due to in-plane and out-of plane failure in reinforced masonary is very little as compared to unreinforced masonary due to the same property. The reinforcement in masonary also increases the compressive strength and shear strength and improves the connection between structural walls.

The stress distribution is theoretically uniform throughout but depends on the uniformity of the masonary and the mortar.

Subsequently, consider a lateral load 'W' kN/m^2 applied to the section of wall and the corresponding bending moment diagram applied to this alone. The stress distri~~buted~~ bution from the flexure of wall is calculated from eqn of theory of flexure.

i.e,
$$\frac{\sigma_b}{y} = \frac{M}{I} \qquad \text{—— (9.2)}$$

or
$$\sigma_b = \frac{M}{I} \cdot y = \frac{M}{\frac{I}{y}}$$

$$\therefore \sigma_b = \frac{M}{Z} \qquad \text{—— (9.3)}$$

where,

σ_b = bending stress

M = maxm bending moment

I = moment of Inertia about the length of wall.

Z = section modulus.
$= \frac{I}{y}$.

y = distance of considered layer from N.A (Neutral axis)

By considering the deflected shape of wall, it can be seen that the inner surface will be in tension and a corresponding stress diagram can be drawn. The simplified stress diagrams for the masonary wall indicate that tension is theoretically eliminated when combining the stress distributions from pre-compression with those ~~from~~ from the out of plane loading (see fig 9.7).

A wall restrained from the top and bottom only will fail by cracking on tension side following minor deflections when subjected to lateral loads. Explained in simple terms, the loaded wall will be in compression on the side of the loading and in tension on the other face. Hence cracks will develop on the tensile side with a minor load. To prevent cracking on tension face, the tensile stresses should not be greater than the applied pre-compression stresses.

For the loading as shown in figure 9.8. shear stress can be calculated as

$$\tau = \frac{P}{L \times t}$$

where,

L = length of wall

t = thickness of wall

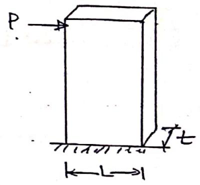


Fig. 9.8

For combined loading system, shear ~~stress~~ stress can be calculated as.

$$\tau = \frac{VQ}{Ib}$$

where,

V = shear force at the section

Q = moment about the N.A. of section of that portion of cross-section lying between the plane under consideration and extreme fibre

I = moment of inertial about N.A

b = width of section

(for detail see Theory of Flexure in any book of Theory of structure)

9.5 Calculation of stresses for lateral loads

When a wall is subjected to a lateral load such as that resulting from wind pressure, bending will occur depending on the lateral support conditions. In typical construction vertical and horizontal supports are provided by elements such as cross walls and concrete floors or roofs respectively. The lateral support conditions of a laterally loaded panel have a considerable bearing on the actions that it will go through following out-of-plane loading.

Laterally loaded walls transfer their loads through combined vertical and horizontal bending. The transfer of stress in masonary wall, vary most widely in their distribution throughout the panel following a combination of the lateral loading together with a vertical and horizontal preload. The combination of the effect of these loads on the wall will vary from point to point in the structural volume due to vast transfer of stresses. The state of stress within the masonary block depends on the transfer of stresses from one block to the other through the mortar. On the other hand the mortar is found to be in a tri-axial stress state depending on the loading characteristics.

Masonary is a material which exhibits distinct directional properties as the joints act as planes of weakness. The main weakness of masonary construction can be considered to be the mortar joint at which in elastic deformations occur in the mortar itself or by sliding of the joint. This causes masonary to have a very complex behaviour due to plane weakness at the interface.

Consider a wall with a uniformly distributed force 'p' applied at each end along the center line in one axis. This force is caused by self weight of wall and other external loading and applies a uniform compressive stress across the section.

Compressive stress due to load 'P' $= \frac{P}{A}$ —— (9.1)

Where,
A = cross-section area of wall.

