GRADING

Midterm (25%)

Quizzes (20%)

Term Project (15%)

Final examination (40%)

IMPORTANT NOTES

- Lab attendance is **compulsory**. Lab reports by of those who did not attend the lab session **will NOT be accepted**.
- Use of **programmable** calculators in the exams is **strictly forbidden**.
- Grading on a **CURVE** will be applied.
- Additional course documents can be provided from www.insaat.ege.edu.tr

REFERENCE BOOKS

1. Neville, A.M., PROPERTIES OF CONCRETE, Longman Group Limited, England, 1995.
2. Taylor, G.D., MATERIALS OF CONSTRUCTION, Construction Press, London and New York, 1983.
3. Neville, A.M. and Brooks, J.J., Concrete Technology, Longman Group, 1997, updated.
4. Erdoğan, T.Y., BETON, Metu Press, Ankara 2003.
5. Portland Cement Association, Course Material provided from Dr. Mike Thomas

CONTENTS

CONTENTS

CONTENTS

CONCRETE TECHNOLOGY

CONCRETE TECHNOLOGY

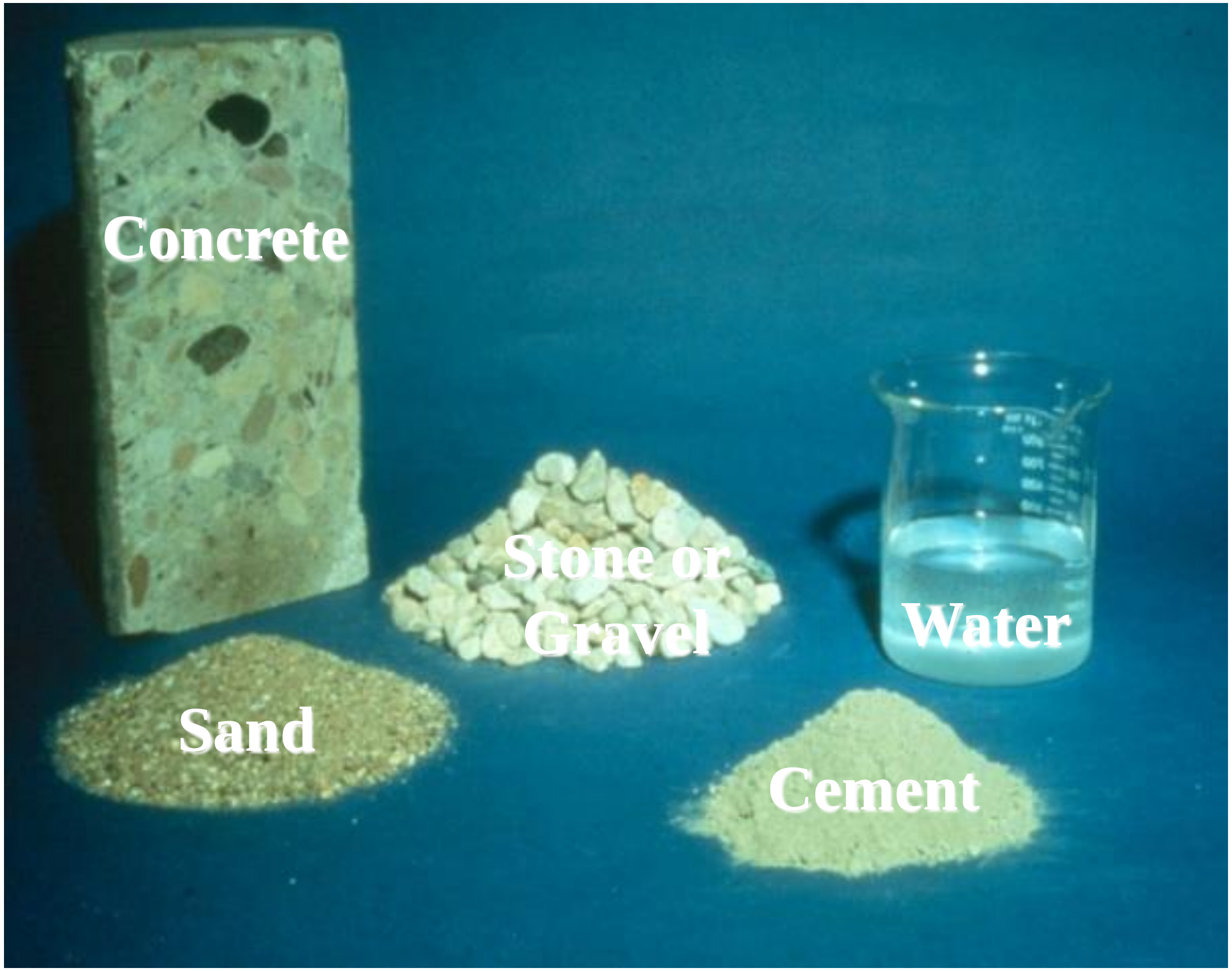
TES

Prof.Dr

Öz

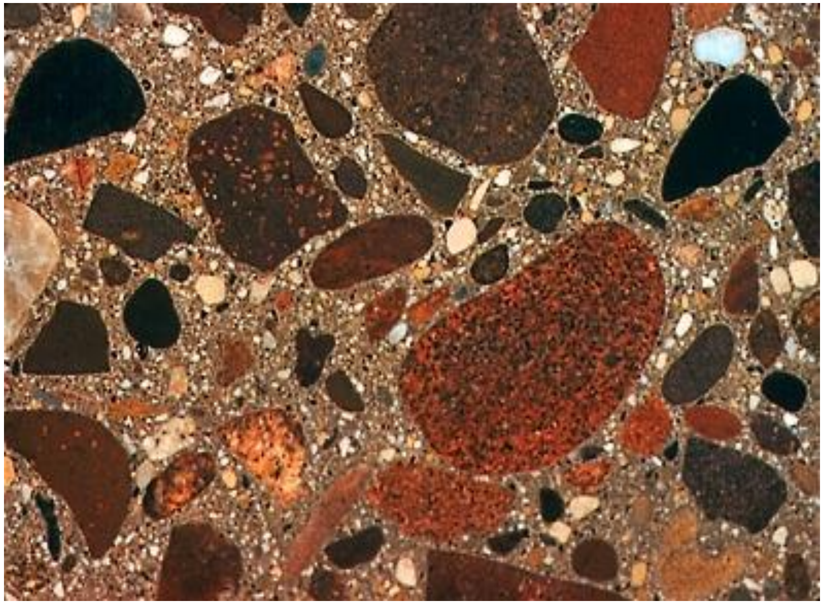
sist. Prof. Dr

Concrete Basics



PORTLAND CEMENT

Concrete Basics



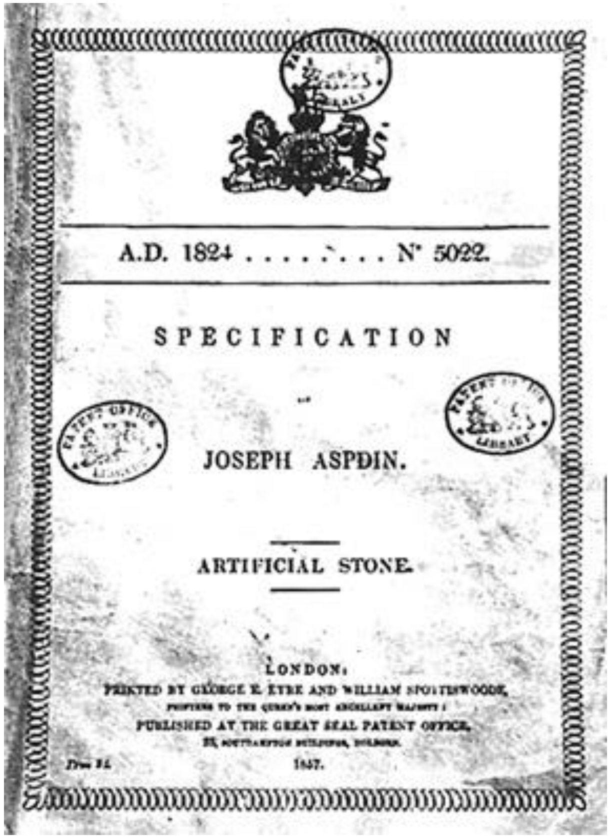
Concrete Basics

Cement is to concrete

… as flour is to cake!

Brief History of Cement

Joseph Aspdin’s patent for **portland cement**



A.D. 1824 . . . . N° 5022.

SPECIFICATION

JOSEPH ASPDIN.

ARTIFICIAL STONE.

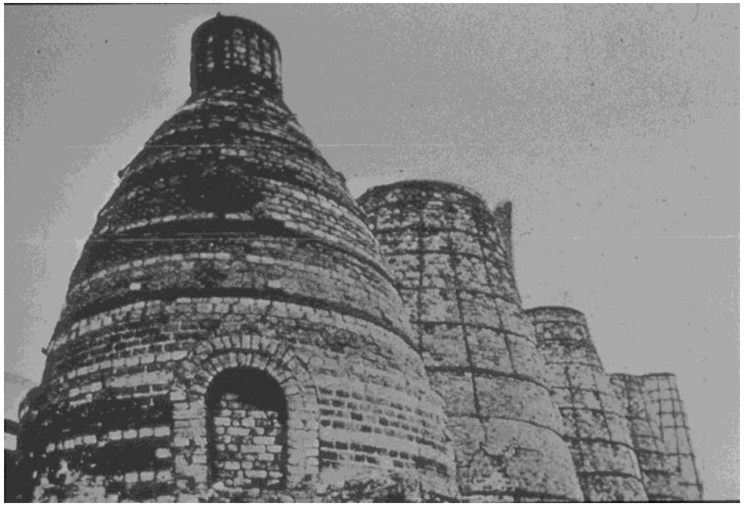
LONDON:
PRINTED BY GEORGE E. EYRE AND WILLIAM SPOTTISWOODE,
PRINTERS TO THE QUEEN'S MOST EXCELLENT MAJESTY:
PUBLISHED AT THE GREAT SEAL PATENT OFFICE,
25, SOUTHAMPTON BUILDINGS, HOLBORN.

Price 3d. 1857.

Brief History of Cement

Portland Cement

- Development of rotary kilns



Brief History of Cement

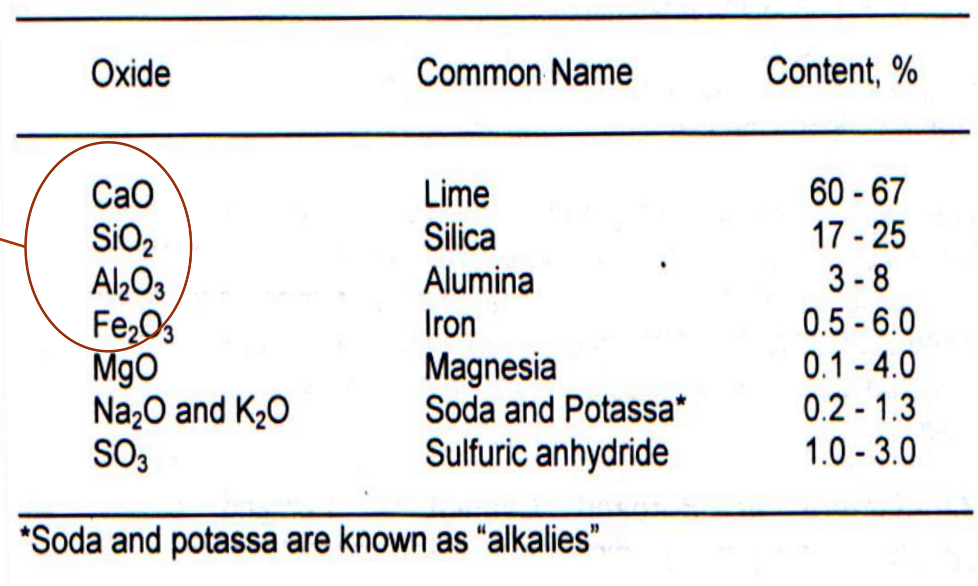
Developments since late 1800’s

- Development of rotary kilns
- Higher burning temperatures
- Introduction of ball mills
- Incorporation of gypsum to control set
- Introduction of dry process
- Use of precalciners
- Improved energy efficiency
- Improved quality control
- Wider range of cements – e.g. blended cements
- Reduced emissions

In addition to major oxides, PC contains small quantities of minor oxides among which *MgO, SO_3 (sulfur trioxide or sulfuric anhydrite) and alkalies (Na_2O and K_2O) are found in relatively higher proportions*. The term minor refers to the quantity of these oxides rather than their importance.

Cement will also show some mass loss when ignited in a furnace and this is due to small amounts of carbonate and water ions that have reacted with the cement – this is termed the loss-on-ignition or LOI and is typically less than 2%

MAJOR OXIDES > 90%



Oxide	Common Name	Content, %
CaO	Lime	60 - 67
SiO_2	Silica	17 - 25
Al_2O_3	Alumina	3 - 8
Fe_2O_3	Iron	0.5 - 6.0
MgO	Magnesia	0.1 - 4.0
Na_2O and K_2O	Soda and Potassa*	0.2 - 1.3
SO_3	Sulfuric anhydride	1.0 - 3.0

*Soda and potassa are known as "alkalies"

The major oxides play a role in the formation of major compounds which will be discussed later. *The minor oxides do not make a sufficient contribution to the strength,* they lead to harmful expansions in hardened cement paste and concrete.

Except for SO_3, all the oxides in portland cement are obtained by burning raw mix which consists of calcareous and clayey materials.

CaO → from limestone

SiO_2 and Al_2O_3 → from clay

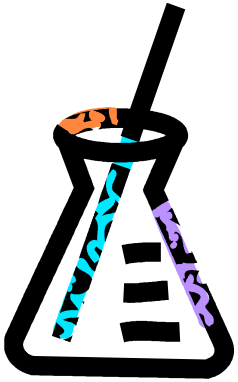
Fe_2O_3 → usually impurity in clay

SO_3 → mainly from the gypsum rock interground with clinker.

The amount of gypsum rock ($CaSO_4.2H_2O$) can be approximated by multiplying the amount of SO_3 by 2.15. (This value is 1.7 for $CaSO_4$)

$CaSO_4.2H_2O$: 172, SO_3: 80, $172/80 \cong 2.15$

Shorthand Notation



Oxide	Shorthand	Common Name
CaO	C	lime
SiO_2	S	silica
Al_2O_3	A	alumina
Fe_2O_3	F	ferric oxide
MgO	M	magnesia
K_2O	K	alkalis
Na_2O	N	
SO_3	$\bar{S}$	sulfate
CO_3	$\bar{C}$	carbonate
H_2O	H	water

+ other trace elements

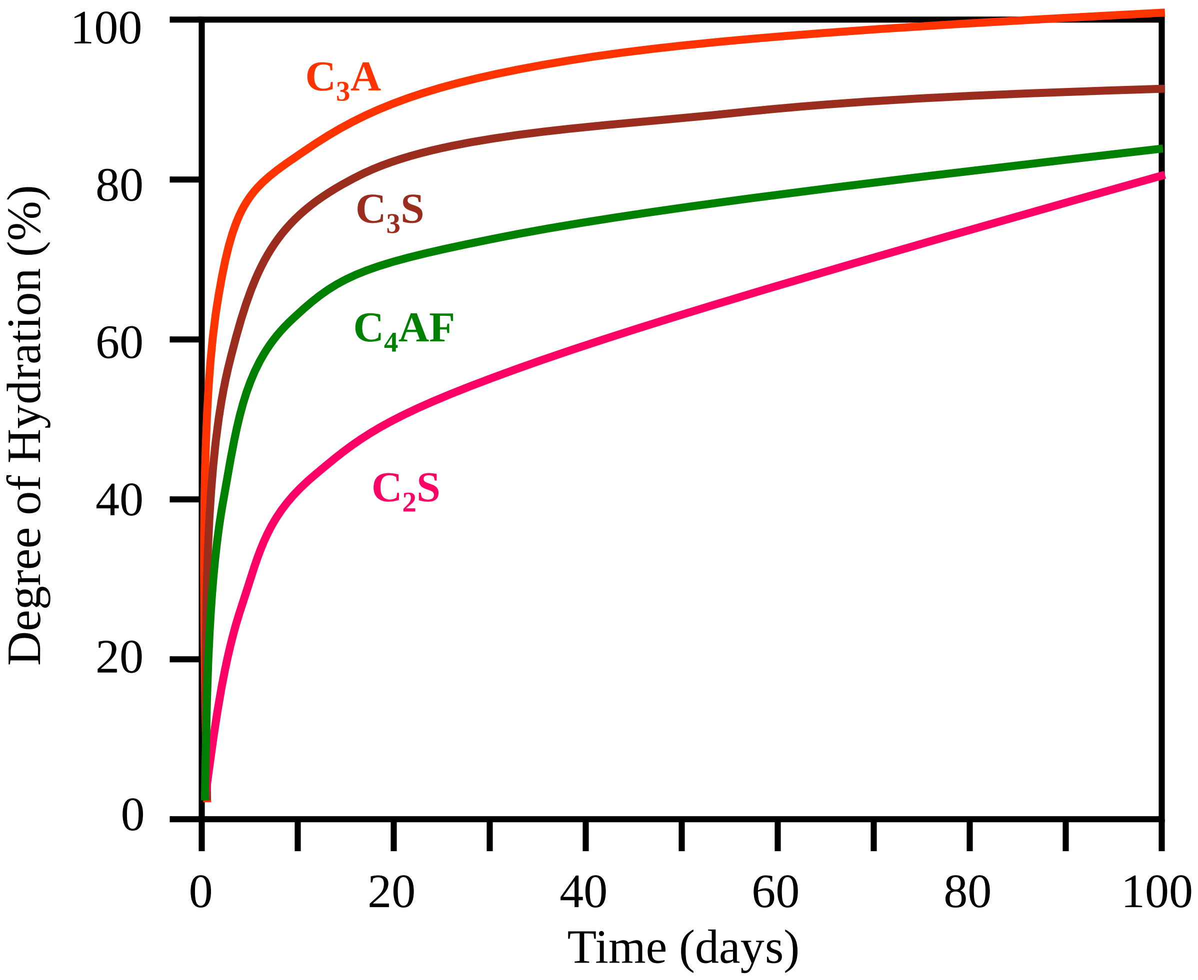
COMPOUND COMPOSITION OF PORTLAND CEMENT

The oxides interact with each other in the rotary kiln to form a series of complex compounds.

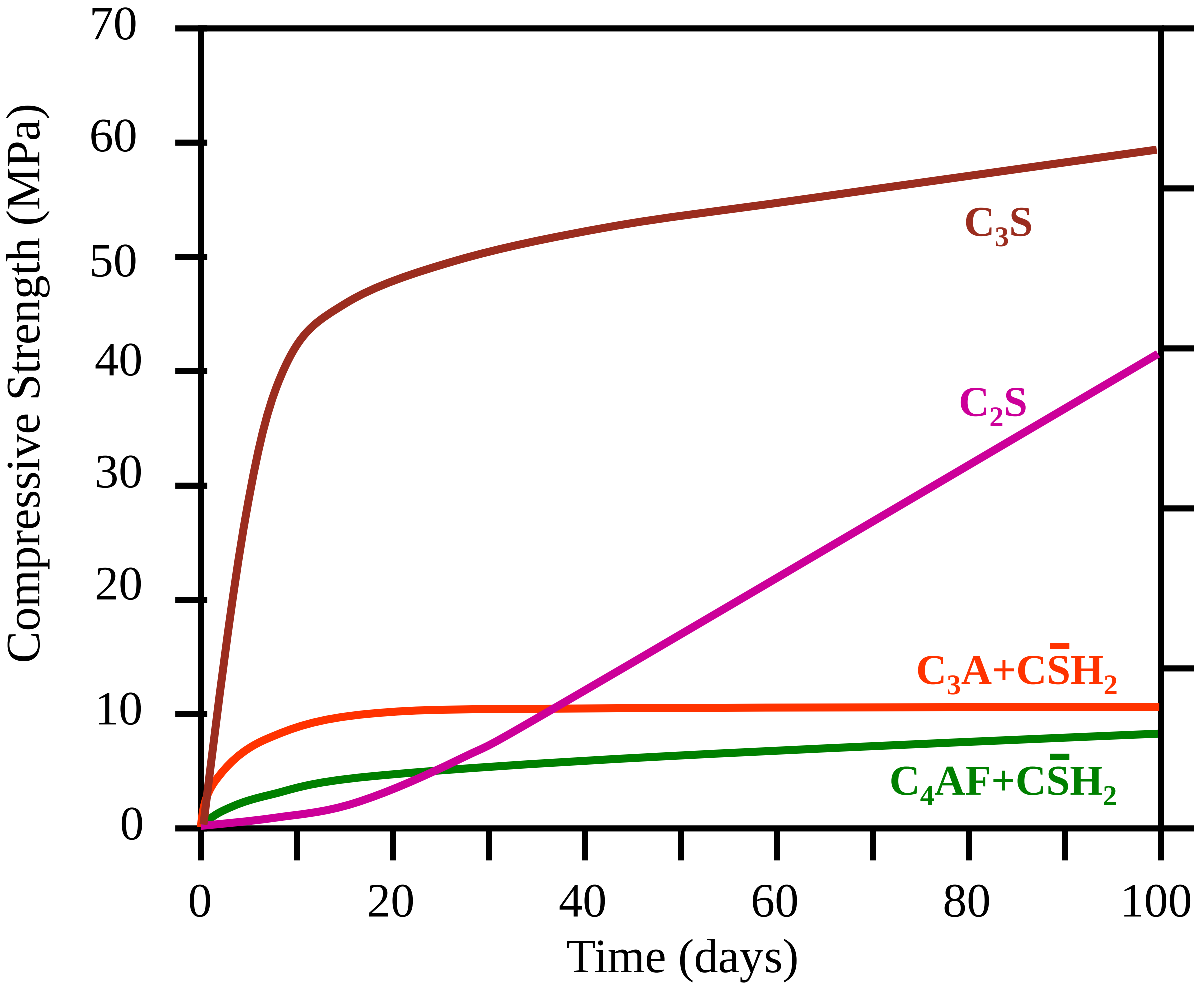
Portland cement clinker is usually regarded as constituted of four major compounds as shown below:

Chemical Name	Chemical Formula	Shorthand Notation	Mass (%)
Tricalcium silicate	$3CaO \bullet SiO_2$	C_3S	50 - 70
Dicalcium silicate	$2CaO \bullet SiO_2$	C_2S	15 - 30
Tricalcium aluminate	$3CaO \bullet Al_2O_3$	C_3A	5 - 10
Tetracalcium aluminoferrite	$4CaO \bullet Al_2O_3 \bullet Fe_2O_3$	C_4AF	5 - 15
Calcium sulfate dihydrate	$CaSO_4 \bullet 2H_2O$	CSH_2	~ 5

Hydration of Cement Compounds



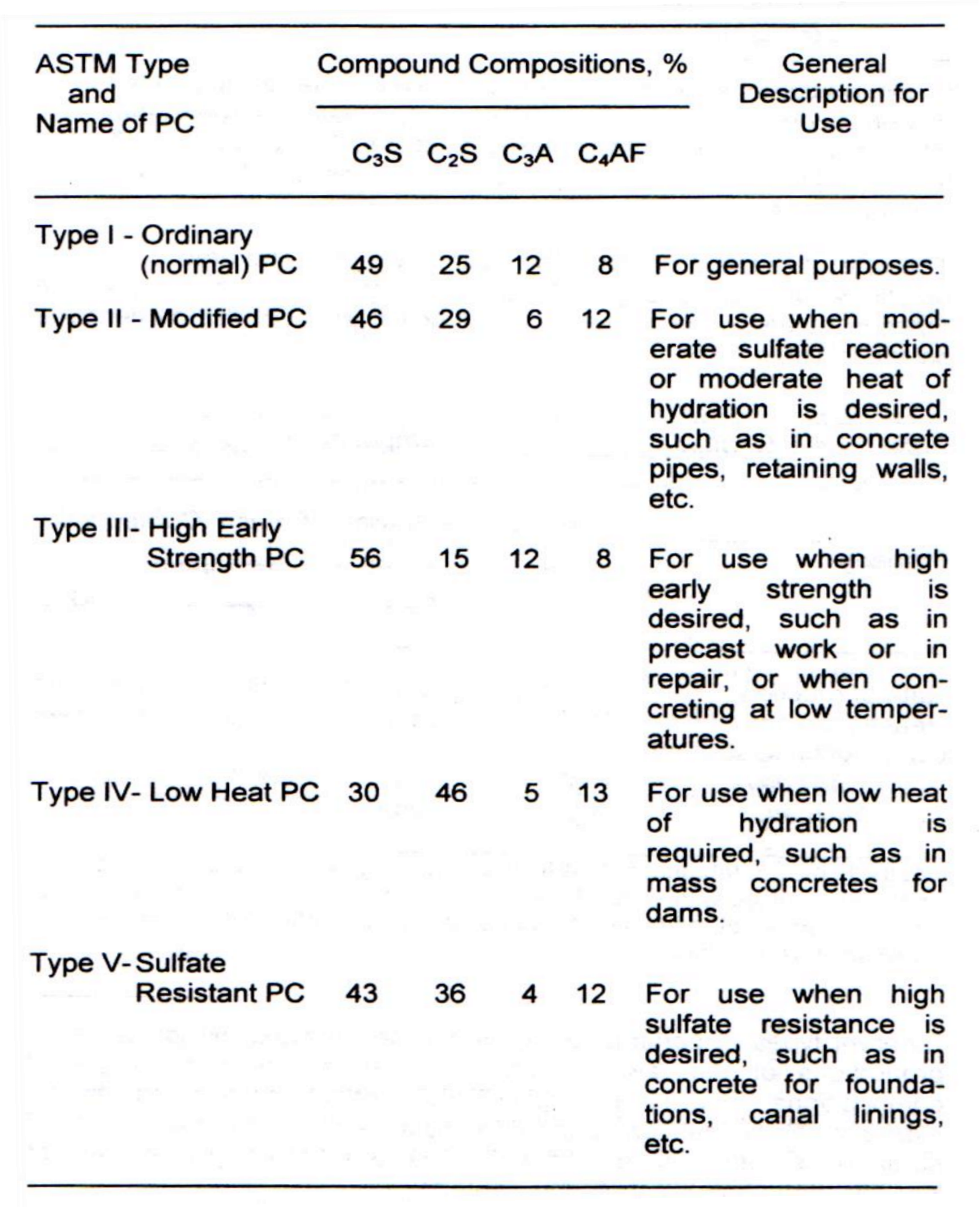
Hydration of Cement Compounds



Properties of Cement Compounds

C_3S	Tricalcium silicate (alite)	➢ Hydrates & hardens rapidly ➢ Responsible for initial set and early strength
C_2S	Dicalcium silicate (belite)	➢ Hydrates & hardens slowly ➢ Contributes to later age strength (beyond 7 days)
C_3A	Tricalcium aluminate	➢ Liberates a large amount of heat during first few days ➢ Contributes slightly to early strength development ➢ Cements with low percentages are more resistant to sulfates
C_4AF	Tetracalcium aluminoferrite (ferrite)	➢ Reduces clinkering temperature ➢ Hydrates rapidly but contributes little to strength ➢ Colour of hydrated cement (gray) due to ferrite hydrates

Different types of PCs can be obtained by adjusting the proportions of the raw materials to give the desired compound compositions. In order to explain this concept more clearly by an example, the typical compound compositions of the five main types of PCs given in ASTM specifications are presented:

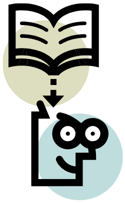


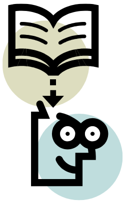
ASTM Type and Name of PC	Compound Compositions, %				General Description for Use
	C_3S	C_2S	C_3A	C_4AF	
Type I - Ordinary (normal) PC	49	25	12	8	For general purposes.
Type II - Modified PC	46	29	6	12	For use when moderate sulfate reaction or moderate heat of hydration is desired, such as in concrete pipes, retaining walls, etc.
Type III- High Early Strength PC	56	15	12	8	For use when high early strength is desired, such as in precast work or in repair, or when concreting at low temperatures.
Type IV- Low Heat PC	30	46	5	13	For use when low heat of hydration is required, such as in mass concretes for dams.
Type V- Sulfate Resistant PC	43	36	4	12	For use when high sulfate resistance is desired, such as in concrete for foundations, canal linings, etc.

THE TYPES OF CEMENTS PRODUCED IN TURKEY-CEN Cements

Types of cements, TS EN 197-1 covers 27 standard cements classified into five main types:

CEM I Portland cement (ordinary)
CEM II Portland composite cement
CEM III Blast furnace slag cement
CEM IV Pozzolanic cement
CEM V Composite cement





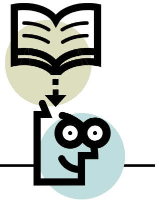
- Portland Cement Clinker (K)
- Granulated blast furnace slag (S)
- Pozzolanic Materials (P, Q)
  - Natural pozzolana (P)
  - Natural calcined pozzolana (Q)
- Fly ashes (V, W)
  - Siliceous fly ash (V) reactive CaO<10% by mass
  - Calcareous fly ash (W) reactive CaO>10% by mass

- Burnt shale (T)
- Limestone (L, LL)
  - LL: total organic matter < 0.20 % by mass
  - L: total organic matter< 0.50 % by mass
- Silica fume (D)
- Minor additional constituents (<5% by mass)

- Types of CEN cements

Main type	Name	Symbol	
CEM I	Portland cement	CEM I	
CEM II	Portland blast furnace slag cement	CEM II/A-S CEM II/B-S	Blast furnace slag
	Portland silica fume cement	CEM II/A-D	Silica fume
	Portland pozzolan cement	CEM II/A-P CEM II/B-P	Natural pozzolan
		CEM II/A-Q CEM II/B-Q	Calcined pozzolan
	Portland fly ash cement	CEM II/A-V CEM II/B-V	Low lime fly ash
		CEM II/A-W CEM II/B-W	High lime fly ash
	Portland burned schist cement	CEM II/A-T CEM II/B-T	Burned schist
	Portland limestone cement	CEM II/A-L CEM II/B-L	organic material content<0.5%
		CEM II/A-LL CEM II/B-LL	organic material content<0.2%
	Portland composite cement	CEM II/A-M CEM II/B-M	

Types of CEN cements

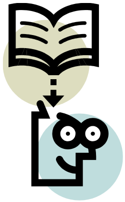


Main type	Name	Symbol
CEM III	Blast furnace slag cement	CEM III/A CEM III/B CEM III/C
CEM IV	Pozzolanic cement	CEM IV/A CEM IV/B
CEM V	Composite cement	CEM V/A CEM V/B

In addition to TS EN 197, TS 21 covers “ White portland cement”, TS 22-1 or 2 EN 413-1 or 2 cover “Masonry cement Part 1: “Properties” or Part 2:”Test methods”.

The Composition of TS EN 197-1 Cemen

Cement type	**clinker**	**S**	**D**	**P**	**Q**	**V**	**W**	**T**	**L-LL**	**Minor**
CEM I	95-100									0-5
CEM II/A-S	80-94	6-20								0-5
CEM II/B-S	65-79	21-35								0-5
CEM II/A-D	90-94		6-10							0-5
CEM II/A-P	80-94			6-20						0-5
CEM II/B-P	65-79			21-35						0-5
CEM II/A-Q	80-94				6-20					0-5
CEM II/B-Q	65-79				21-35					0-5



The Composition of TS EN 197-1 Cements

Cement type	clinker	S	D	P	Q	V	W	T	L-LL	Minor
CEM II/A-V	80-94					6-20				0-5
CEM II/B-V	65-79					21-35				0-5
CEM II/A-W	80-94						6-20			0-5
CEM II/B-W	65-79						21-35			0-5
CEM II/A-T	80-94							6-20		0-5
CEM II/B-T	65-79							21-35		0-5
CEM II/A-L (LL)	80-94								6-20	0-5
CEM II/B-L (LL)	65-79								21-35	0-5

The Composition of TS EN 197-1 Cements

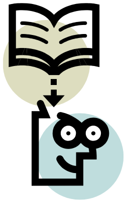
Cement type	clinker	S	D	P	Q	V	W	T	L-LL	Minor
CEM II/A-M	80-94	← 6-20 →								0-5
CEM II/B-M	65-79	← 21-35 →								0-5
CEM III/A	35-64	36-65								0-5
CEM III/B	20-34	66-80								0-5
CEM III/C	5-19	81-95								0-5
CEM IV/A	65-89		← 11-35 →							0-5
CEM IV/B	45-64		← 36-55 →							0-5
CEM V/A	40-64	18-30		← 18-30 →						0-5
CEM V/B	20-38	31-50		← 31-50 →						0-5

Example 1: Portland cement conforming to EN 197-1 of strength class 42.5 with a high early strength is identified by;

Portland cement: CEM I 42.5R

Example 2: Portland-limestone cement containing between 6% and 20% by mass of limestone with a total organic matter content not exceeding 0.50% by mass (L) of strength class 32.5 with an ordinary early strength is identified by;

Portland limestone cement: CEM II A – L 32.5N

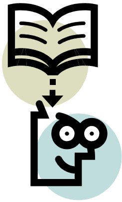


Example 3: Portland-composite cement containing in a total quantity of granulated blast furnace slag (S), siliceous fly ash (V) and limestone (L) of between 6% and 20% by mass and strength class 32.5 with a high early strength is identified by;

Portland-composite cement: CEM II/A-M(S-V-L) 32.5R

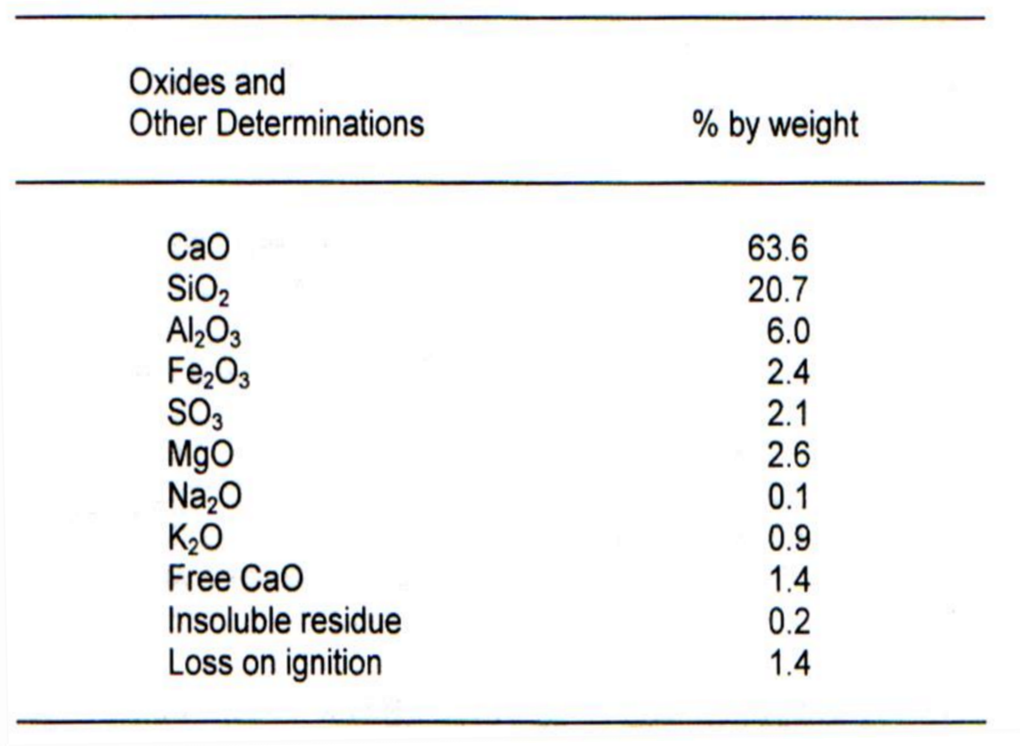
Example 4: Composite cement containing between 18% and 30% by mass of granulated blast furnace slag (S) and between 18% and 30% by mass of siliceous fly ash (V) of strength class 32.5 with an ordinary early strength is identified by;

Composite cement: CEM V/A (S-V) 32.5 N



The amounts of oxides presenting in PC depend on the proportioning of raw materials and how well the burning is done.

The oxide composition of PC is determined by chemical analysis.



Oxides and Other Determinations	% by weight
CaO	63.6
SiO_2	20.7
Al_2O_3	6.0
Fe_2O_3	2.4
SO_3	2.1
MgO	2.6
Na_2O	0.1
K_2O	0.9
Free CaO	1.4
Insoluble residue	0.2
Loss on ignition	1.4

As it was mentioned earlier, the amount of SO_3 indicates the amount of gypsum present in the cement. Gypsum retards the very quick setting (flash setting) of PC. On the other hand, if too much gypsum is added, it leads to disruptive expansion of hardened paste or concrete.. ASTM allows a maximum of 3% $S0_3$ in ordinary portland cement.

High content of MgO and free lime cause expansion upon slaking. The reaction of these oxides with water in the cement are rather slow since they are in a hard-burned state. Therefore, the reactions usually take place after the cement is set or hardened. Expansion in a hardened paste or concrete leads to cracking. ASTM allows 5% MgO and 3% free lime in ordinary portland cement.

The presence of alkalies generally does not cause any problem except when the cement is used with certain types of reactive aggregates in making concrete. The alkalies (in the form of hydroxides) can react with reactive silica of the aggregate and result in the formation of alkali-silica gel which can absorb water and swell a lot. The formation of this reaction may take months or years. Therefore, the expansion in hardened concrete causes disintegration and cracking. Standards limit the total alkali content ($\%Na_2O+0.658\%K_2O=$equivalent Na_2O) to 0.6% when the cement is to be used with reactive aggregates.

IR is that fraction of the cement which is not soluble in HCl; it derives mainly from the silica which has not reacted to form compounds during the burning process in the kiln. Thus, the IR is a measure of completeness of the reactions in the kiln. (ASTM limits IR to max. 0.75% in PC).

LOI is the loss in weight of cement after being heated to 1000 °C. Water and CO_2 are the substances expelled from cement during heating. Water may be due to taking moisture from atmosphere by cement particles (prehydration). CO_2 may be present in cement when free CaO and MgO absorb moisture from the air, slake, and then react with the CO_2 of the air to form $CaCO_3$ or $MgCO_3$ (precarbonation). *Prehydration and precarbonation are due to improper or prolonged storage of cement. Both prehydration and carbonation reduce the binding potential of cement. (ASTM allows a maximum of 3% LOI for OPC).*

1	2	3	4	5
Property	**Test reference**	**Cement type**	**Strength class**	**Requirements** [a]
Loss on ignition	EN 196-2	CEM I CEM III	all	≤ 5,0 %
Insoluble residue	EN 196-2 [b]	CEM I CEM III	all	≤ 5,0 %
Sulfate content (as SO_3)	EN 196-2	CEM I CEM II [c] CEM IV CEM V	32,5 N 32,5 R 42,5 N	≤ 3,5 %
			42,5 R 52,5 N 52,5 R	≤ 4,0 %
		CEM III [d]	all	
Chloride content	EN 196-21	all [e]	all	≤ 0,10 % [f]
Pozzolanicity	EN 196-5	CEM IV	all	Satisfies the test

a Requirements are given as percentage by mass of the final cement.

b Determination of residue insoluble in hydrochloric acid and sodium carbonate.

c Cement type CEM II/B-T may contain up to 4,5 % sulfate for all strength classes.

d Cement type CEM III/C may contain up to 4,5 % sulfate.

e Cement type CEM III may contain more than 0,10 % chloride but in that case the maximum chloride content shall be stated on the packaging and/or the delivery note.

f For pre-stressing applications cements may be produced according to a lower requirement. If so, the value of 0,10 % shall be replaced by this lower value which shall be stated in the delivery note.

Homework – Remember Bogue's calculations from Materials of Construction course

Two portland cements with the following oxide compositions are available. Compare the cements from following points of view:

	%	
Oxide	A	B
CaO	62.8	61.8
SiO_2	20.4	21.4
Al_2O_3	5.2	5.2
Fe_2O_3	3.0	3.0
MgO	4.0	4.0
SO_3	2.5	2.5

A) rate of hydration

B) heat of hydration

C) early cementitious value

D) ultimate cementitious value

The setting and hardening of concrete are the result of some chemical and physical processes that take place between cement and water.

Hydration of Pure Cement Compounds

The chemical reactions describing the hydration of cement are usually explained by considering the hydration of individual compounds.

It is assumed that the hydration of each compound takes place independently of the others that are present in portland cement.

This assumption is not completely valid, since interactions between hydrating compounds can have important consequences.

The hydration reactions of the two calcium silicates namely, C_3S (tricalcium silicate) and C_2S (dicalcium silicate), are stoichiometrically* very similar, differing only in the amount of CH (calcium hydroxide) formed.

$$2C_3S + 6H \rightarrow C_3S_2H_3 + 3CH$$

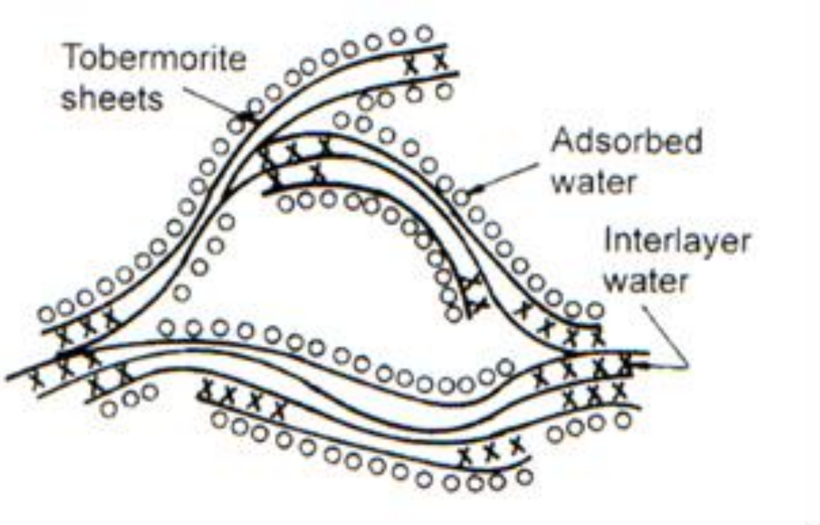
$$2C_2S + 4H \rightarrow C_3S_2H_3 + CH$$

The principal hydration product is a calcium silicate hydrate (C-S-H). The formula $C_3S_2H_3$ is only approximate because the composition of this hydrate is actually variable over quite a wide range. It is a poorly crystalline material which forms extremely small particles in the size of ~l µm. In contrast, calcium hydroxide is a crystalline material with fixed composition.

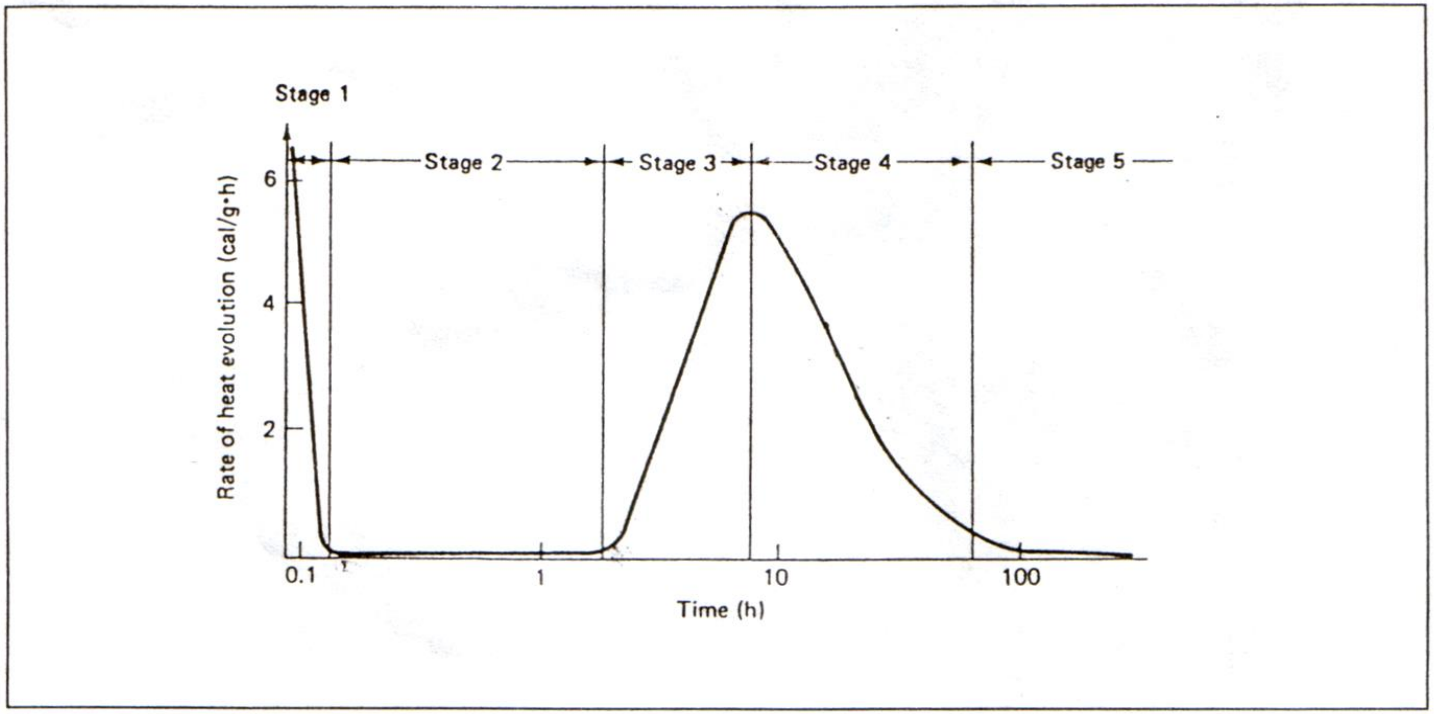
* Stoichiometry: determination of exact proportions of elements to make pure chemical compounds

Some of the water that occurs in the structure of C-S-H is held by the surface forces of the C-S-H particles as adsorbed water, and some of the water is held between the surfaces of certain planes in the crystal as interlayer water.

C_3S and C_2S control most strength giving properties. CH does not contribute very much to the strength of PC. C-S-H gels are the major products of hydration that provide the binding characteristics of cements.



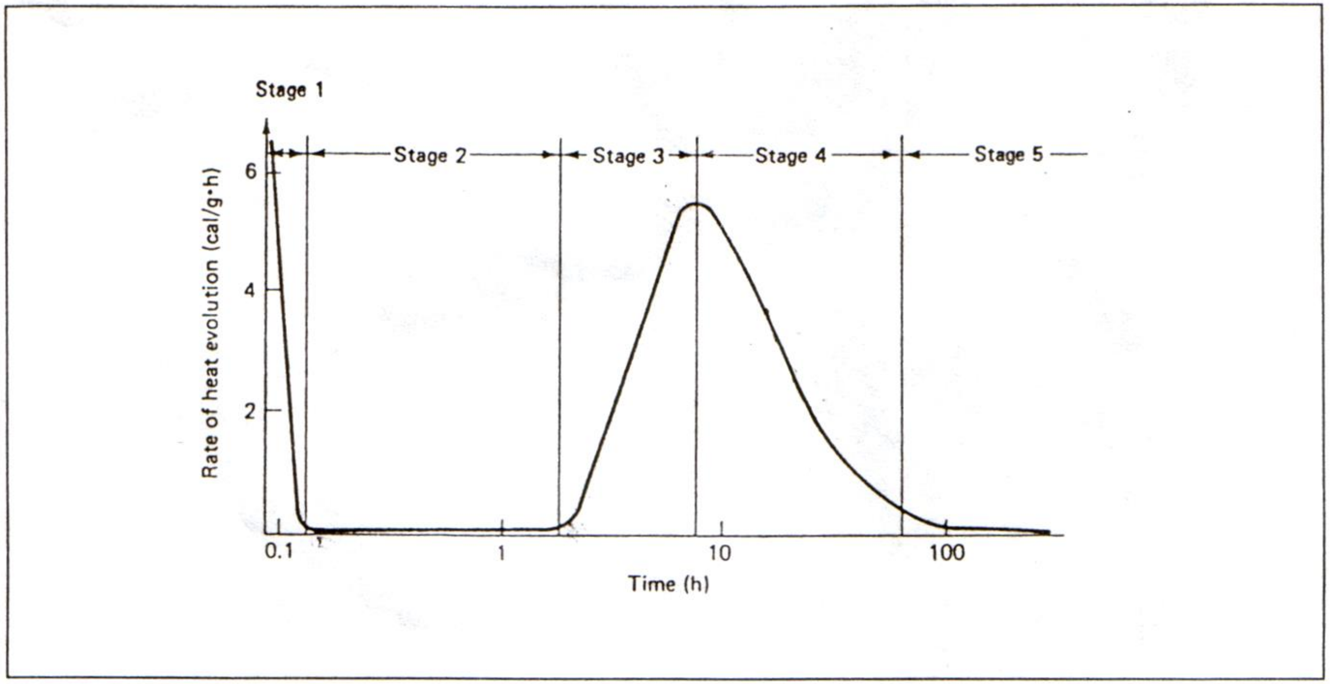
Calorimetric curve of C_3S hydration



Stage 1: When first mixed with water, a period of rapid evolution of heat occurs, which stops within about 15 minutes.

Stage 2: There follows a period of relative inactivity, the dormant period which is the reason why portland cement concrete remains in the plastic state for several hours. Initial set occurs in 2 to 4 hours, about the time C_3S has begun to react again.

Calorimetric curve of C_3S hydration (ctd.)



Stage 3: The silicate continues to hydrate rapidly, reaching a maximum rate at the end of the acceleration period. By this time final set has been passed and early hardening has begun.

Stage 4: The rate of reaction again slows down.

Stage 5: Then it reaches a steady state within 12 to 24 hours.

At each stage: On first contact with water, calcium ions and hydroxyl ions are rapidly released from the surface of C_3S grain; the pH rises to over 12 within a few minutes, which indicates a very alkaline solution. This continues very slowly in the dormant period. When the calcium and hydroxyl ion concentrations reach a critical value, CH and C-S-H start to crystallize from solution and the reaction of C_3S again proceeds rapidly. C-S-H develops at the surface of the C_3S grain and forms a coating.