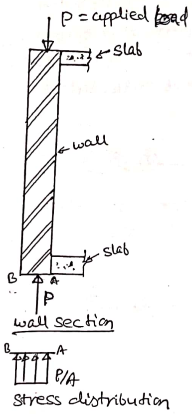


Fig. 9.6 (a). wall subjected to pre-compression

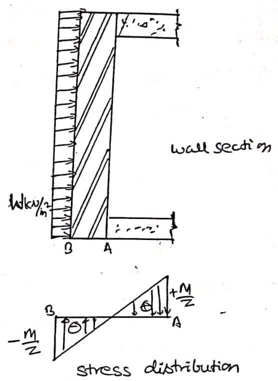


Fig 9.6 (b) : wall subjected to lateral load

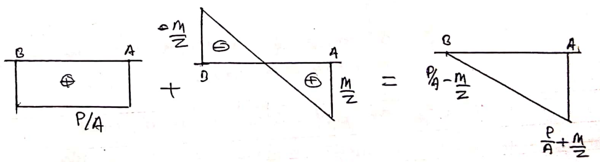


Fig. 9.7. simplified stress diagram for pre-compression and lateral loading.

9.6 Elements of lateral load resisting masonary system

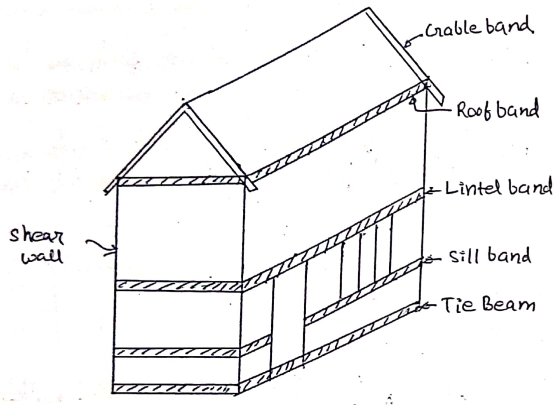


Fig 9.9. Different elements of masonary structure

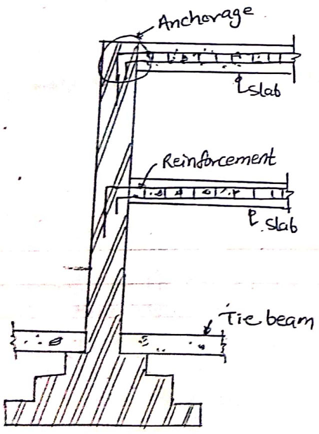


Fig. 9.10 Connection between slab & wall

The elements of masonary structure resisting lateral load are listed below.

i) Tie beam
ii) Sill band
iii) Lintel band
iv) Roof band
v) Gable band
vi) Anchorage with walls
vii) Shear wall
viii) Anchorage between slabs and wall.

The elements are shown clearly in the fig 9.9 and 9.10.

CHAPTER–10 Testing of Masonary Elements.

10.1 Compressive strength of bricks and walls.

Compressive strength of brick is an important material property for structural applications. In general, increasing the compressive strength of unit will increase the masonary assemblage compressive strength and elastic modulus. Bricks are frequently specified by material standard rather than by a particular minimum unit compressive strength. ASTM (American society for Testing and Material) material standards for brick require minimum compressive strengths to ensure durability. Which may be as little as one-fifth the actual unit compressive strength. The compressive strength of bricks depends upon the type of soil (material) used and method of manufacturing and uniformity of brick.

The factors governing the strength of a brick structure (walls) include compressive strength of brick unit, mortar strength and elasticity, bricklayer workmanship, brick uniformity and the method used to lay the brick (i.e type of bond used). As Portland cement-lime mortar is stronger than the brick, brick masonary laid with this mortar is stronger than an individual brick unit. The load carrying capacity of a wall or column made with plain lime mortar is less than half that made with portland cement-lime mortar.

The compressive strength of individual bricks or walls ~~are~~ is tested in lab by using compression testing machine or Universal testing machine. To test the compressive strength of brick or wall samples should be prepared.

Sampling.

Brick

→ Remove unevenness observed in the bed faces to provide two smooth parallel faces by grinding.