As hydration continues, the thickness of this layer increases and forms a barrier through which ions must diffuse to reach the growing crystals.

Eventually, movement through the C-S-H layer determines the rate of reaction. It becomes slower as the thickness of the diffusion barrier increases. Thus, hydration tends to approach completeness asymptotically.

C_2S hydrates in a similar manner, but it is much slower because it is a less reactive compound than C_3S.

Tricalcium Aluminate

The reaction of C_3A with water is very rapid leading to the formation of C-A-Hs. The rapid hydration of C_3A causes immediate stiffening of the cement accompanied by a large amount of heat liberation, called flash set. In order to prevent flash set, a small amount of $C\overline{S}H_2$ is added to the clinker during the grinding process.

In portland cement the hydration of C_3A involves reactions with sulfate ions which are supplied by the dissolution of gypsum. The primary initial reaction of C_3A is:

$$C_3A + 3C\overline{S}H_2 + 26H \rightarrow C_6A\overline{S}_3H_{32}$$

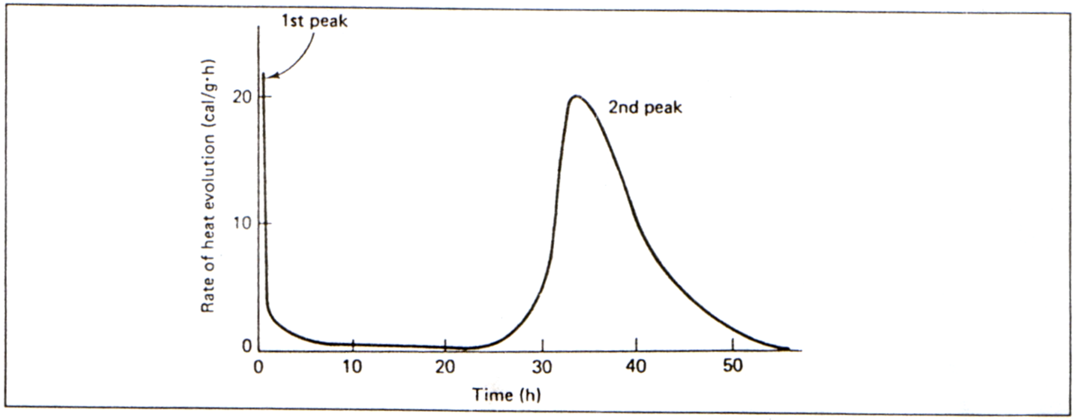
The calcium sulfoalumino hydrate obtained as a result of the above reaction is commonly called *ettringite*.

If the sulfate ions are all consumed before the C_3A has completely reacted, ettringite transforms to another calcium sulfoaluminate:

$$2C_3A + C_6A\overline{S}_3H_{32} + 4H \rightarrow 3C_4A\overline{S}H_{12}$$

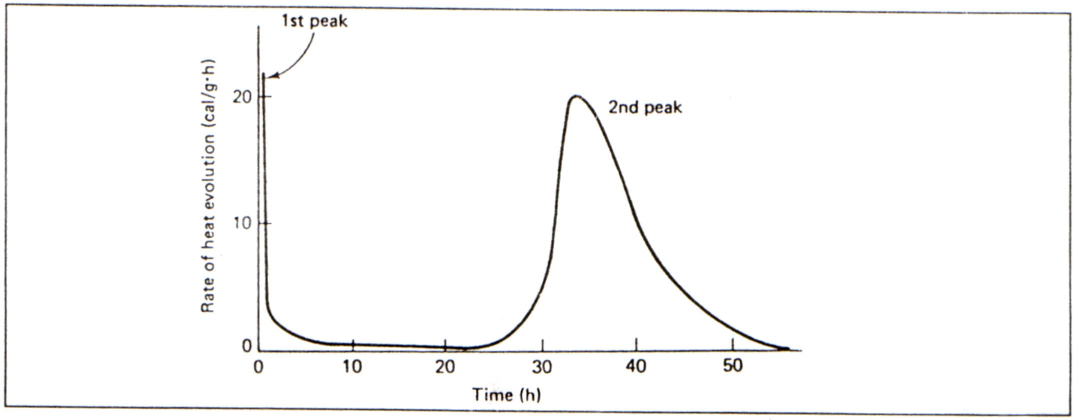
This second product is called *monosulfoaluminate*.

Calorimetric curve of C_3A hydration



Both steps in hydration of C_3A are exothermic. The formation of ettringite slows down the hydration of C_3A by creating a diffusion barrier around C_3A, analogous to the behavior of C-S-H during the hydration of calcium silicates. This barrier breaks down during the conversion to monosulfoaluminate and allows C_3A to react rapidly again. The calorimeter curve for C_3A hydration looks like that of C_3S although the reactions are much different and the amount of heat evolved is much greater.

Calorimetric curve of C_3A hydration (ctd.)



The first heat peak is completed in 10 to 15 minutes but the time necessary for the second peak to occur depends on the amount of sulfate available in the system. The more gypsum there is in the system, the longer the ettringite will remain stable. In most cements, the conversion of ettringite to monosulfoaluminate takes 12 to 36 hours, after all the gypsum has been used to form ettringite.

C_3A hydration (contd.)

The formation of monosulfoaluminate occurs because of the deficiency of sulfate ions to form ettringite from all the available aluminate ions. When monosulfoaluminate contacts with a new source of sulfate ions, the ettringite can be formed once again.

$$C_4A\overline{S}H_{12} + 2C\overline{S}H_2 + 16H \rightarrow C_6A\overline{S}_3H_{32}$$

This potential for reforming ettringite is the basis for sulfate attack on portland cements when exposed to an external supply of sulfate ions.

Thus, if no or very little gypsum is used in the production of cement, flash setting occurs; if too much gypsum is available, ettringite formation increases, resulting in expansion and cracking of the hardened cement paste or concrete.

Ferrite Phase (C_4AF)

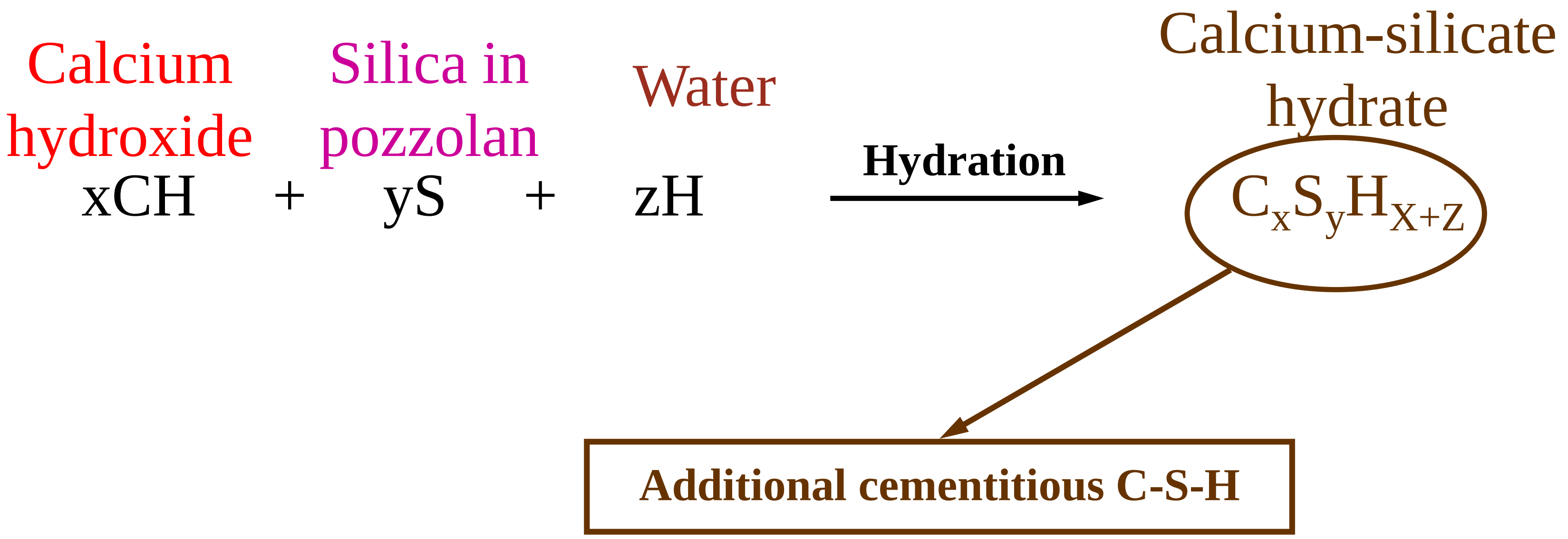
C_4AF hydration results in the formation of iron analogs of C_3A hydration products. The reactions are much slower and involve much less heat evolution.

$$C_4AF + 3C\bar{S}H_2 + 21H \rightarrow C_6(A, F)\bar{S}_3H_{32} + (A, F)H_3$$

$$C_4AF + C_6(A, F)\bar{S}_3H_{32} + 7H \rightarrow 3C_4(A, F)\bar{S}H_{12} + (A, F)H_3$$

Pozzolanic Reaction

Reaction of silica in pozzolan with calcium hydroxide:



Calcium hydroxide | Silica in pozzolan | Water | Calcium-silicate hydrate

$$xCH + yS + zH \xrightarrow{\textbf{Hydration}} \boxed{C_xS_yH_{X+Z}}$$

Additional cementitious C-S-H

In alumino-siliceous pozzolans (e.g. fly ash and metakaolin) the alumina also participates in reactions with calcium hydroxide producing various calcium-aluminate hydrates (C-A-H) and calcium-alumino-silicate hydrates (C-A-S-H).

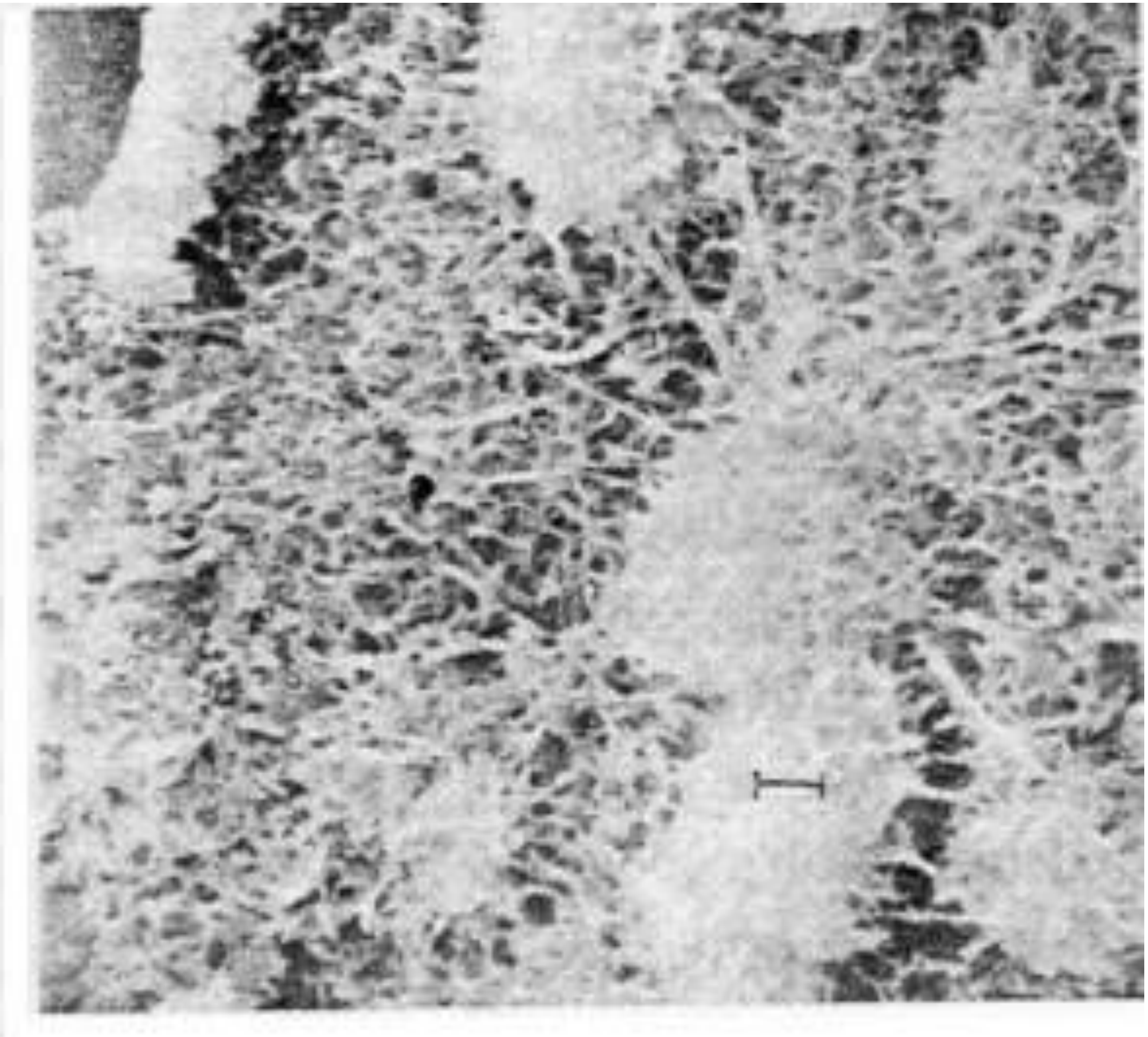
Hydration of Portland Cement - *Kinetics*

The rate of hydration during the first few days is in the approximate order

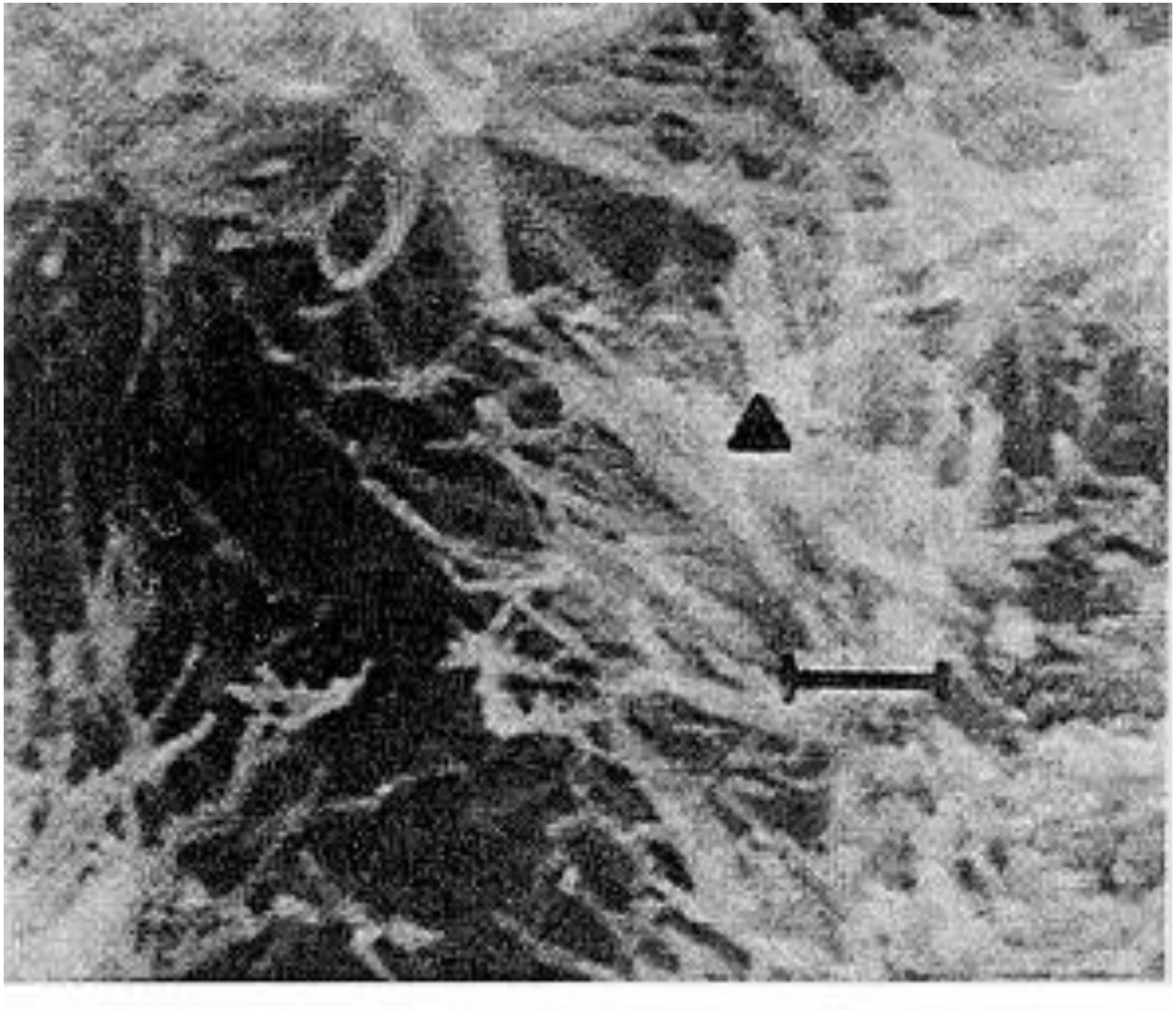
$C_3A > C_3S > C_4AF > C_2S$

It must be remembered however that the reaction rates of these compounds are not exactly the same in different cements because their reactivities will be affected by the fineness, rate of cooling of the clinker, presence of impurities, and the presence of other cement compounds.

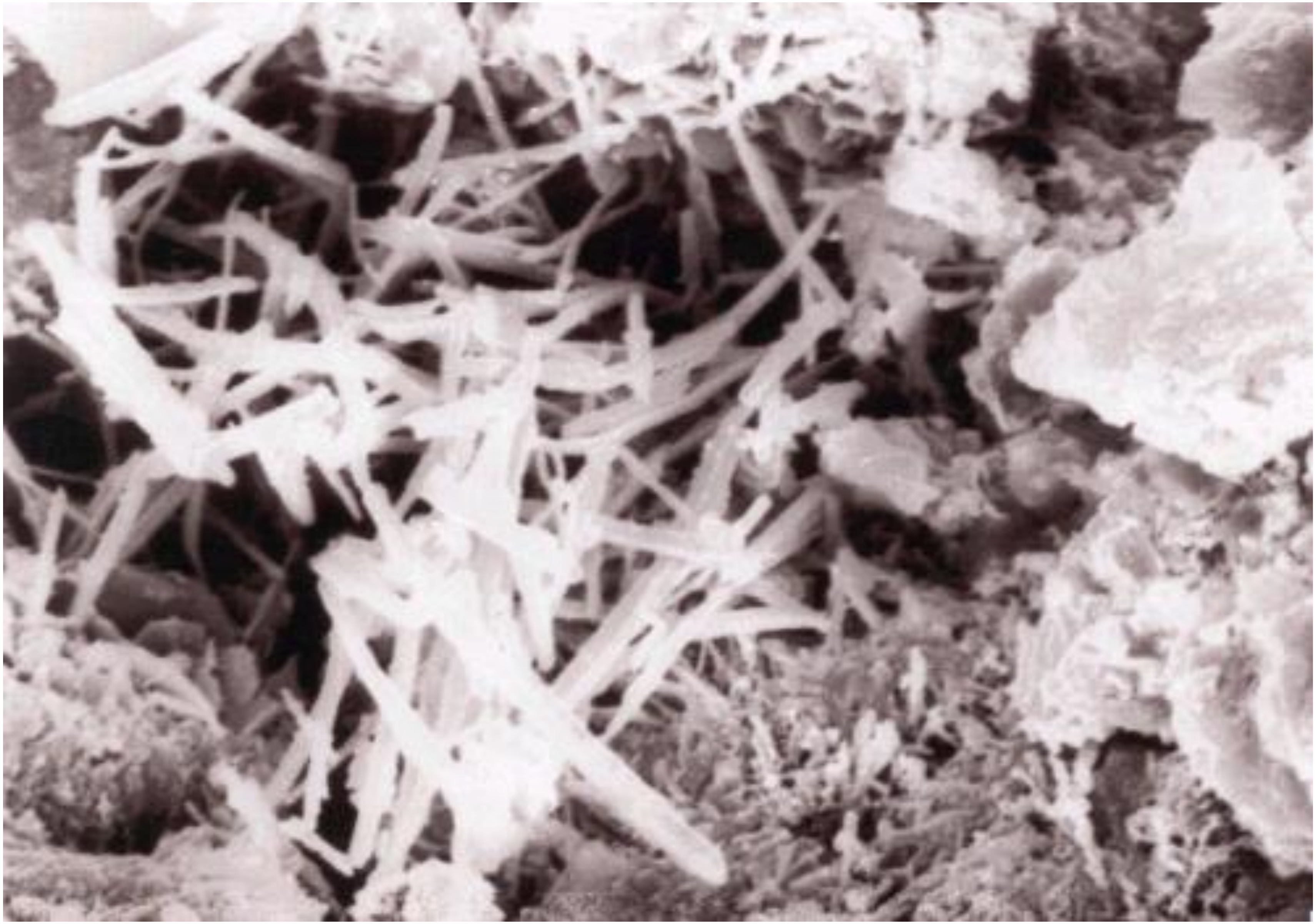
İMENTO HARCI İÇ YAPISI



ÇİMENTO HARCI İÇ YAPISI

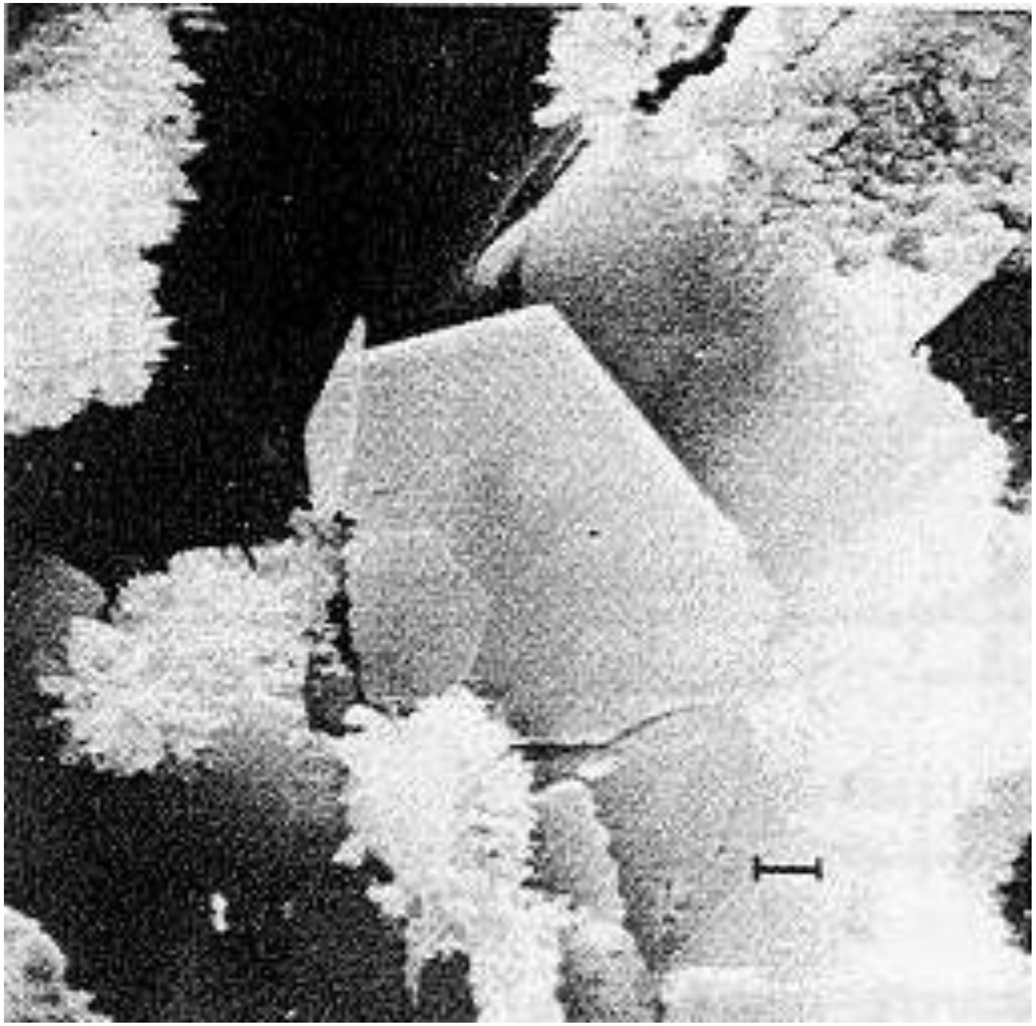


ÇİMENTO HARCI İÇ YAPISI



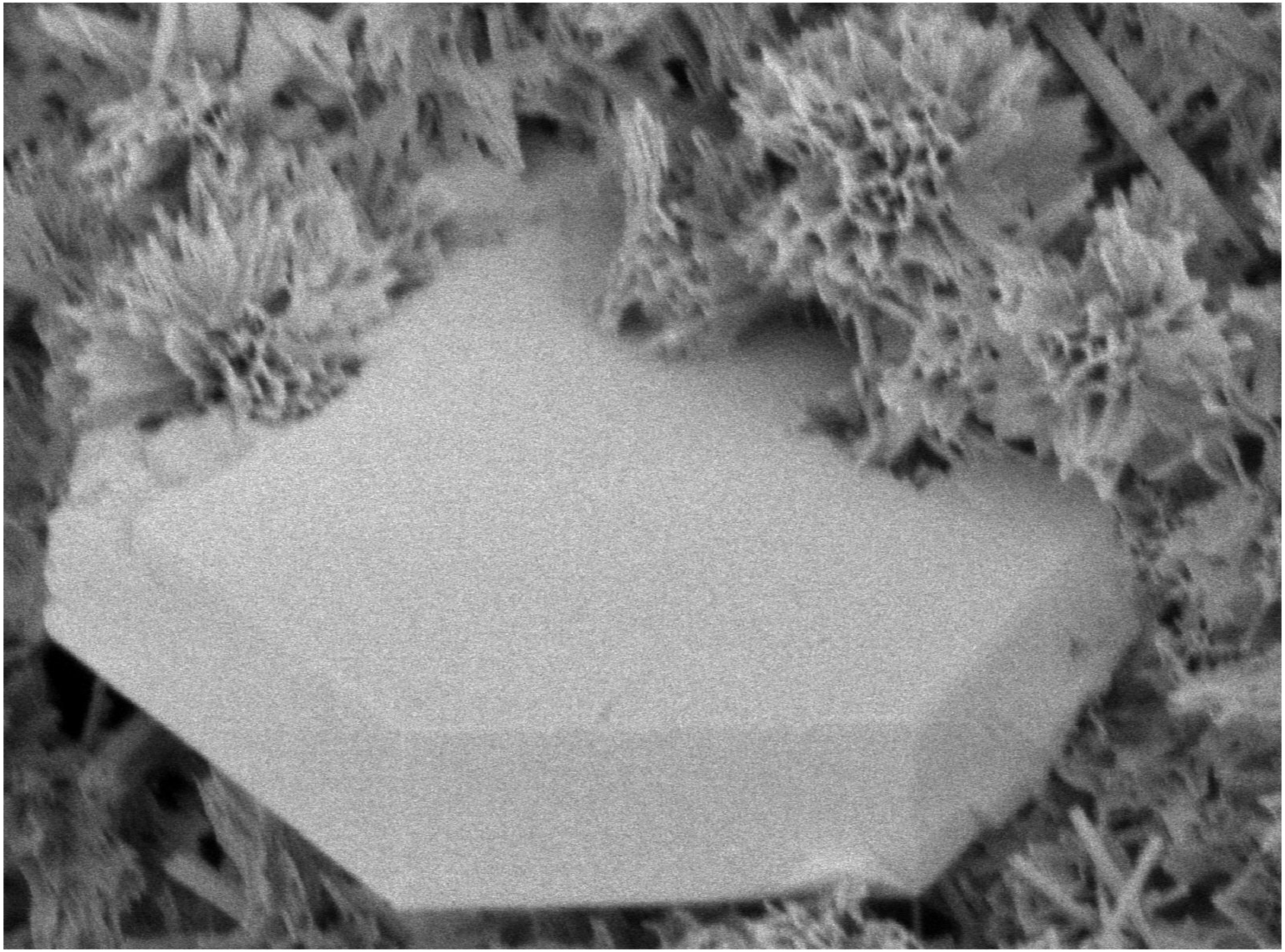
ÇİMENTO HARCI İÇ YAPISI

$Ca(OH)_2$ ve CSH kristalleri (S/Ç=0.6 24ºC'de 5 gün hidrate olmuş)



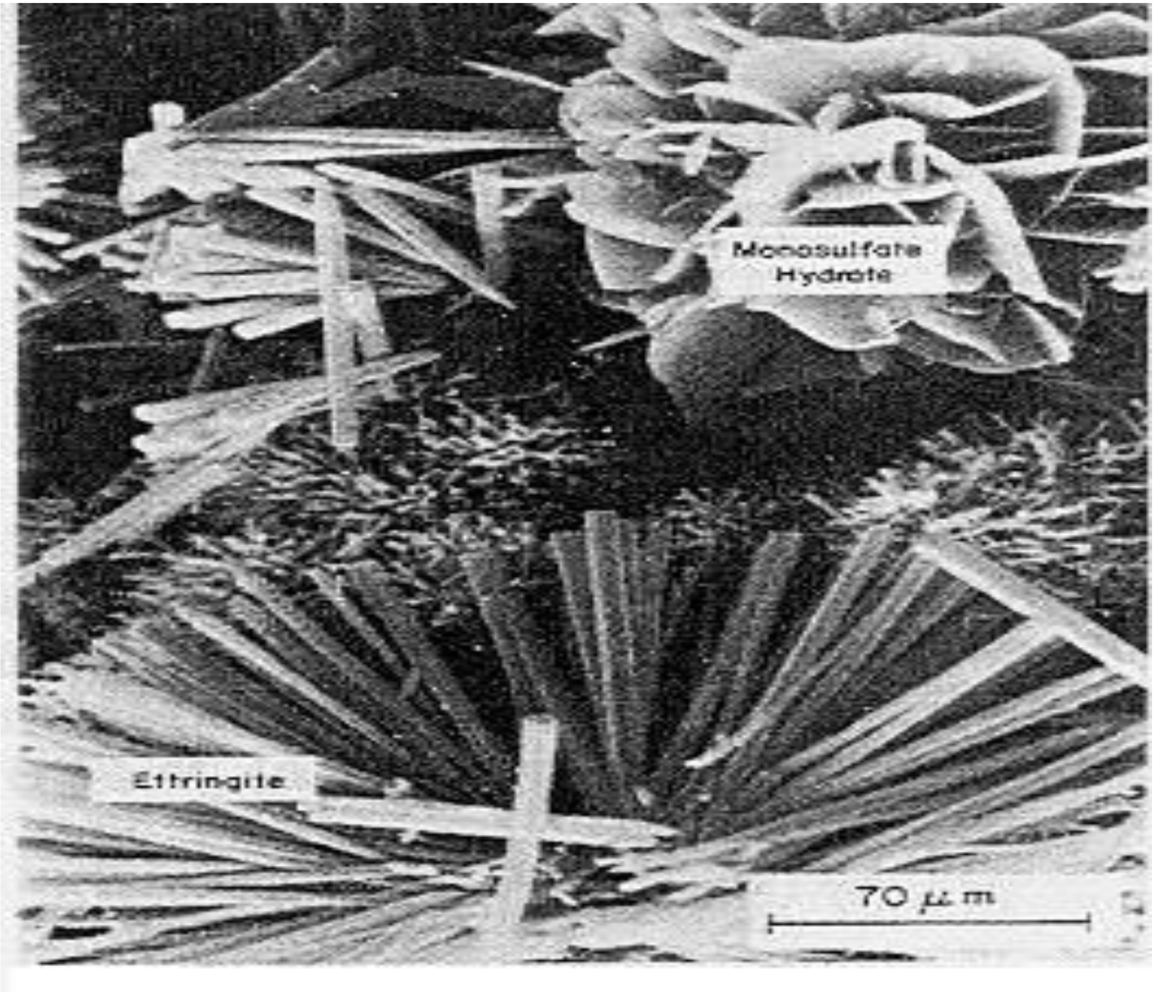
ÇİMENTO HARCI İÇ YAPISI

$Ca(OH)_2$



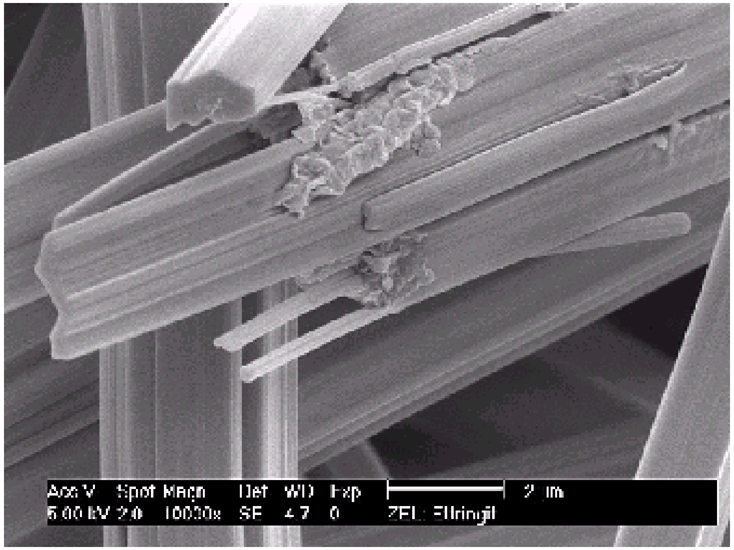
ÇİMENTO HARCI İÇ YAPISI

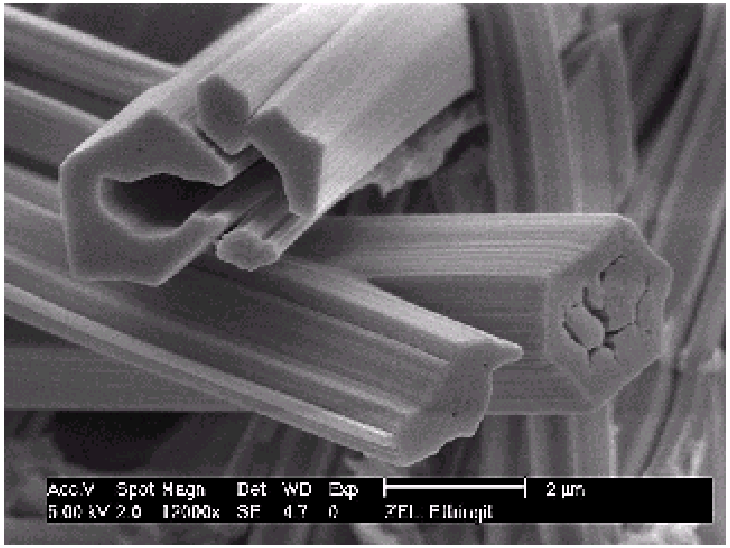
Monosülfat Alimünat ve etrenjit'in hekzegonal kristal yapısı

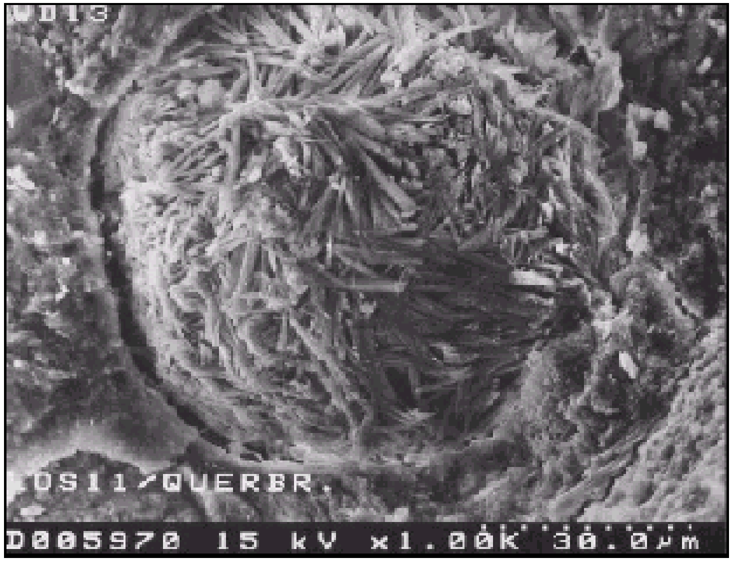


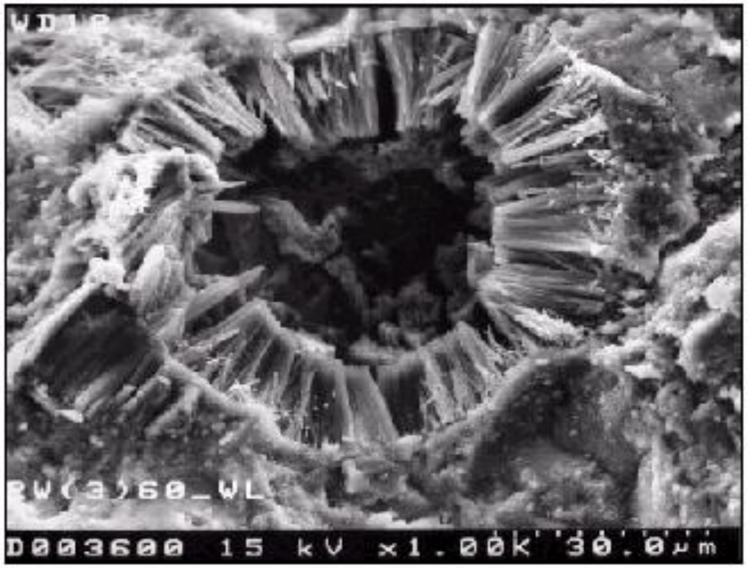
ÇİMENTO HARCI İÇ YAPISI

ETRENJİT



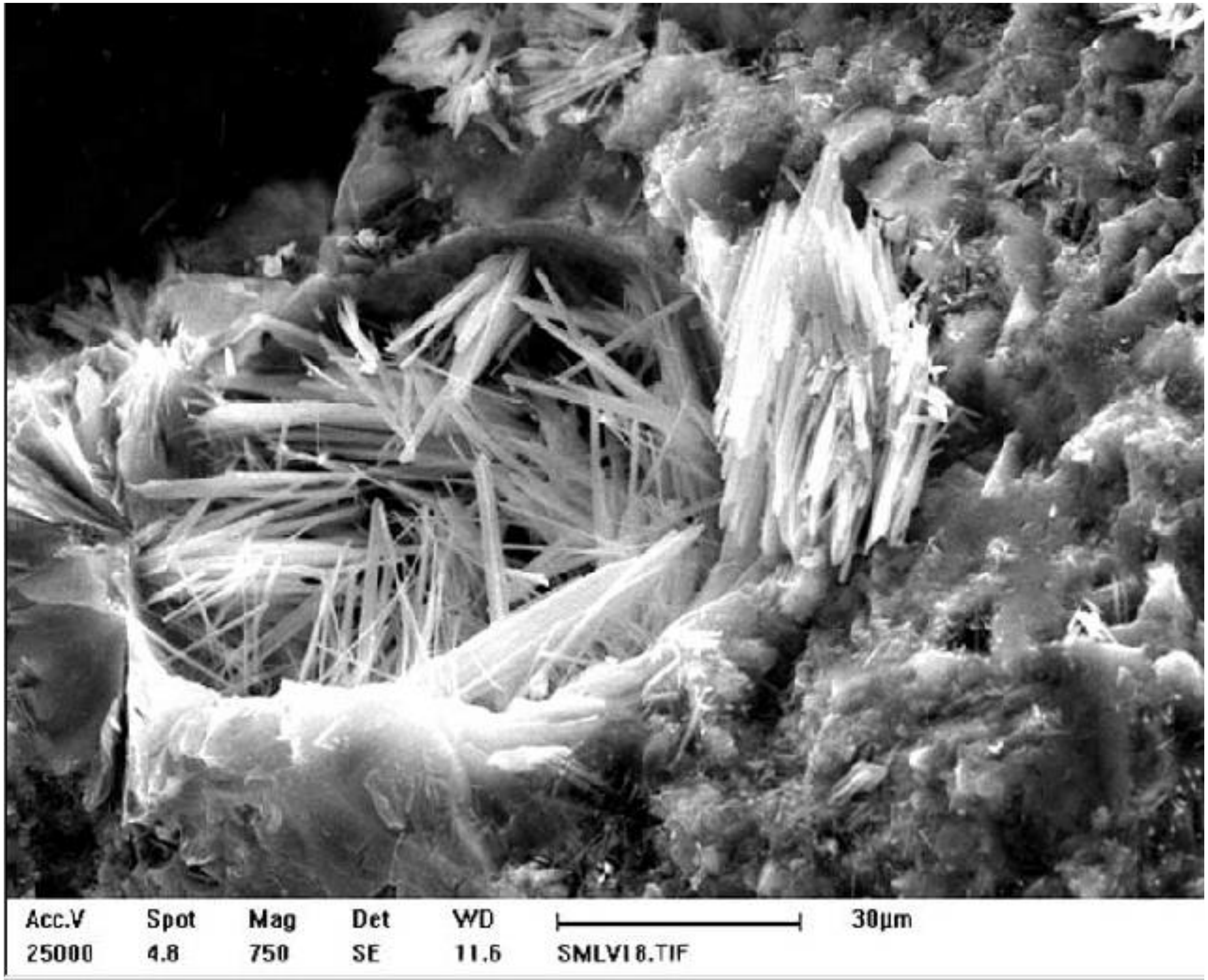






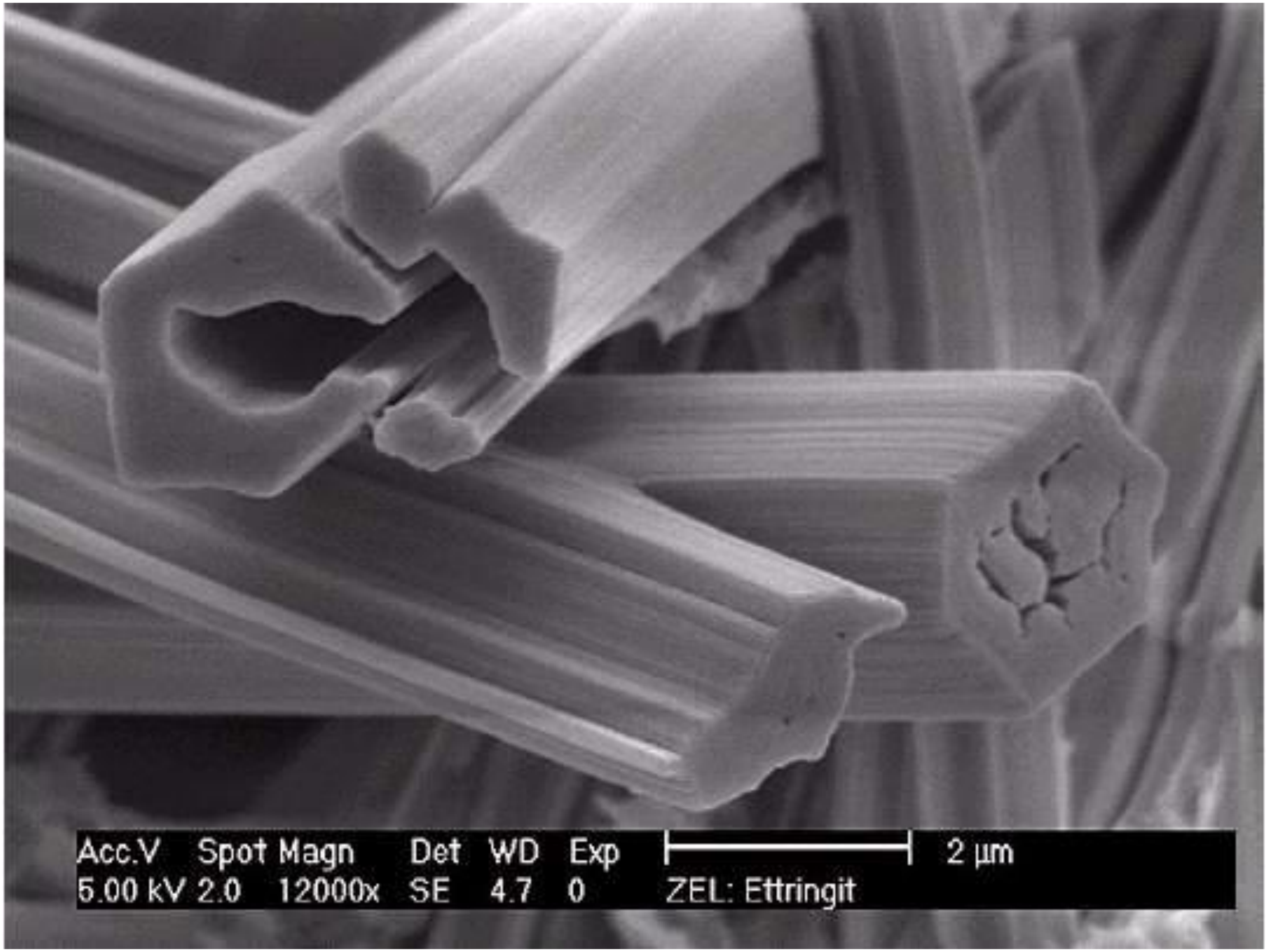
ÇİMENTO HARCI İÇ YAPISI

Etrenjit



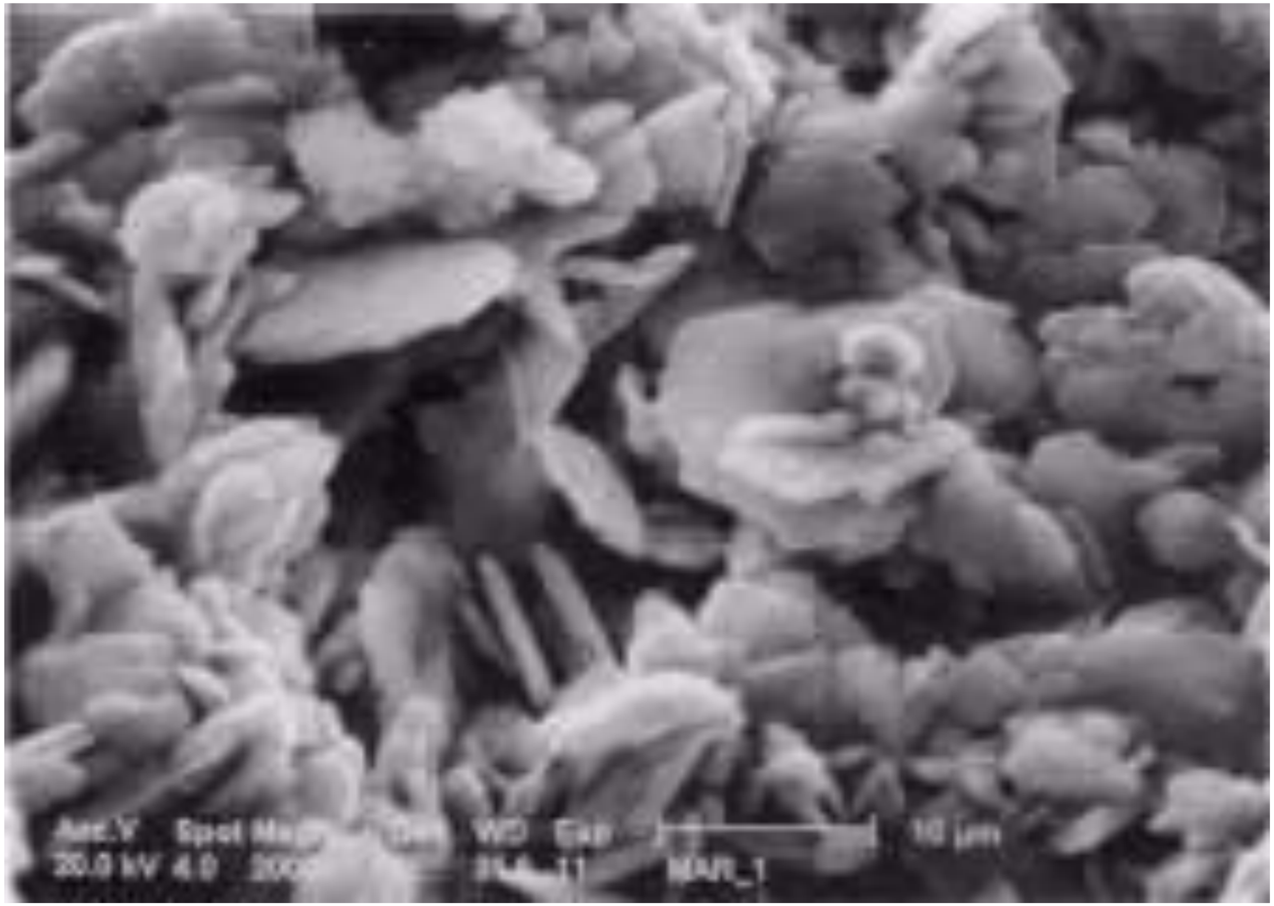
ÇİMENTO HARCI İÇ YAPISI

Etrenjit



ÇİMENTO HARCI İÇ YAPISI

Monosülfat



Hydration of Portland Cement - *Kinetics*

There are various methods of measuring the degree of hydration. In a simple empirical equation given by Rastrup, the development of hydration is given by a factor g, which corresponds to the degree of hydration for a given condition:

$$g = e^{\left(\frac{-d}{t^{0.5}}\right)}$$

where,

t: age of hardened cement paste (days)

d: a parameter depending on (i) type of cement, (ii) W/C, and (iii) moisture conditions. It is 0.85 for OPC at 20°C and W/C=0.50.

Hydration of Portland Cement – *Heat of Hydration*

The hydration of cement is accompanied by liberation of heat. Concrete being a fair insulator, in large masses, the generated heat during setting and hardening is not readily dissipated. This results in thermal gradients, expansion and development of serious cracks. The heat of hydration of a normal portland cement is of the order of 80 to 120 cal/g.

For the usual range of portland cements, about one half of the total heat of hydration is liberated in between 1 and 3 days, and about three quarters in 7 days. In fact, the rate of heat evolution is affected by factors such as fractions of the compounds in the cement, the temperature at which the hydration takes place and the fineness of cement particles.