→ Immerse in water at room temperature for 24 hours

→ Remove the specimen and drain out any surplus moisture at room temperature.
→ Fill the frog and all voids in the bed faces flush with cement mortar (1 cement : 1 clean course sand of grade 3mm and down).
→ store it under the damp jute bags for 24 hours filled by immersion in clean water for 3 days. Remove and wipe out any traces of moisture.
→ Now specimen is ready to test

walls

→ Prepare the walls (at least 3 in number) with a specific bond.
→ Finish the top and ~~both~~ bottom of the wall by placing mortar in the frogs and gaps.
→ If cement mortar is used cover the wall with jute bags for 24 hours filled by immersion in clean water for 3 days.
→ Remove the jute bag after 3 days and wipe out any traces of moisture and specimen is ready now.

Procedure

i). Place the specimen with flat faces between the plates of testing machine

ii): Apply load axially at a uniform rate of 140 kg/cm^2 per minute till failure occurs and note down the maximum load at failure

iii). The load at failure is maximum load at which the specimen fails to produce any further increase in the indicator reading on the testing machine

Calculation

$$\text{Compressive strength} = \frac{\text{max}^m \text{ load at failure}}{\text{Average contact area}}$$

⇒ Contact area should be calculated by measuring length and breadth of brick and length and thickness of wall, before testing the specimen.

normal compressive & shear stress

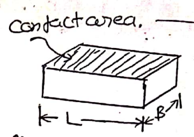


Fig. 10.1 Brick
Contact area = L*B = A

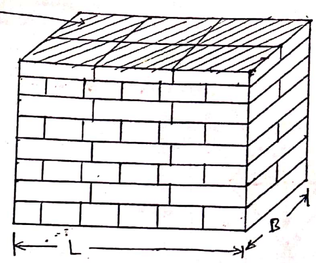


Fig 10.2 wall specimen.

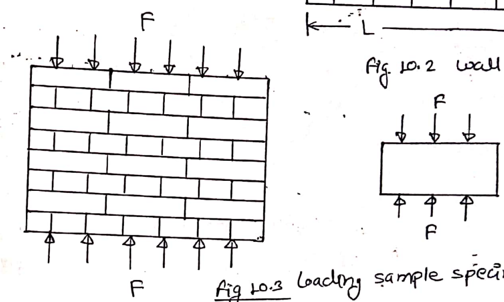


Fig 10.3 loading sample specimen.

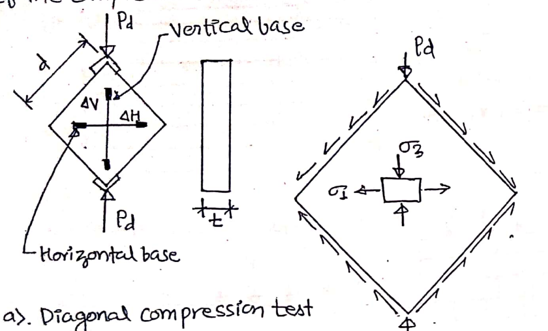
∴ Compressive strength = $\frac{F}{A}$ (= N/mm^2 or kg/cm^2)

The average of compressive strength of samples tested will be the required compressive strength.

10.2 Diagonal shear test

The failure of walls during earthquake is caused by combined effect of normal compressive and shear stresses, which is represented by the principal tensile stress and when it exceeds, the diagonal tensile strength of the masonary failure will take place.

The diagonal shear test is based on subjecting a 1.2m x 1.2m square section of wall by the thickness of the wall type to diagonal compression through loading shoes at two diagonally opposite corners of the specimen as shown in Fig 10.4(a). The failure mode of the test is through formation of diagonal crack parallel to the line of action of the compression force.