Hydration of Portland Cement – *Heat of Hydration*

Heat of hydration of cements is determined experimentally by means of calorimetric methods. The following empirical equation gives a rough estimate for heat of hydration:

Heat of Hydration (cal/g) = $136(\%C_3S)+62(\%C_2S)+200(\%C_3A)+30(\%C_4AF)$

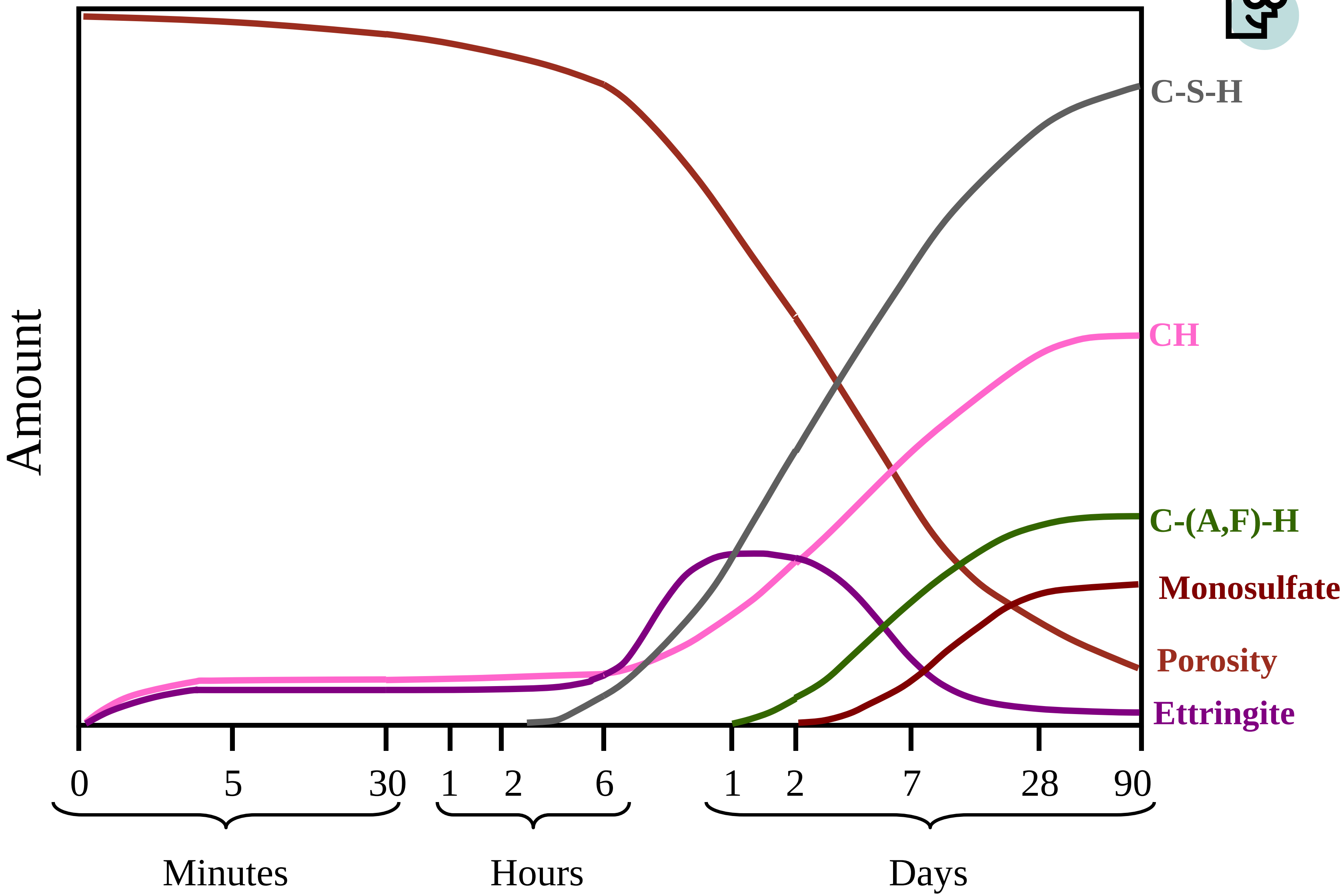
Structure of Hydrated Cement

Immediately upon mixing cement and water, the major compounds start to produce calcium-silicate-hydrate, calcium hydroxide, and calcium-alumino sulfohydrates. There are also some minor components.

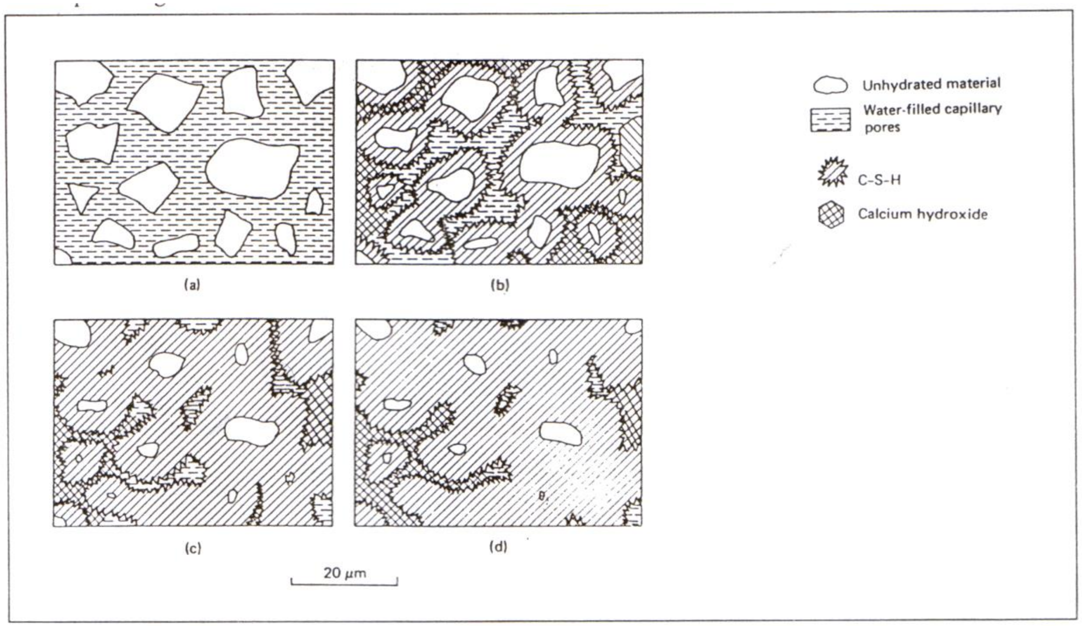
It is assumed that volume of paste does not change during hydration thus the hydration products are formed within the boundary of the initial volume (volume of cement+volume of water).

1 cc of cement produces 2.1 cc of hydration products when it hydrates completely.

Hydration Products



Structure of Hydrated Cement



Schematic outline of microstructural development in portland cement pastes, (a) initial mix, (b) 7 days, (c) 28 days, and (d) 90 days.

Many of the mechanical properties of hardened cement depend much on the physical structure of the products of hydration. Fresh cement paste is a plastic network of cement particles in water but once the paste has set, its apparent volume remains approximately constant. At any stage of hydration, the hardened paste consists of hydrates of various compounds.

Hydrated cement paste contains gel pores and capillary pores. The characteristics of these pores are different from each other.

Gel pores are interconnected interstitial spaces within the gel, they constitute a part of the gel structure. Gel pores occupy about 28% of the total volume of gel. The diameter of gel pores is about 2-3 nm (1 nm=10^{-9} m). Since the size of the gel pore is very small, the loss of water from the gel pore by evaporation, or the gain of water by the gel pore is not easy. If water is gained or lost under the effect of some forces, expansion or contraction of the paste takes place; the water in gel pores freezes at -78 °C.

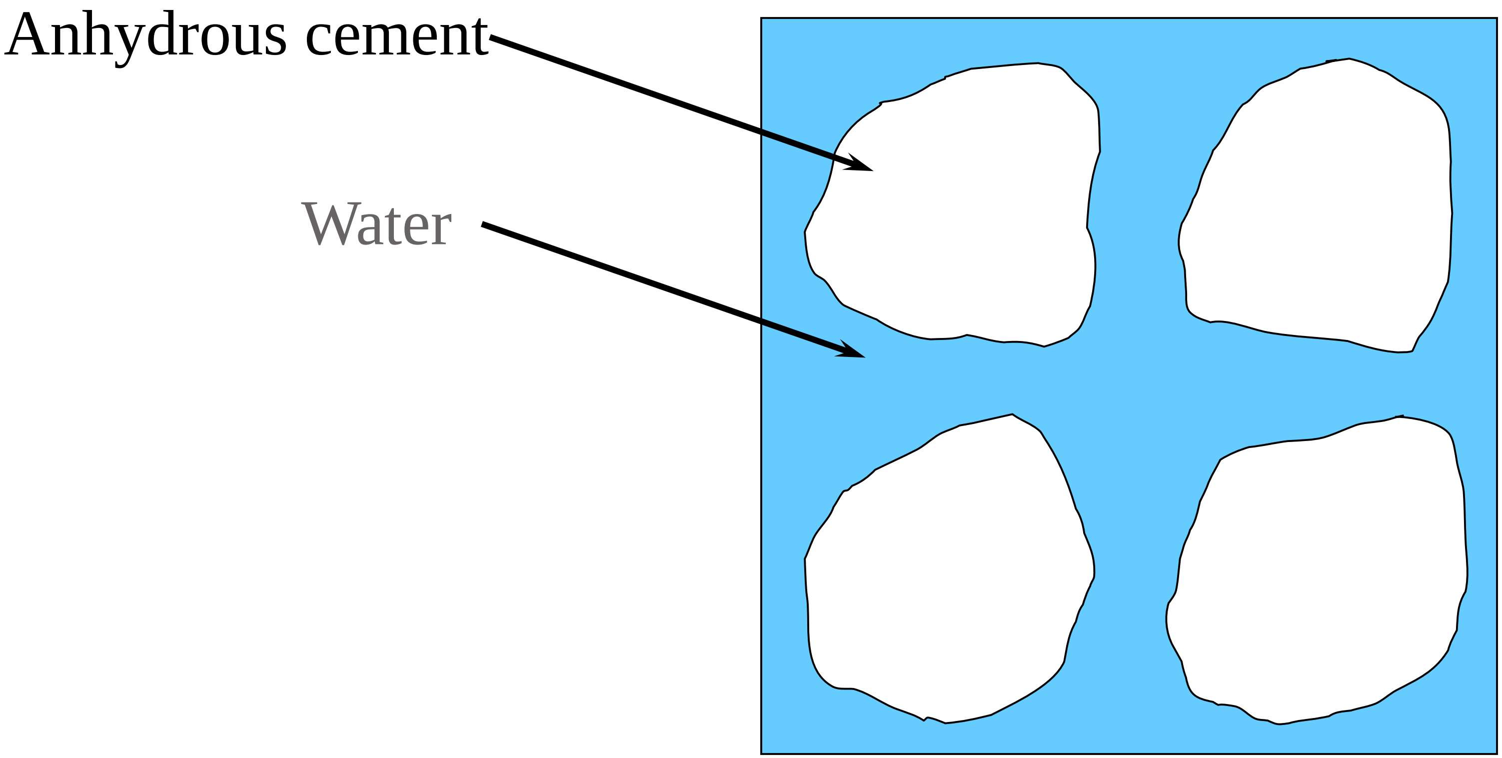
Capillary pores are relatively large pores in the hardened cement paste. The average size of the capillary pores is about 1-3 µm (µm=10^{-6} m). Capillary pores are randomly distributed; having varying sizes and shapes.

When cement paste is first prepared, the voids between the cement particles are full of water, and these voids are interconnected. Since the volume of the gel produced by the hydration of 1 ml of cement is about 2.1 ml, the gels occupy some of the capillary spaces. Thus, the volume of capillary pores decreases as more hydration takes place.

Capillary porosity of the paste depends on the W/C ratio of the paste and on the degree of hydration of the paste.

The water in the capillary pores is mobile. water can easily get into the pores or be lost from these. Interconnected capillary pores are mainly responsible for the permeability of the hardened paste. The water in the capillary pores can freeze at 0 °C or a few degrees below that. If the pores are *saturated* with water and *freezing* takes place, the formation of ice leads to high internal stresses in the hardened paste.

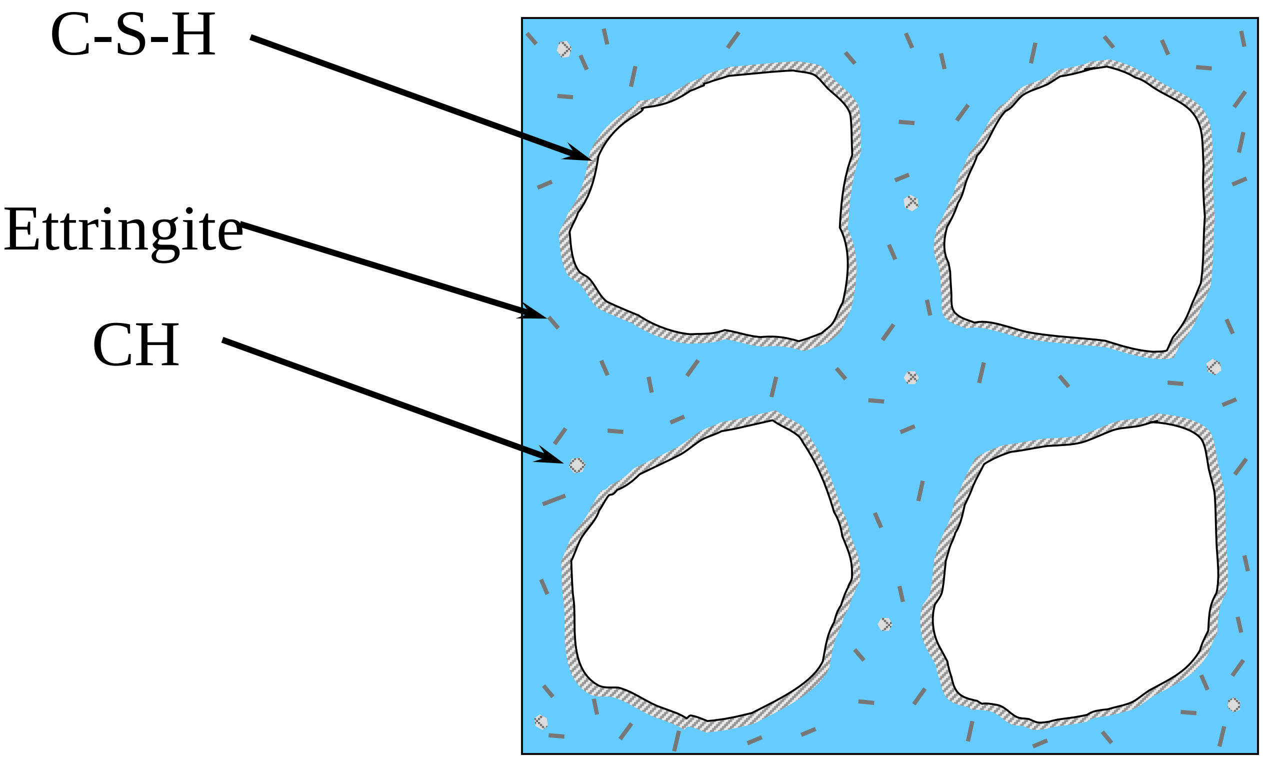
Development of Microstructure



Anhydrous cement

Water

Development of Microstructure

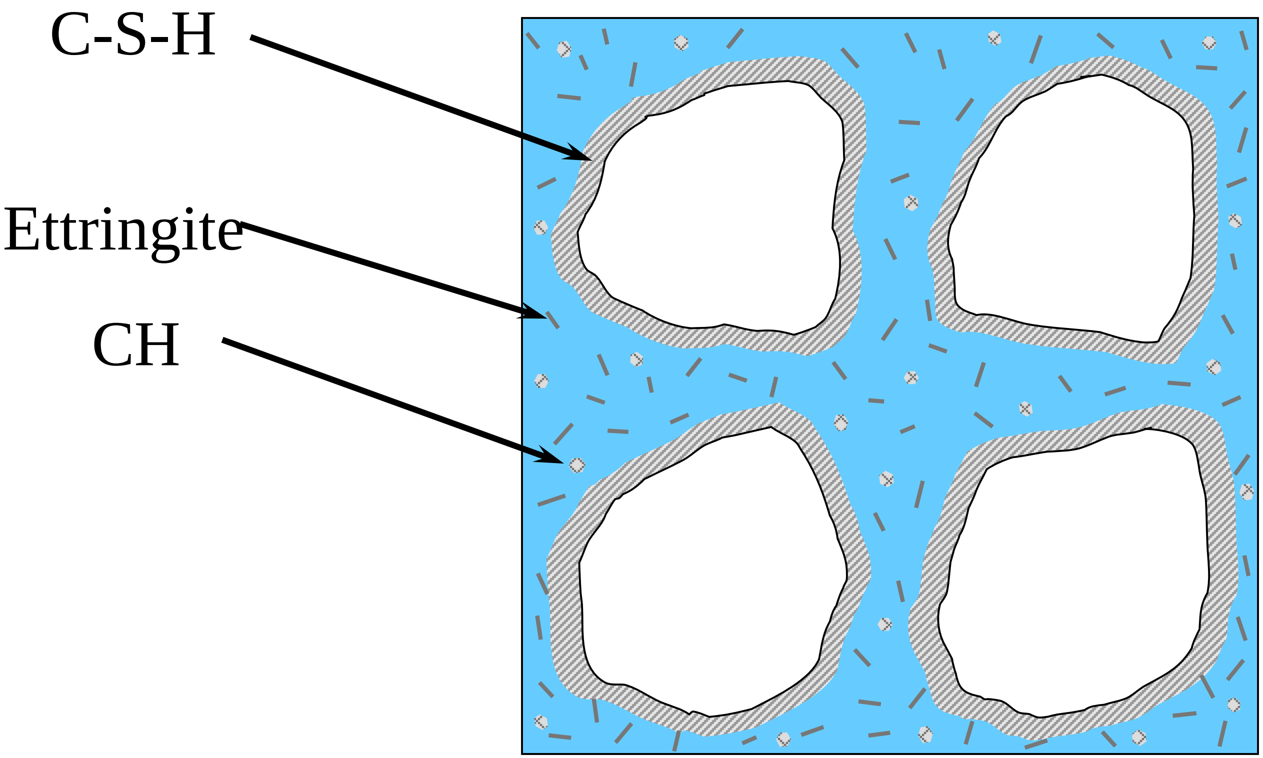


C-S-H

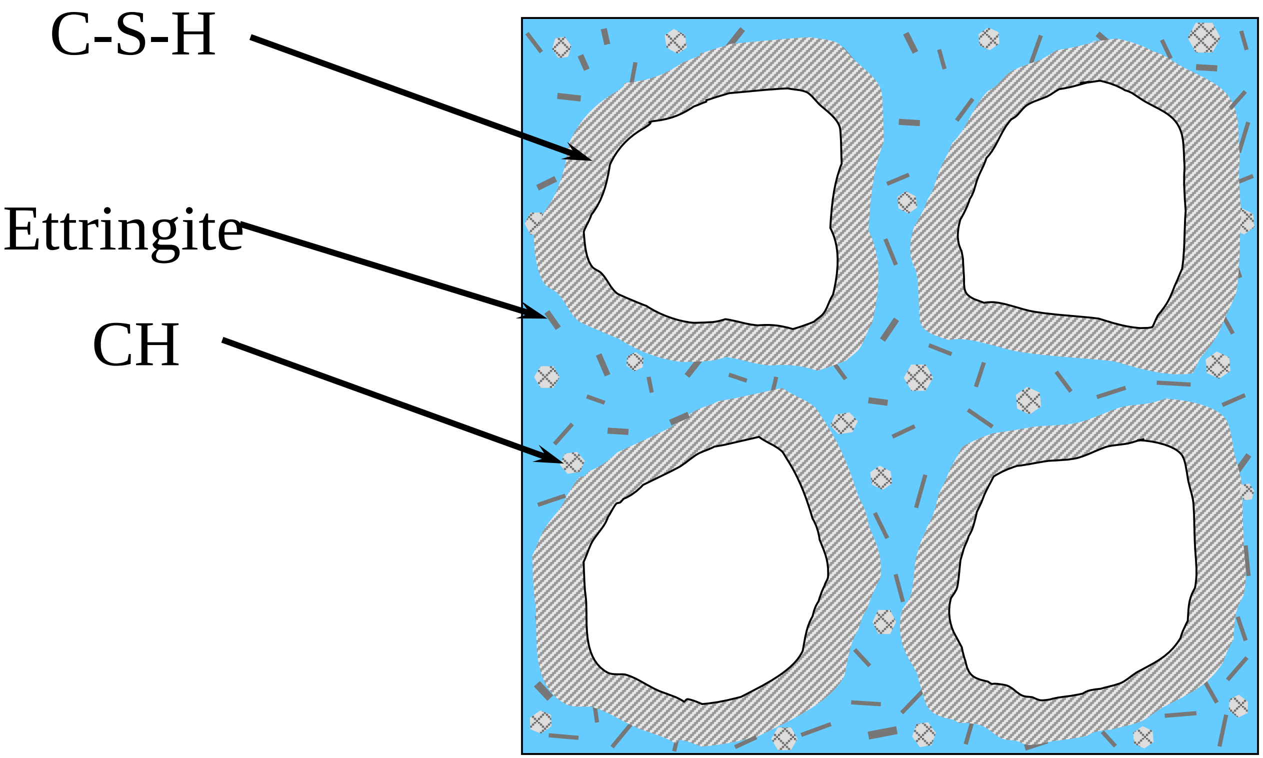
Ettringite

CH

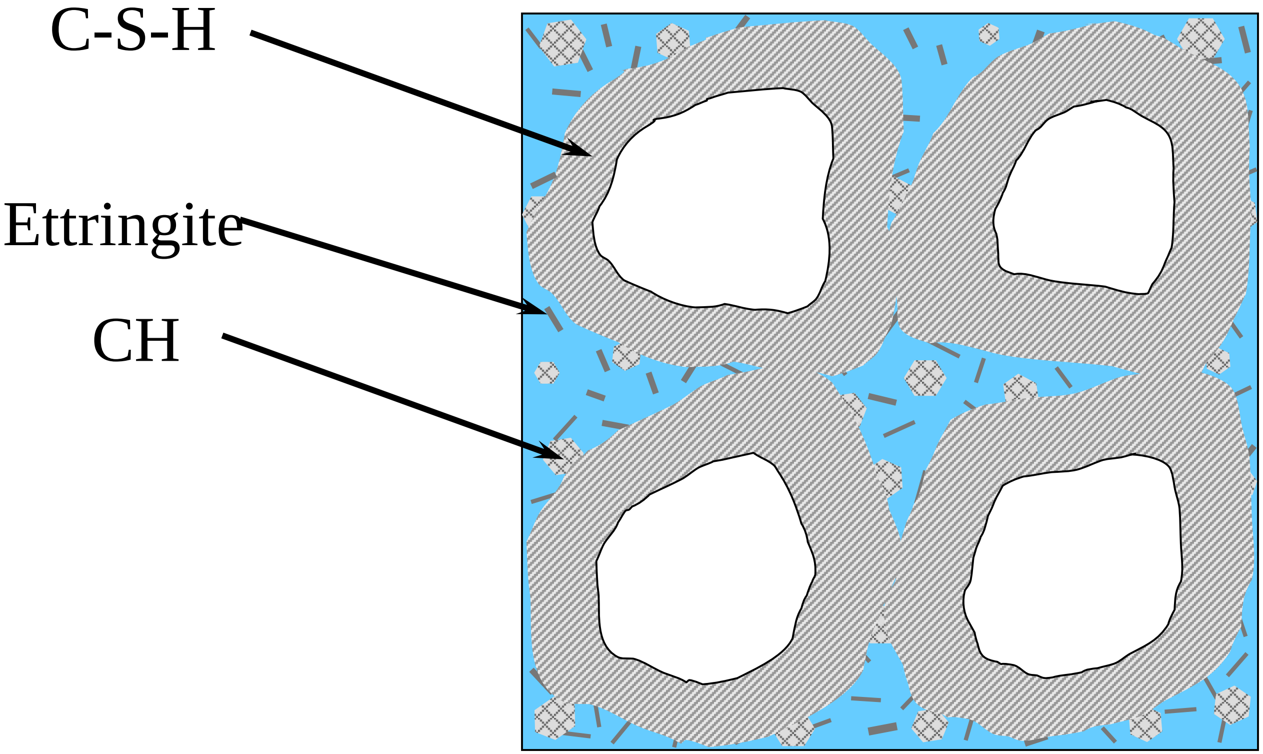
Development of Microstructure



Development of Microstructure



Development of Microstructure

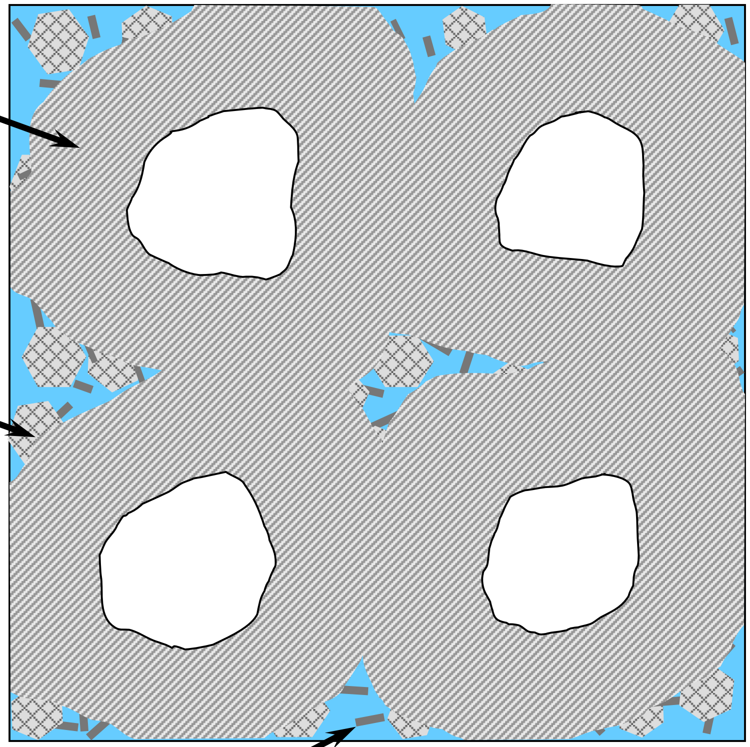


C-S-H

Ettringite

CH

Development of Microstructure



C-S-H

CH

Monosulfate

Let us, therefore, consider the hydration of 100 g of cement.

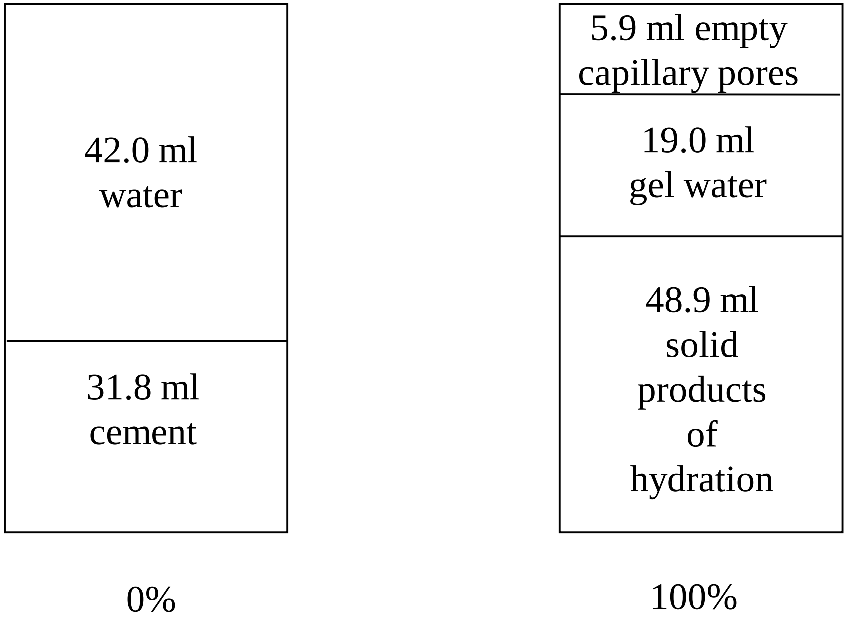
Taking the specific gravity of dry cement as 3.15, the absolute volume of unhydrated cement is 100/3.15 = 31.8ml.

The non-evaporable water is about 23% of the weight of cement, i.e., 23ml. The solid products of hydration occupy a volume equal to the sum of volumes of anhydrous cement and water less 0.254 of the volume of non-evaporable water,
i.e., 31.8 + 0.23[100(1-0.254)] = 48.9ml.

Since the paste in this condition has a characteristic porosity of about 28%, the volume of gel water, Wg is given by the following equation. Thus, the volume of hydrated cement is
48.9+19.0 = 67.9 ml.

$$\frac{W_g}{48.9 + W_g} = 0.28$$

Schematic representation of hydration of cement


42.0 ml water

31.8 ml cement

0%

5.9 ml empty capillary pores

19.0 ml gel water

48.9 ml solid products of hydration

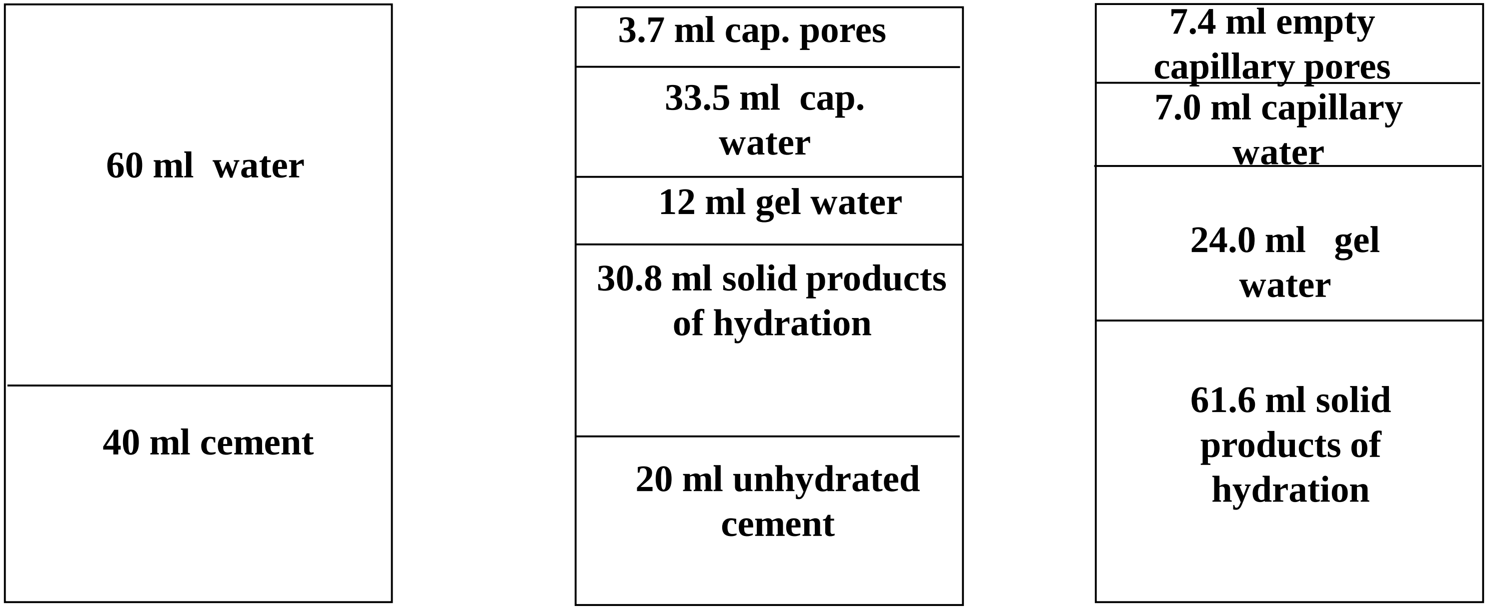
100%

Summarizing, we have:
Weight of dry cement=100 g
Absolute volume of dry cement= 31.8 ml.
Weight of combined water= 23 g
Volume of gel water= 19.0 ml.
Total water in the mix= 42.0 ml.
W/C (by weight) = 0.42
W/C (by volume) = 1.32 .
Volume of hydrated cement= 67.9 ml.
Original volume of cement + water= 73.8 ml.
Decrease in volume due to hydration = 5.9 ml
Volume of products of hydration per 1cm^3 of dry cement = 2.1 ml.

As a more specific example let us consider the hydration of a paste with W/C = 0.475, sealed in a tube.
Let the weight of dry cement be 126 g, which corresponds to 40 ml. The volume of water is then 0.475x126 = 60 ml. Let us now consider the situation when the cement has hydrated **fully**. The non-evaporable water is 0.23 x 126 = 29 ml. and the gel water Wg is such that
Wg/[40+29(l-0.254) + Wg] = 0.28. Thus Wg = 24.0 ml.

The volume of hydrated cement is 40 + 29(1-0.254) +24 = 85.6 ml. There is thus 60 - (29+24) = 7 ml water left as capillary water in the paste. In addition, 100 - (85.6 +7) = 7.4 ml form empty capillaries. If the cement is half hydrated, the gel/space ratio becomes:

$$\frac{0.5\left[40+29(1-0.254)+24\right]}{100-20}=0.535$$



60 ml water

40 ml cement

3.7 ml cap. pores

33.5 ml cap. water

12 ml gel water

30.8 ml solid products of hydration

20 ml unhydrated cement

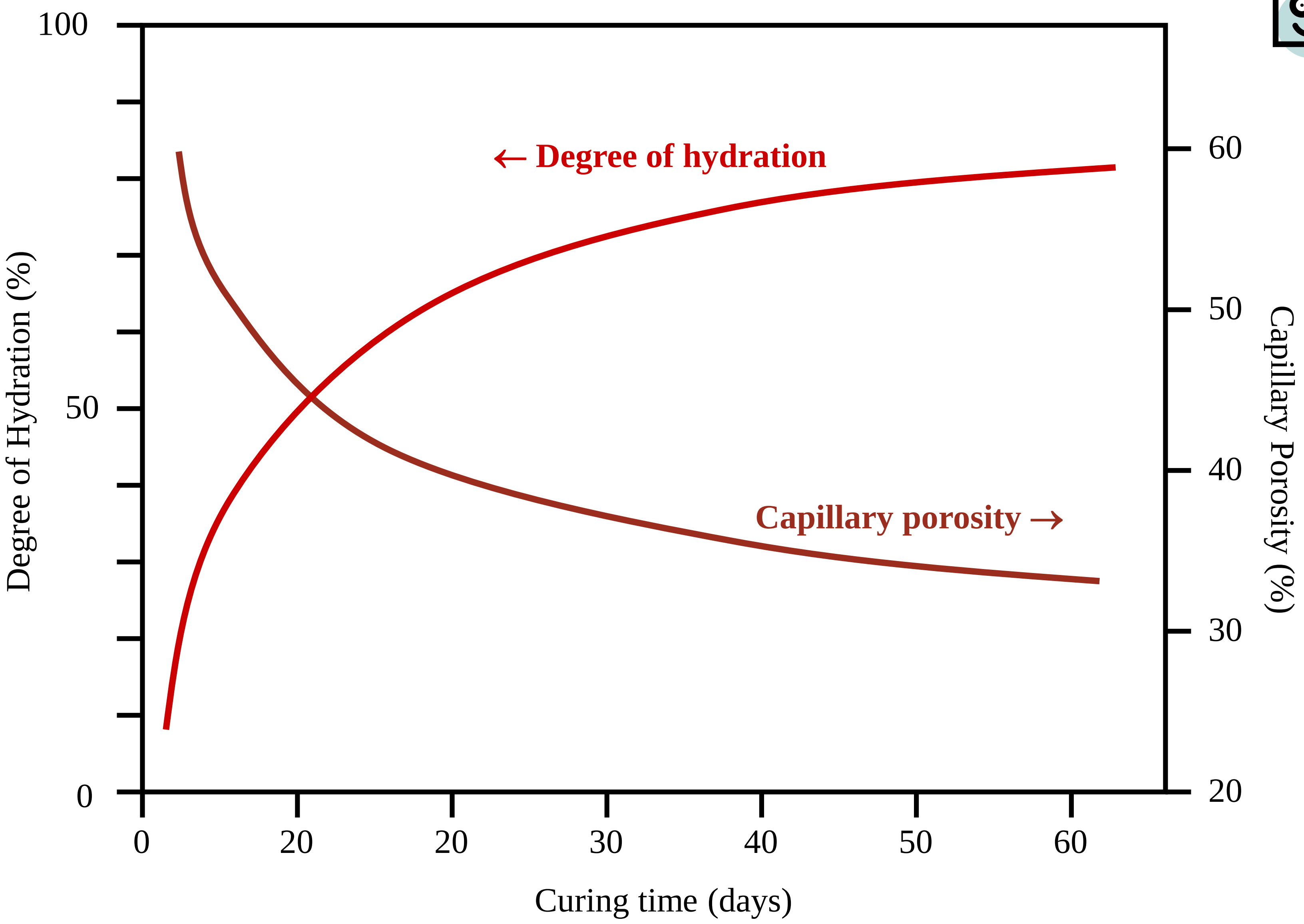
7.4 ml empty capillary pores

7.0 ml capillary water

24.0 ml gel water

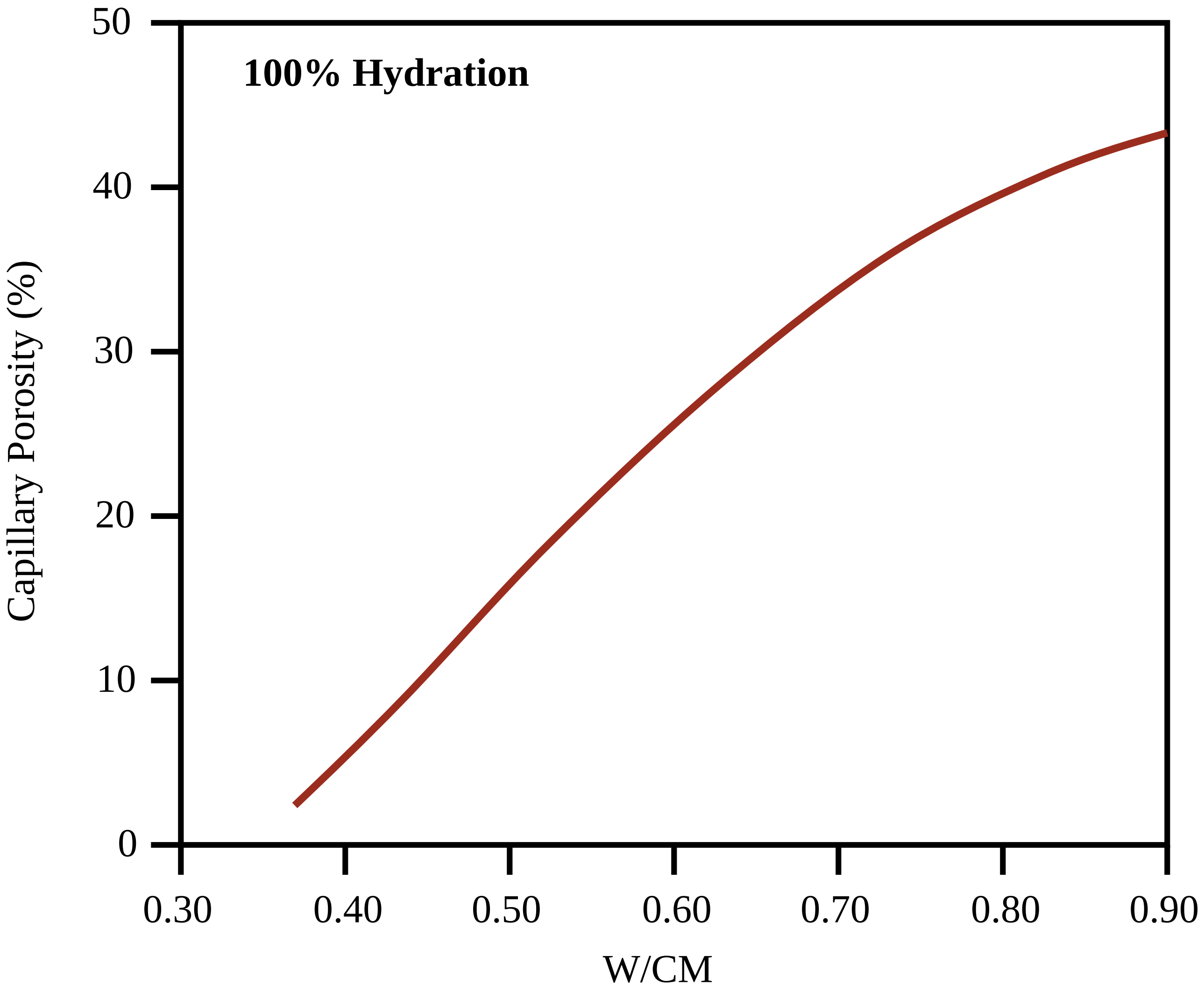
61.6 ml solid products of hydration

Development of Microstructure



Young et al. 1998

Development of Microstructure

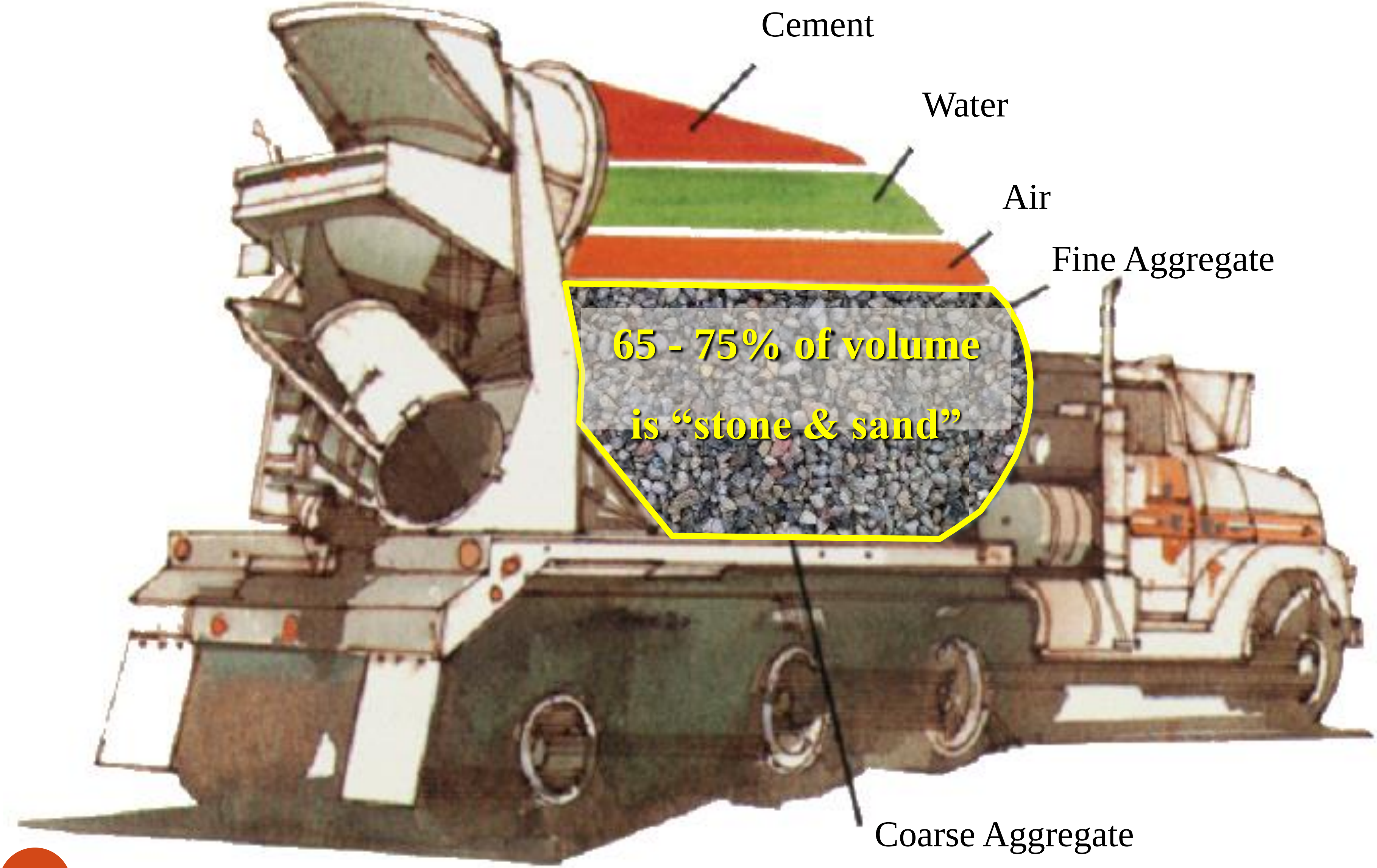


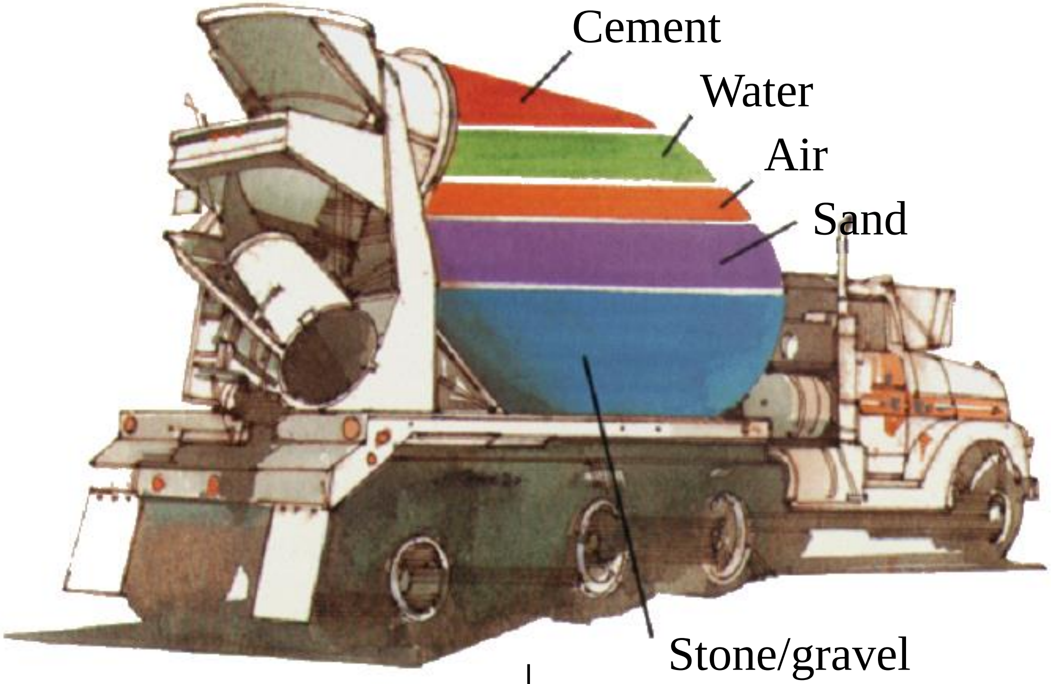
Young et al. 1998

ABSOLUTE HIGHEST POINT ON EARTH
CIVIL ENGINEERING

CONCRETE TECHNOLOGY

LECTURE NOTES
PROF. DR. KAMBİZ RAMYAR


Cement
Water
Air
Fine Aggregate
65 - 75% of volume
is “stone & sand”
Coarse Aggregate



Fresh Concrete Properties

- Workability
- Consistency
- Bleeding
- Pumpability
- Finishing
- Air Content

Hardened Concrete Properties

- Strength
- Thermal
- Modulus of Elasticity
- Shrinkage
- Abrasion
- Durability

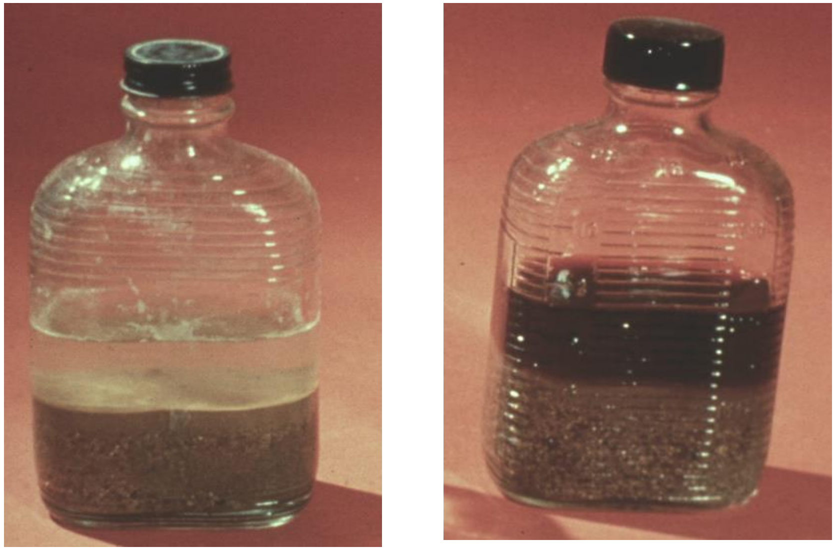
Influence of Aggregate on Fresh Concrete Properties

Aggregate Property	**Concrete Property**
Particle size distribution (grading)	Workability & water demand
Particle shape	Paste content
Particle texture	