a). Diagonal compression test setup

b). Diagonal test

Fig. 10.4

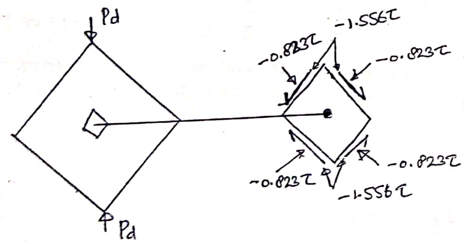


Fig 10.5 state of stress in the center of Panel

The Interpretation of this posses some interrogative: This test was introduced to simulate a pure ~~state~~ shear stress state, according to the scheme in Fig 10.4(b). The diagonal shear stress may be calculated from the equation,

$$\tau = \frac{P_d}{\sqrt{2}A} \quad \text{------} \quad (10.1)$$

where,

P_d = Applied load (N)

A = Average of the gross or net area of the cross-section along the bed and head joints in mm²

The gross area is used in calculation if the specimens are constructed with solid units while net area is used in case of specimen with hollow units.

Net Area of specimen $(A_n) = d * t * n$

where,

d = side of square panel. wall (mm)

t = total thickness of specimen. (mm)

n = % of the gross area of the unit that is solid, expressed in a decimal

As 'A' is the area of the side of the square panel; hence, the principal tensile stress σ_I is equal to the shear stress. if P_{du} denotes the maximum value of P_d the shear strength is given by.

$$\tau_u = \frac{P_{du}}{\sqrt{2}A} \quad \text{------} \quad (10.2)$$

On the other hand a linear elastic analysis of the panel considered as an homogeneous solid, gives the localized value of the principal tensile stress in the center of the specimen as $\sigma_I \cong \frac{P_d}{2A}$ $(0.519\, P_d/A)$, whereas the maximum value of shear stress is $\tau_{max} = 1.1\frac{P_d}{A}$. According to this interpretation some Authors, as in (Tubi, 1993) and [Calderoni, 1996], equalize the shear without normal stress (namely fvko in the Italian standards, 1987) to the tensile strength, assuming the equation

$$\tau_u = \frac{P_{du}}{2A} \quad \text{------} \quad (10.3)$$

Therefore two different interpretations of diagonal test results are possible. The first interpretation [Eqn (10.2)] is the most commonly used for comparison purposes although some Authors proposed modifications to be used for interpreting and evaluating tensile strength of masonary by diagonal tests [Ghanem, sheirf and Honsy, 1994].

The modulus of rigidity or shear modulus may be calculated as follows (ASTM, 2002):

$$G = \frac{\tau_s}{\gamma} \quad \text{------} \quad (10.4)$$

where,

G = modulus of rigidity, MPa

τ_s = Diagonal shear ~~strength~~ stress

γ = Shear strain calculated ~~as~~

$= \frac{\Delta V + \Delta H}{g}$

ΔV = Vertical shortening (mm)

ΔH = Horizontal shortening, (mm)

g = vertical gauge length (mm)

(ΔH must be based on the same gauge as for ΔV).

10.3 Non-destructive tests - Elastic wave tomography, Flat jack, Push shear test and others.

Non-destructive test is the test used to determine the strength of the specimen (masonary) without failure of the specimen (i.e without destruction of specimen). There are so many types of Non-destructive tests available to test masonary.

1). Elastic wave tomography.
2). Flat jack Test.
3). Push shear Test
4). Impact Echo Test
5). Ultrasonic Pulse Velocity etc

1). Elastic wave tomography.

→ Elastic wave tomography is a Non-destructive testing technique for masonary or concrete or other material

→ This technique is used for locating shallow delaminations, cracks and voids.

→ Elastic wave tomography is based on two basic principles from heat transfer: conduction and radiation

→ Sound materials with no voids, gaps or cracks are more thermally conductive than materials that are delaminated or contain moisture.