Influence of Aggregate on Fresh Concrete Properties

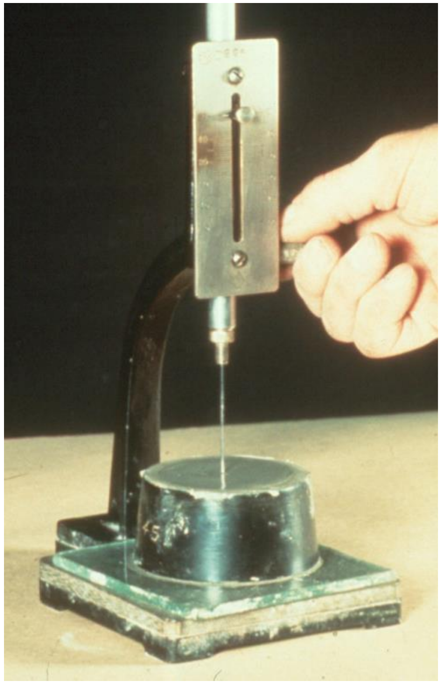
Aggregate Property

Organic impurities



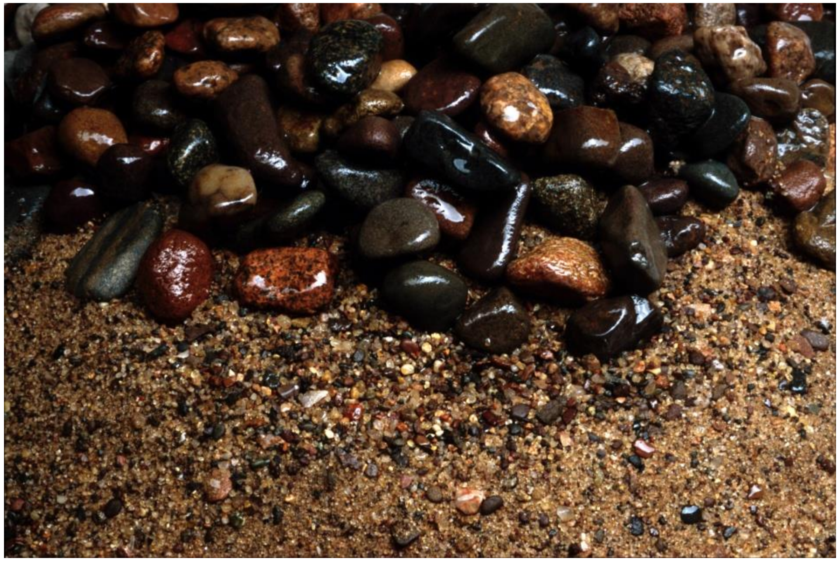
Concrete Property

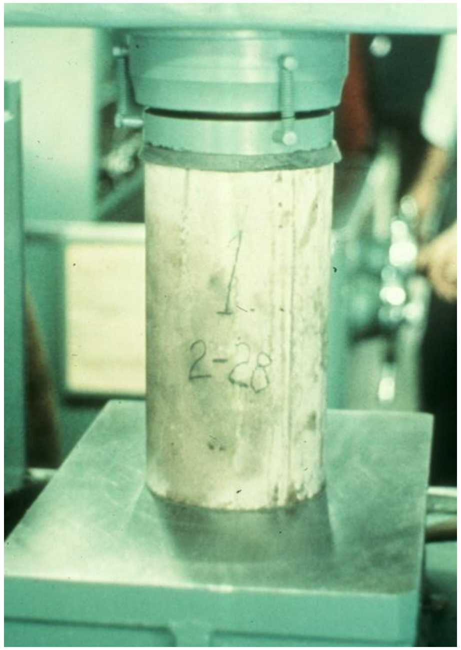
Setting behaviour



Influence of Aggregate on Hardened Concrete Properties

Aggregate Property	**Concrete Property**
Strength	Mechanical behaviour
Stiffness	Strength
Size and shape	Modulus of Elasticity
Organic impurities	Shrinkage



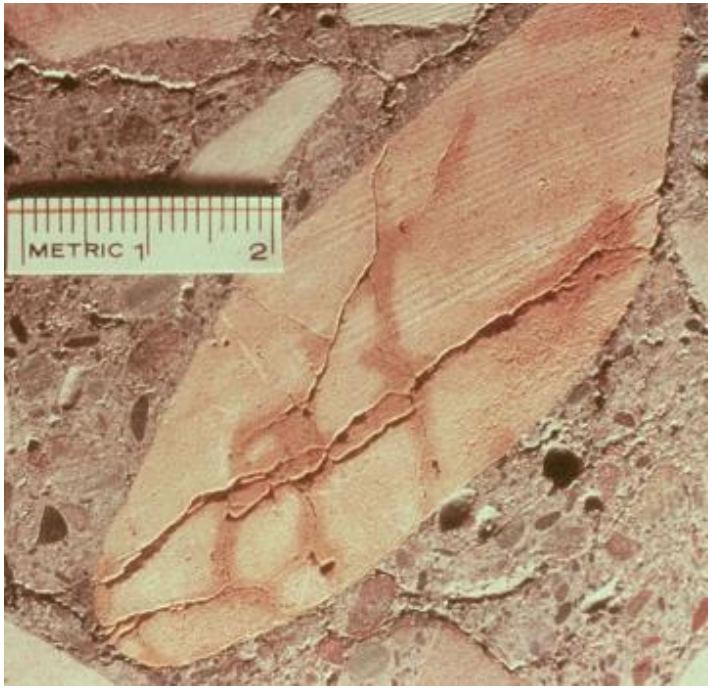


Influence of Aggregate on Hardened Concrete Properties

Aggregate Property	**Concrete Property**
Coefficient of thermal expansion	Coefficient of thermal expansion
Thermal conductivity	Thermal conductivity
Specific heat	Specific heat
Thermal diffusivity	Thermal diffusivity

Influence of Aggregate on Hardened Concrete Properties

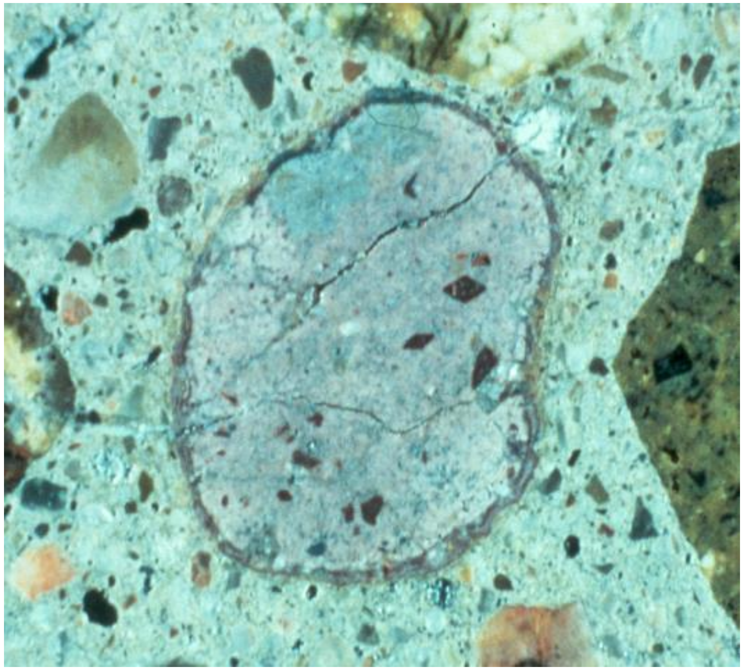
Aggregate Property	**Concrete Property**
Soundness	Resistance to freezing and thawing
Frost resistance	
Particle size	



Influence of Aggregate on Hardened Concrete Properties

Aggregate Property

Presence of alkali-reactive minerals

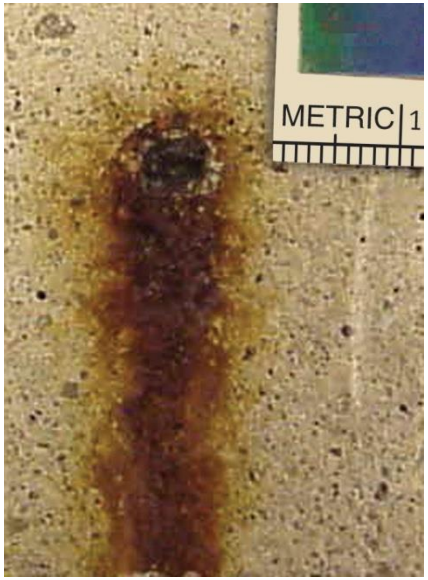


Concrete Property

Resistance to alkali-aggregate reaction

Influence of Aggregate on Hardened Concrete Properties

- Density
- Porosity
- Volume stability
- Abrasion resistance
- Frictional properties (pavements)
- Aesthetics
- Economics



- **EVALUATION OF AGGREGATES**

Aggregates generally occupy about 70 to 80% of the volume of concrete and therefore have an important influence on its properties.

Concrete Property	Relevant Aggregate Property
Durability	
Freezing-Thawing Resistance	Soundness, porosity, clay minerals
Wetting-Drying Resistance	Pore structure
Heating-Cooling Resistance	Coefficient of thermal expansion
Abrasion Resistance	Hardness
Alkali-Aggregate Reaction	Presence of amorphous silica
Strength	Strength, surface texture, cleanliness, particle shape, maximum size
Shrinkage and Creep	Modulus of Elasticity, surface texture, grading, particle shape, maximum size, cleanliness
Economy	Particle shape, grading, maximum size, availability
Unit weight	Specific gravity, particle shape, grading, maximum size

PROPERTIES REQUIRED FOR MIX DESIGN

In order to be able to proportion suitable concrete mixes, certain properties of the aggregates must be known:

(1) shape and surface texture,

(2) particle size distribution (gradation),

(3) moisture content,

(4) specific gravity,

(5) bulk density.

Shape and Texture

Effect on Workability-1

In a good concrete, all fine aggregates must be surrounded by cement paste and all coarse aggregates by mortar to provide lubrication which decreases the interactions between aggregate particles during mixing.

The ideal aggregate particle is one that is close to spherical in shape with a relatively smooth surface. Most natural river sands and gravels come close to this ideal.

Shape and Texture

Effect on Workability-2

Crushed stone is much more angular and may have a rough surface texture. Such particles interfere more with the movement of adjacent particles. They also have a higher surface-to-volume ratio and therefore require more paste to coat the surface of each particle.

Aggregates that are flat or elongated also increase the interparticle interaction and surface-to-volume ratio.

The surface texture of the aggregate is also important for workability, since rough surface requires more lubrication for movement.

Shape and Texture

Effect on Mechanical Properties-1

The shape and texture of the fine aggregate affects only workability, but the characteristics of coarse aggregate may also affect the mechanical properties of the concrete by affecting the mechanical bond between the cement paste matrix and the aggregate.

Shape can favorably increase the strength by increasing the amount of surface area available for bonding with the paste for a given aggregate content. However, extremes in aggregate shape may lead to high internal stress concentrations and hence easier bond failure.

Shape and Texture

Effect on Mechanical Properties-2

Rough surface texture will improve the bond.

It is also thought that the mineralogical character of the aggregate can play an important role in determining the strength of the aggregate-matrix bond, since a chemical interaction may occur at the interface between certain kinds of rocks and the cement paste.

Size Gradation

The particle size distribution or grading of an aggregate is an important characteristic because it determines the paste requirements for a workable concrete.

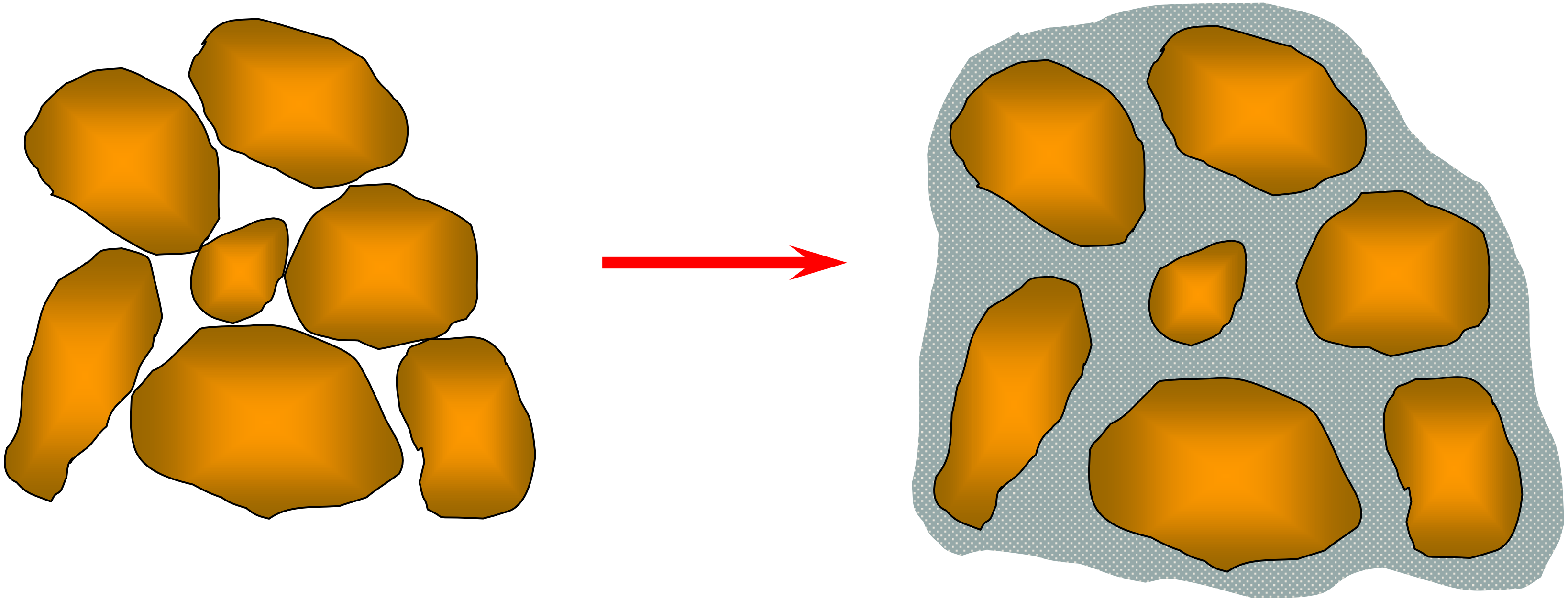
Since cement is the most expensive component of concrete, it is desirable to minimize the cost of concrete by using the smallest amount of paste consistent with the production of a concrete that can be handled, compacted, and finished and provide the necessary strength and durability.

DIGITAL SIEVE SHAKER

Aggregate Grading

Amount of paste required > volume of voids between aggregates

Aggregates need to be coated by and become dispersed in the paste to provide workability



Gap-grading

Gap-grading is defined as grading in which one or more intermediate-size fractions are omitted. It can be used to produce more economical concrete. Less sand can be used for a given workability, hence allowing the use of less cement or lower W/C ratio for a given slump.

However, the omission of some size fractions can lead to segregation problems, particularly in mixes of high workability. Thus, gap-grading is recommended for stiff mixes. Gap-graded aggregate concretes are not superior to continuous graded ones except for reduced W/C ratios for a given cement content and slump value.

NO-FINES CONCRETE

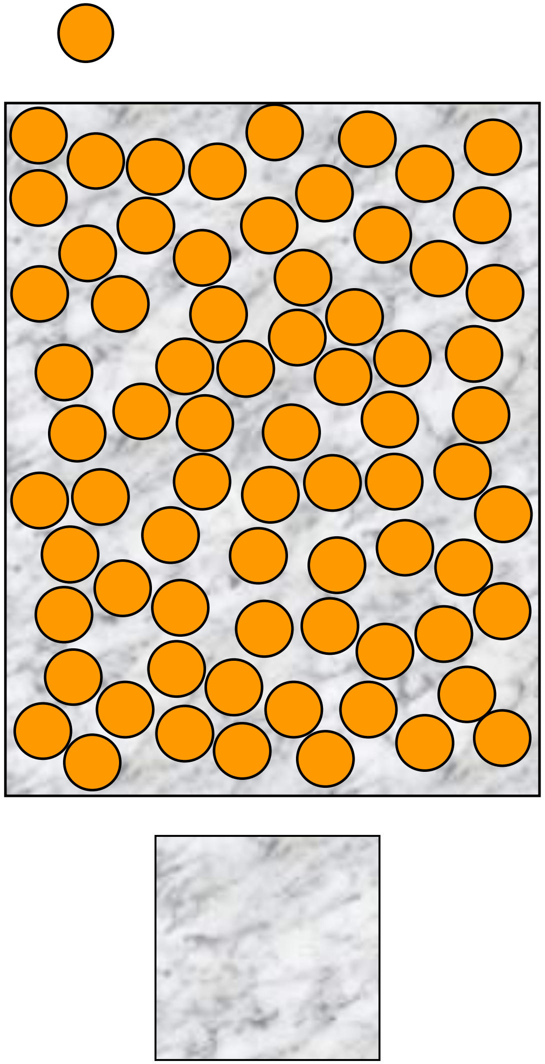
A special case of gap-grading is no-fines concrete in which the fine aggregate is entirely omitted.

The advantages of no-fines concrete are its low density, low drying shrinkage, and high thermal insulation.

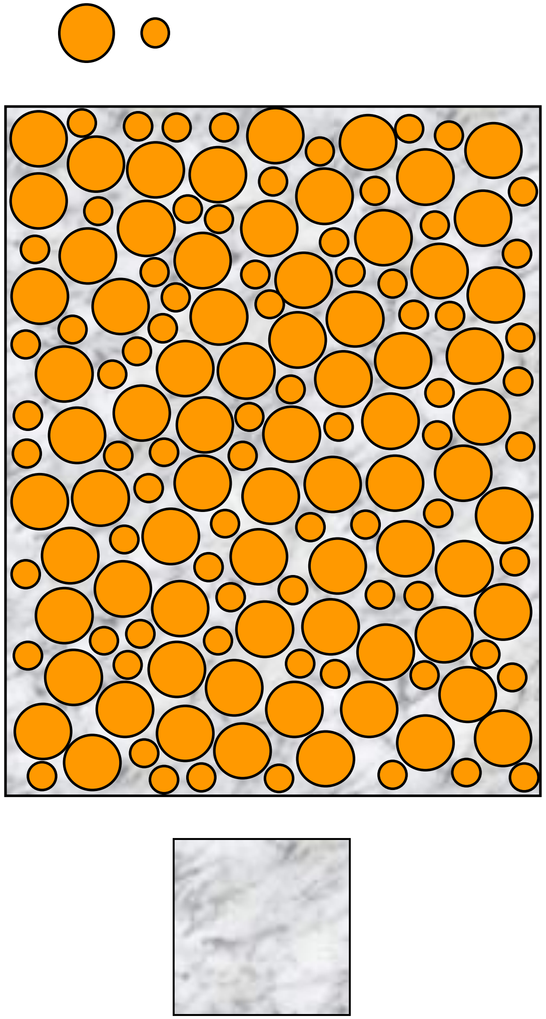
This concrete is highly prone to segregation. Therefore it should be compacted lightly by rodding.

Aggregate Grading

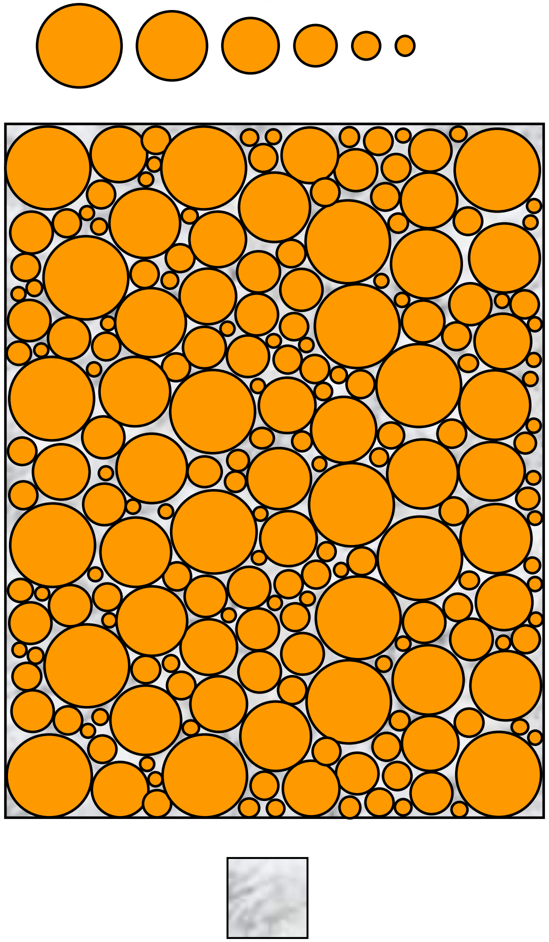
Single-sized



Gap-graded



Well-graded



Volume of paste (cement + water) to fill voids

The volume of voids is maximum when the particles are of the same size. When particles of different sizes are used smaller ones may pack in between the larger ones thus decrease the void space and amount of paste to fill the voids.

Using a larger maximum size of aggregate can also reduce the voids space. There are certain grading requirements stated in standards and specifications.

Aggregate Grading

Reducing the paste content of concrete leads to:

- Reduced cost
- Reduced temperature rise
- Reduced shrinkage
- Reduced permeability

Aggregate Grading

Maximum Aggregate Size

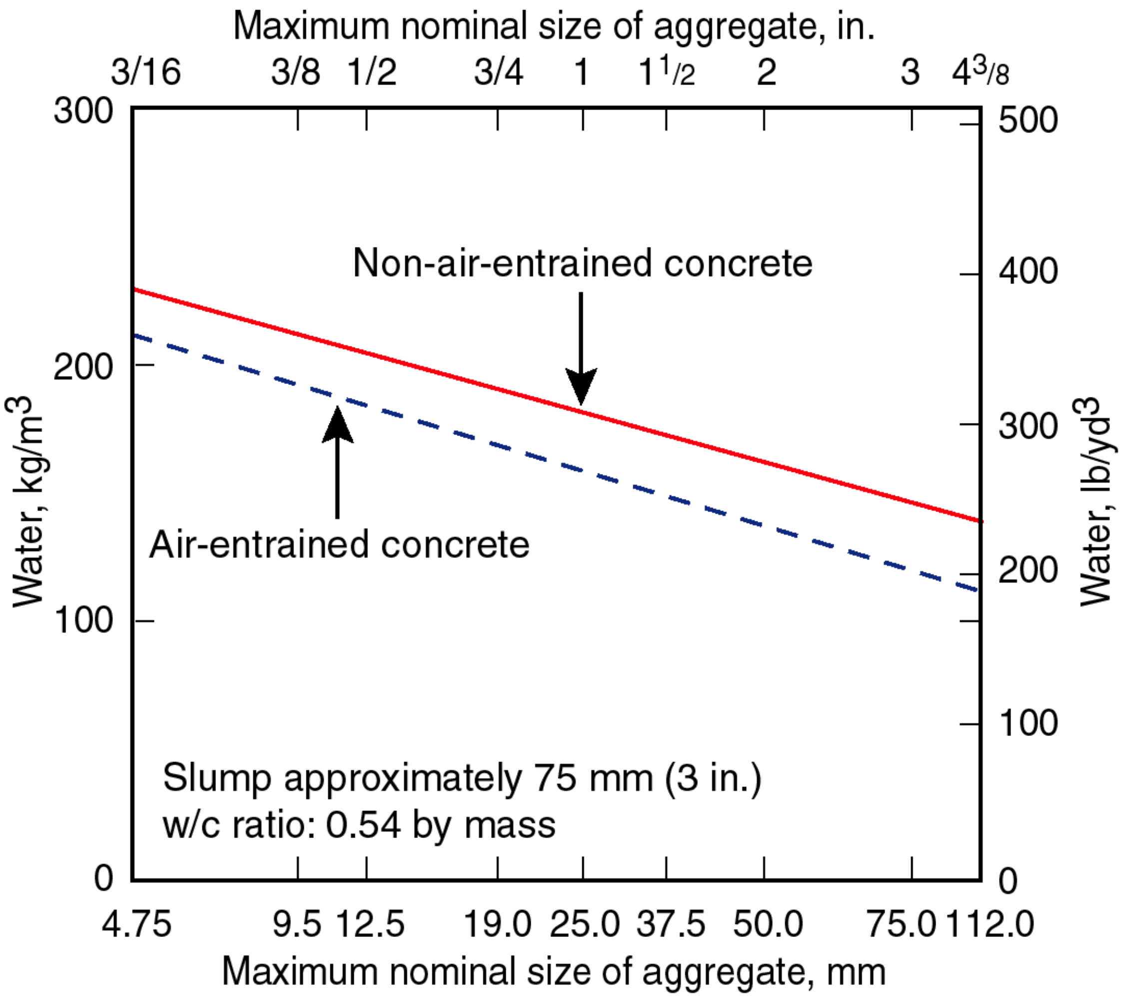
As max. aggregate size increases:

Water demand for a given slump decreases

∴ Cement content for given W/CM decreases

Paste content of mix decreases

Cost, shrinkage and heat evolution decrease

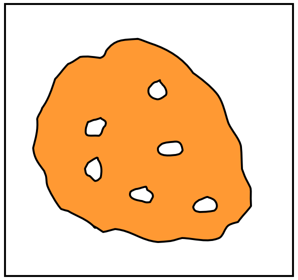


Moisture Contents

Since aggregates contain some porosity, water can be absorbed by them. Also, water can be retained on the surface of aggregate particles as a film. Thus, stockpiled aggregates can have variable moisture contents.

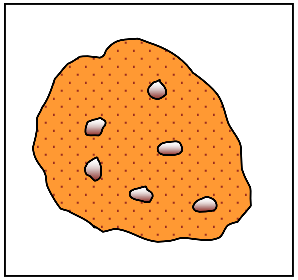
It is necessary to have information about the moisture conditions of aggregates because when the aggregate absorbs water it will be removed from the paste and the W/C ratio will be lowered and workability will be adversely affected; on the other hand if excess water is present on the aggregate surface it will be added to the paste and W/C ratio will be higher and strength will be lower.