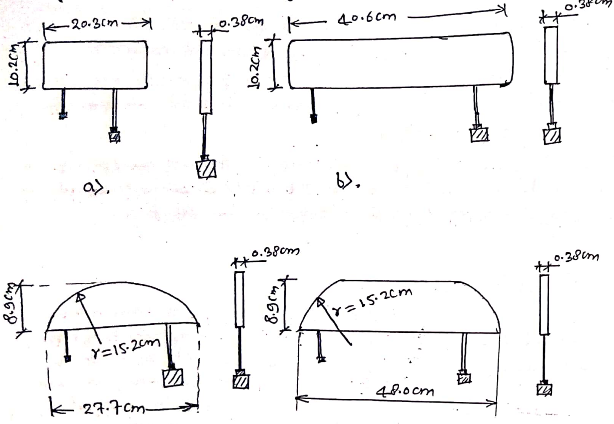
→ This allows rapid areal mapping of internal conditions. It should be noted that the IT method is most useful for the detection of shallow defects and flaws

2). Flat Jack Test.

Flat jack testing is a Non-destructive, versatile and powerful technique of testing masonary existing masonary structures that provides significant in information on mechanical properties of historical construction (masonary). Flat jack testing is direct and in-situ testing method that requires only removal of a portion of mortar from the bed joints. So It's considered non destructive because the damage is temporary and is easily repaired after testing

Description of Flat Jack.

Flat Jack is a "thin envelope-like bladder with inlet and outlet ports which may be pressurized with hydraulic oil". Some typical configurations are shown in fig 10.6. A flat Jack may be manufactured in many shapes and sizes. The actual dimensions are determined by its function, slot preparation technique and properties of the masonary being tested. Flat jacks with curved edges are designed to fit in a slot cut by a circular saw. Rectangular jacks (types a, b) are used where mortar must be removed by hand or with stitch drilling. Regardless of the shape, a flat jack must fit the slot well. The thickness of the Flat Jack is determined by its specific function. An ideal flat jack will completely fill the slot in the mortar joint. However, if such flat-jack is not available, then shims are used together with the flat Jack to completely fill the slot thickness.



c). Flat jack with curved edge d). Flat jack with curved edge

Fig 10.6 Different flat jack configurations.

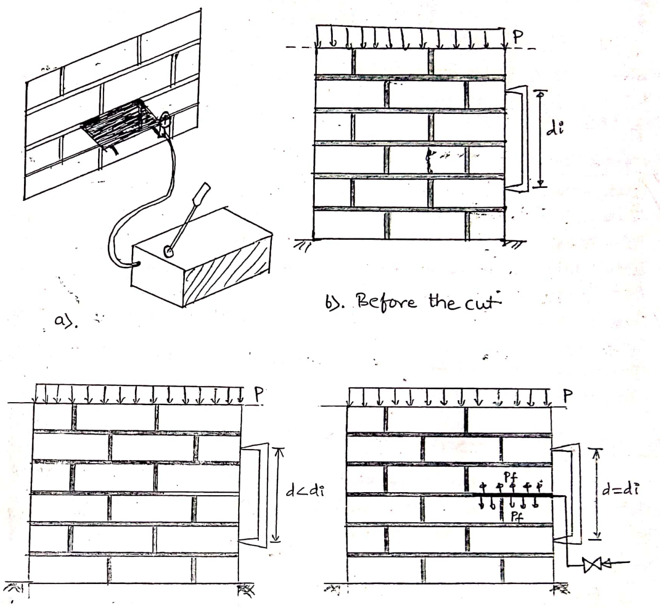
There are basically two types of Flat jack Tests

1). <u>In-situ stress test (single flat-jack Test)</u>

This test is based on the principle of partial stress release and involves the local elliminimination of stresses, followed by controlled stress compensation (see fig 10.7)

The reference field of displacements is first determined by measuring distances between the gauge points fixed to the surface of the masonary (distance di in fig 10.7(b)). Then a slot is cut in a plane normal to the direction of measured stresses. This allows deformations in a direction normal to the ~~direction~~ slot. Distance between gauge points

decreases (i.e distance d in fig 10.7(c) is less than di). Cutting the slot causes partial stress relief in masonary above and below.



c). after the cut d). pressure Pf

Fig 10.7 Phases of Flat-Jack test (P= Pf when d=di)

Afterwards, a thin flat jack is introduced into the slot. with the aid of this device, pressure is applied to the masonary. This causes a partial restoration of the initial displacement field, ~~which at or point~~ the displacement some point reach previously measured value (see fig 10.7(d)). The necessary pressure Pf (called cancelling pressure) can be related to the compressive stress in the direction normal to the slot. The Hydraulic pressure in the Flat-Jack necessary to restore the undamaged state is higher than the actual stress. This is caused by inherent stiffness of the flat-Jack, which resists expansions when the Jack is pressurized. Another factor that contributes to this effect is the difference between the area of the Jack and the area of slot (The latter being greater than the former) Both these factors are taken in account when the interpeting test results.

Assumptions:

→ The stress in place of test is compressive
→ The masonary surrounding the slot is homogeneous
→ The masonary deforms symmetrically around the slot
→ The state of stresses in the place of the measurement is uniform.
→ The stress applied to the masonary by flat-Jack is uniform.
→ The value of stresses allows the masonary to work in elastic regime.

2). In-situ deformability test (two flat-Jack test)

The principle of the test is similar to a standard compressive test. The difference is that it is performed in-situ and two flat jacks are used to apply load. A typical setup of the in-situ deformability test is shown in fig 10.8

By cutting two parallel slots, a part of the wall is ~~associated~~ isolated from the surrounding masonary forming a "specimen". Masonary between the flat-jack is assumed to be unstressed. The flat jacks are then introduced into both slots, and the initial distance between gauge points are measured. By pressurizing flat-jacks, the load is applied to the 'specimen' creating an approximately uniaxial state of compressive stress. with a pressure increase in the flat jacks, the distance between gauge point pairs decrease. By gradually increasing the pressure, the stress-strain relationship can be determined. Loading and unloading cycles can also be performed

Based on an experimental stress-strain curve, the value of Young's modulus of elasticity can be calculated. If extended damage in the specimen is acceptable, the compressive strength of masonary can be obtained. During testing, the load-displacement diagram is monitored and, when it becomes highly non-linear (indicating imminent failure) loading is usually terminated. Even in this case, it is possible to estimate peak compressive strength by extraplating the stress-strain curve

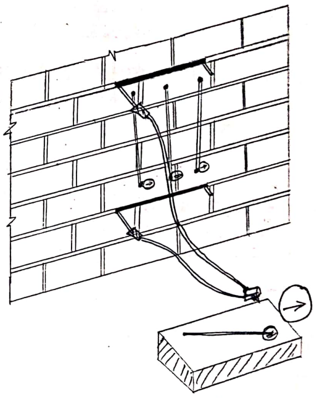


fig 10.8 Typical set-up for in-situ deformability.

Assumptions:

→ Masonary structures surrounding the slot is homogeneous
→ The stress applied to the masonary by flat-jack is uniform and the state of stress in test specimen is uniaxial i.e. a lateral constraining effect of adjacent masonary can be neglected.

ASTM recommendation for testing.

→ for the stress test reference points should be placed symmetrically on second courses (counting from the slot) above and below the slot.

→ At least four pair of equally spaced points should be placed, for each test

→ for the deformability test, the reference points are placed symmetrically in the masonary courses immediately above and below slots.

→ The value of pressure increment to be equal to 25% of estimated maximum flat jack pressure,

→ The time taken for the load application should be approximately same as the time required for making the cut and preparing the test.

Interpretation of test Results

For both the stress and deformability tests it is necessary to convert the flat-jack pressure to the actual compressive stress. The stress can be calculated by

$$\sigma_m = k_m k_a P \quad \text{------} \quad (10.5)$$

where,

k_m = calibration factor (< 1)

$$k_a = \frac{\text{measured area of flat-jack}}{\text{Avg. measured area of slot}} = (< 1)$$

p = flat-jack pressure.

3). Push shear Test

Push shear test is a minimally destructive technique that is used for in-situ measurement of masonary mortar joint strength index. This test uses a calibrated hydraulic ram and pressure gauge to measure the actual shear strength of a traditional brick wall, and thus its seismic resistance. A single brick beside the brick being tested is removed to accomodate the hydraulic ram (and is replaced in the wall after completion of test)