Moisture Content & Absorption



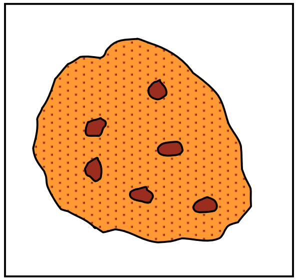
Oven dry

All moisture is removed from the aggregate by heating in an oven at 105°C to constant weight. All pores are empty.



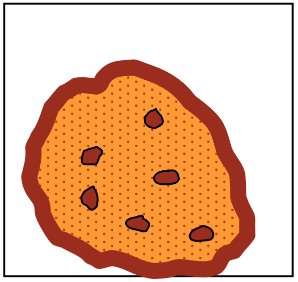
Air dry

All moisture is removed from the surface but pores are partially full.



Saturated surface dry (SSD)

All pores are filled with water but the surface is dry.



Damp or wet

All pores are completely filled with water and the surface is covered with a film of water.

Absorption and Surface Moisture

In order to calculate the amount of water that the aggregate will add to or subtract from the paste, three quantities must be defined:

(1) Absorption capacity,

$$AC = \frac{W_{SSD} - W_{OD}}{W_{OD}} x100\%$$

(2) Effective absorption,

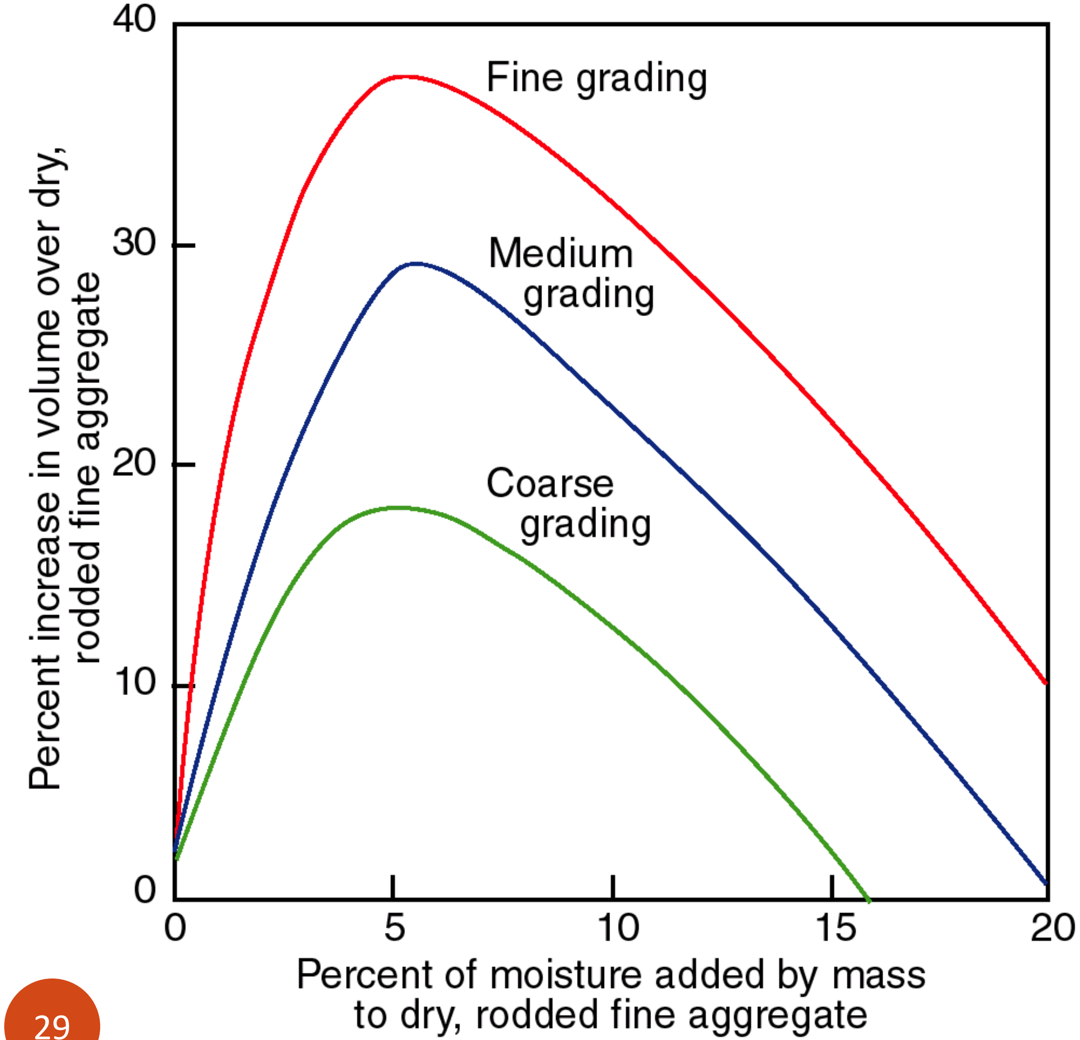
$$EA = \frac{W_{SSD} - W_{AD}}{W_{SSD}} x100\%$$

(3) Surface moisture.

$$SM = \frac{W_{wet} - W_{SSD}}{W_{SSD}} x100\%$$

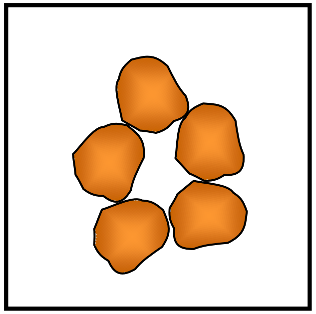
Moisture Content & Absorption

Bulking of Sand

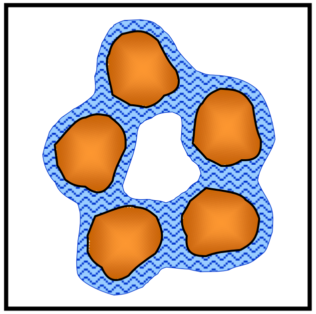


Stockpile

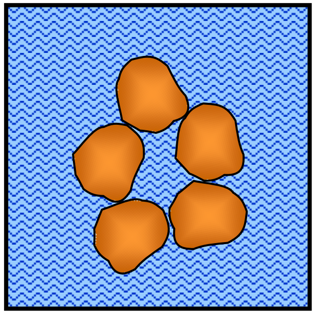
dry



Stockpile
partially
saturated



Stockpile
saturated



Bulking of Sand

A wet fine aggregate may show a larger apparent volume than the actual during stockpiling.

Additional moisture can be held in the interstices between particles as the result of menisci formed. The menisci create thicker films of water and push the fine aggregate particles apart and thus increase the apparent volume.

Bulking can cause errors in proportioning by volume.

Specific Gravity

$$S.G. = \frac{density\, of\ solid}{density\, of\ water}$$

$$ASG = \frac{weight\, of\ aggregate(solid\, only)}{volume\, of\ aggregate(solid\, only)}\, x \frac{1}{\rho_{water}}$$

$$BSG = \frac{weight\, of\ aggregate(solid + water\, in\, pores)}{volume\, of\ aggregate(solid + pores)}\, x \frac{1}{\rho_{water}}$$

BSG is the most realistic to use since the effective volume that aggregate occupies in concrete includes its pores.

$ASG > BSG_{SSD} > BSG_{OD}$

Since the porosity of most aggregates used in normal weight and heavy weight concrete is of the order of 1-2%, the specific gravities are approximately the same. The BSG of most natural normal weight aggregates range between 2.4 and 2.8.

Unit Weight-1

Unit Weight (UW) is the weight of a given volume of graded aggregate. It is also known as Bulk Density.

The UW measures the volume that the graded aggregate will occupy in concrete and includes both the solid aggregate particles and the voids between them.

It is simply measured by filling a container of known volume and weighing it. Clearly, however, the degree of compaction will change the amount of voids and thus the value of unit weight. Most standard methods for determining the UW require compaction by rodding.

Since the weight of the aggregate is dependent on the moisture content of the aggregate, constant moisture content is required. The UW of the coarse aggregate is necessary for the volumetric method of mix proportioning of concrete.

Unit Weight-2

The void space that must be filled with paste and/or mortar can be calculated by using the unit weight as follows:

Total volume = $V_a + V_v = 1m^3$

Therefore, $V_v = (1 - V_a)\ m^3$

where V_a and V_v are the volume of aggregate and void space, respectively.

$$V_a = \frac{W_a}{BSG.\rho_w} m^3$$

Also,

$$V_v = 1 - \frac{W_a}{BSG.\rho_w} m^3$$

where, ρ_w is the density of water and BSG is the bulk specific gravity of aggregate in dry state. Therefore,

$$percent of\ voids = \frac{BSG.\rho_w - UW}{BSG.\rho_w} x100$$

other words,

Mechanical Properties

Strength

Compressive strengths generally 65 to 270 MPa

Not critical for normal concrete, more important in high-strength concrete

Not usually measured directly

Tensile strengths typically 2 to 15 MPa

Mechanical Properties

Modulus of Elasticity

Most aggregates are much stiffer than cement paste

Modulus of elasticity of aggregates affects magnitude of shrinkage and creep in concrete.

Modulus of elasticity of aggregates ranges from 6.9 - 69 GPa

Thermal Properties

Coefficient of Thermal Expansion

Coefficient of thermal expansion (CTE) of concrete is strongly influenced by CTE of aggregate

Paste (saturatd.) : 18-20 x10^{-6}/°C

Concrete : 7-13 x10^{-6}/°C

Rock Type	CTE x10^{-6}/°C
Basalt/dolerite/ gabbro	3.6 – 9.7
Granite	1.8 – 11.9
Sandstone	4.3 – 13.9
Dolomite	6.7 – 8.6
Limestone	0.9 – 1.22
Marble	1.1 – 16.0
Porphyry	5.7 – 8.4
Quartzite	10.8
Flint/chert	7.4 – 13.1
Diorite/ Andesite	4.1 – 10.3

Lea, 1998

Thermal Conductivity

Influenced by aggregate

Quartz & quartzite > 3.0 W/m°C

Igneous rocks < 1.5 W/m°C

DURABILITY OF AGGREGATES

Any lack of durability in the aggregates may lead to disastrous results in the concrete.

Physical durability is concerned with the susceptibility of aggregates to freezing-thawing, wetting-drying and wear.

Chemical durability is concerned with various forms of cement-aggregate reactions.

THESE WILL BE MENTIONED IN *DURABILITY OF CONCRETE* SESSION

EUROPEAN STANDARD
NORME EUROPÉENNE
EUROPÄISCHE NORM

EN 12620

September 2002

ICS 91.100.15; 91.100.30

English version

Aggregates for concrete

Granulats pour bétons

Gesteinskörnungen für Beton

This European Standard was approved by CEN on 1 August 2002.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the Management Centre or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Management Centre has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Czech Republic, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Malta, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

EN 12620 requirements for gradation*

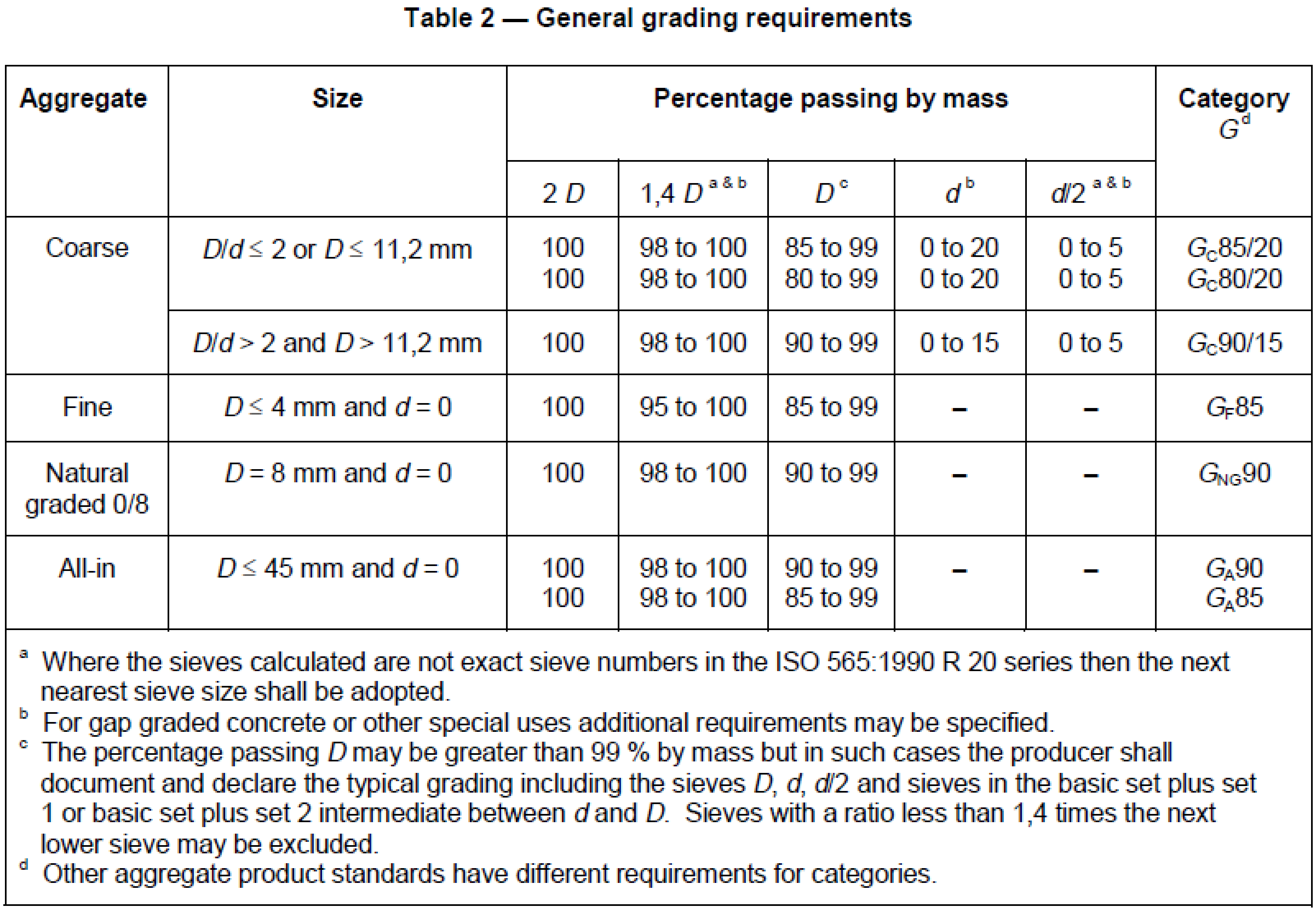


Table 2 — General grading requirements

Aggregate	**Size**	**Percentage passing by mass**					**Category** G^d
		$2D$	$1{,}4D^{a\,\&\,b}$	D^c	d^b	$d/2^{a\,\&\,b}$	
Coarse	$D/d \le 2$ or $D \le 11{,}2$ mm	100 100	98 to 100 98 to 100	85 to 99 80 to 99	0 to 20 0 to 20	0 to 5 0 to 5	$G_C85/20$ $G_C80/20$
	$D/d > 2$ and $D > 11{,}2$ mm	100	98 to 100	90 to 99	0 to 15	0 to 5	$G_C90/15$
Fine	$D \le 4$ mm and $d = 0$	100	95 to 100	85 to 99	–	–	G_F85
Natural graded 0/8	$D = 8$ mm and $d = 0$	100	98 to 100	90 to 99	–	–	$G_{NG}90$
All-in	$D \le 45$ mm and $d = 0$	100 100	98 to 100 98 to 100	90 to 99 85 to 99	–	–	G_A90 G_A85

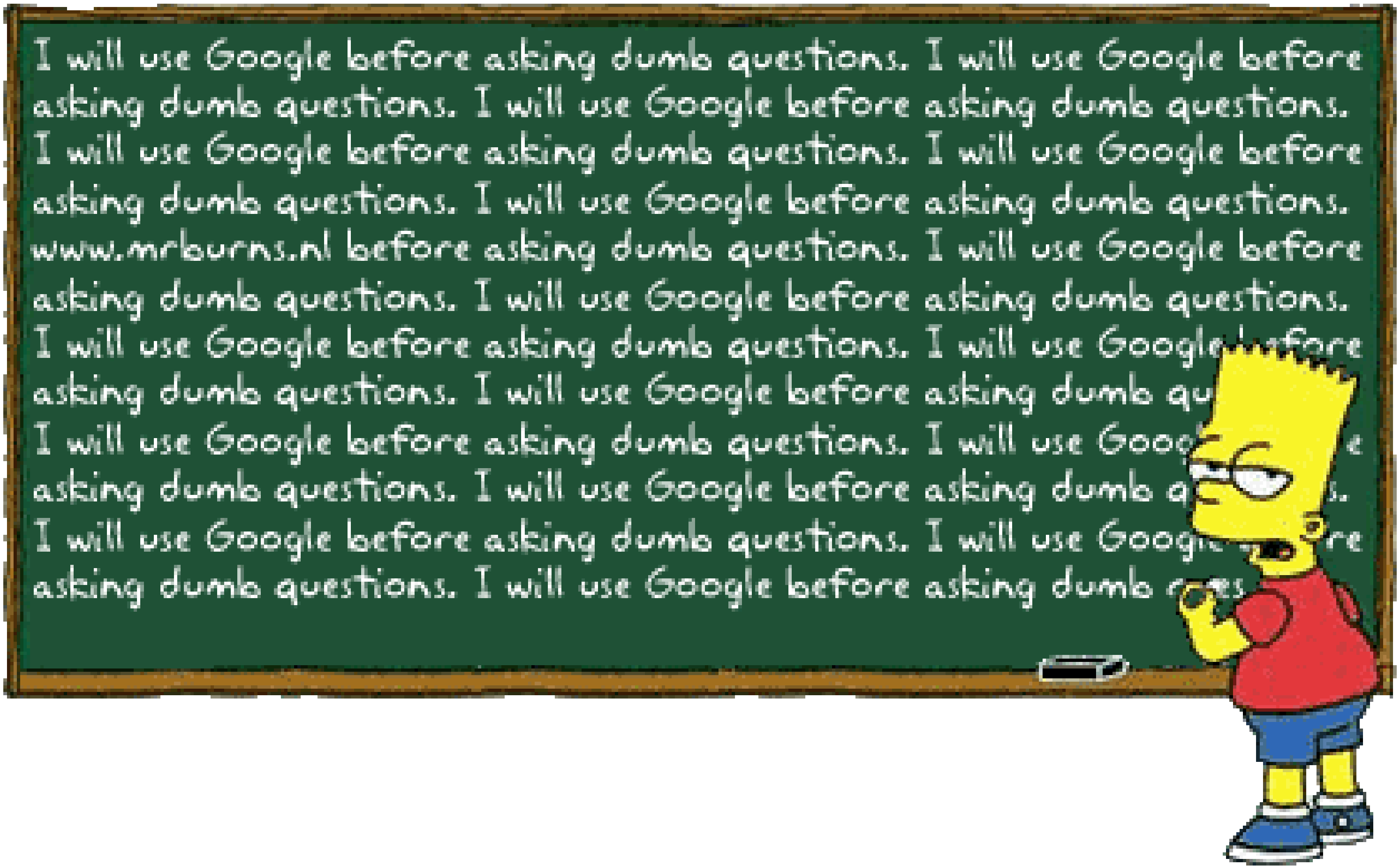
[a] Where the sieves calculated are not exact sieve numbers in the ISO 565:1990 R 20 series then the next nearest sieve size shall be adopted.

[b] For gap graded concrete or other special uses additional requirements may be specified.

[c] The percentage passing *D* may be greater than 99 % by mass but in such cases the producer shall document and declare the typical grading including the sieves *D*, *d*, *d*/2 and sieves in the basic set plus set 1 or basic set plus set 2 intermediate between *d* and *D*. Sieves with a ratio less than 1,4 times the next lower sieve may be excluded.

[d] Other aggregate product standards have different requirements for categories.

* Designation of aggregates in terms of lower (d) and upper sieve sizes are expressed as d/D


I will use Google before asking dumb questions. I will use Google before
asking dumb questions. I will use Google before asking dumb questions.
I will use Google before asking dumb questions. I will use Google before
asking dumb questions. I will use Google before asking dumb questions.
www.mrburns.nl before asking dumb questions. I will use Google before
asking dumb questions. I will use Google before asking dumb questions.
I will use Google before asking dumb questions. I will use Googl efore
asking dumb questions. I will use Google before asking dumb qu
I will use Google before asking dumb questions. I will use Goog e
asking dumb questions. I will use Google before asking dumb q s.
I will use Google before asking dumb questions. I will use Googl re
asking dumb questions. I will use Google before asking dumb q es

CONCRETE TECHNOLOGY

LECTURE NOTES-CHAPTER 3

PROF. DR. KAMBİZ RAMYAR

ADMIXTURES FOR CONCRETE

A material other than,

- ***water,***
- ***aggregates,***
- ***hydraulic cement,***
- ***fiber reinforcement,***

used as an ingredient of concrete or mortar, and added to the batch immediately before or during its mixing.

DEFINITION

Admixture = Additive → synonymous

An addition is a material that is interground or blended in limited amounts into cement during manufacture.

A processing addition helps in the manufacturing or handling of cement and a functional addition modifies the properties of the cement.

Examples of processing addition would be a *grinding aid or a fluxing agent*, and these should not, at least in theory, affect the behavior of the finished cement.

A functional addition is the admixture added to the cement at the plant instead of adding it at the site. For example, a TYPE IA cement has an air-entraining addition that behaves identically to a TYPE I cement to which an air-entraining admixture is added at site.

CLASSIFICATION

Admixtures are generally classified with respect to the characteristic effects of their use as;

- Air-entraining admixtures
- Accelerating admixtures
- Set-controlling admixtures
- Water-reducing admixtures
- Mineral admixtures
- Miscellaneous admixtures.

USES OF ADMIXTURES - 1

A) to modify properties of fresh concrete, mortar, and grout so as to:

- Increase workability without increasing water content or decrease the water content at the same workability,
- Retard or accelerate time of initial setting,
- Reduce or prevent shrinkage or create slight expansion,
- Modify the rate and/or capacity for bleeding,
- Reduce segregation,
- Improve pumpability,
- Reduce the rate of slump loss,

USES OF ADMIXTURES - 2

B) to modify the properties of hardened concrete, mortar, and grout so as to:

- Retard or reduce the early heat evolution
- Accelerate the rate of strength development
- Increase strength
- Increase durability
- Decrease permeability
- Increase concrete-reinforcement bond
- Improve impact and abrasion resistance
- Produce colored concrete or mortar

Considerations in the Use of Admixtures-1

- Admixtures should conform to applicable specifications.
- Careful attention should be given to the instructions provided by the manufacturer.
- The effects of an admixture should be evaluated by means of laboratory investigations whenever possible with the particular materials and conditions of intended use.

Considerations in the Use of Admixtures-2

Such an evaluation becomes particularly important when ;

(1) the admixture has not been used previously with the particular combination of materials;

(2) special types of cements are specified;

(3) more than one admixture is to be used; and

(4) mixing and placing is done at temperatures well outside the common concreting temperatures.

Considerations in the Use of Admixtures-3

Furthermore, it should be noted that

(1) a change in type or source of cement or amount of cement, or a modification of aggregate grading or mix proportions may be desirable;

(2) many admixtures affect more than one property of concrete, sometimes adversely affecting desirable properties;

(3) the effects of some admixtures are significantly changed by such parameters as water content, cement content, aggregate type and grading, and by type and length of mixing.

It should be emphasized that an admixture is not a panacea for poor mix design or sloppy concrete practice, and the user should first ensure that the desired result cannot be attained by improvements in these areas.

Cost Effectiveness for Admixtures-1

Economic evaluation of any given admixture should be based on the results obtained -with the particular concrete in question under conditions simulating those expected on the job.

This is highly desirable since the results obtained are affected significantly by the characteristics of the cement and aggregate and their relative proportions, as well as by temperature, humidity, and curing conditions.

Cost Effectiveness for Admixtures-2

In evaluating an admixture, its effect on the volume of a given batch should be considered. If the admixture changes the yield, as often is the case, the change in the properties of concrete will be due not only to direct effects of the admixture, but also to the changes in the original mix proportions.

All such changes in the composition of a unit volume of concrete must be taken into account when testing the direct effectiveness of an admixture and the benefits resulting from its use.

Cost Effectiveness for Admixtures-3

The increase in cost due to handling an additional ingredient should be considered, as well as the economic effect the use of admixture may have on the cost of transporting, placing, and finishing the concrete. Any effect on rate of strength development and speed of construction should be taken into account.

In addition, placing economies, ability to pump at greater heads, and economies of concrete cost versus competitive building materials must be realized. Water-reducing and set-retarding admixtures permit placement of large volumes of concrete over extended periods, thus minimizing the need for forming, placing, and joining separate units. Accelerating admixtures reduce finishing and forming costs.

AIR-ENTRAINING ADMIXTURES

An important disadvantage of concrete is its susceptibility to freezing and thawing cycles when it is in a saturated or nearly saturated condition. Concrete can be badly damaged after a single winter if necessary measures are not taken.

This deterioration would preclude the use of concrete in numerous applications such as pavements, dams, foundations, and so on. Fortunately, concrete can be made freeze-thaw resistant by the use of air-entraining admixtures.

Freeze-Thaw Effect

100x100x100 mm Concrete Cube Specimens after 140 Freeze-Thaw Cycle

Cement Dosage
350 kg/m³

No strength loss is observed for air-entrained samples,

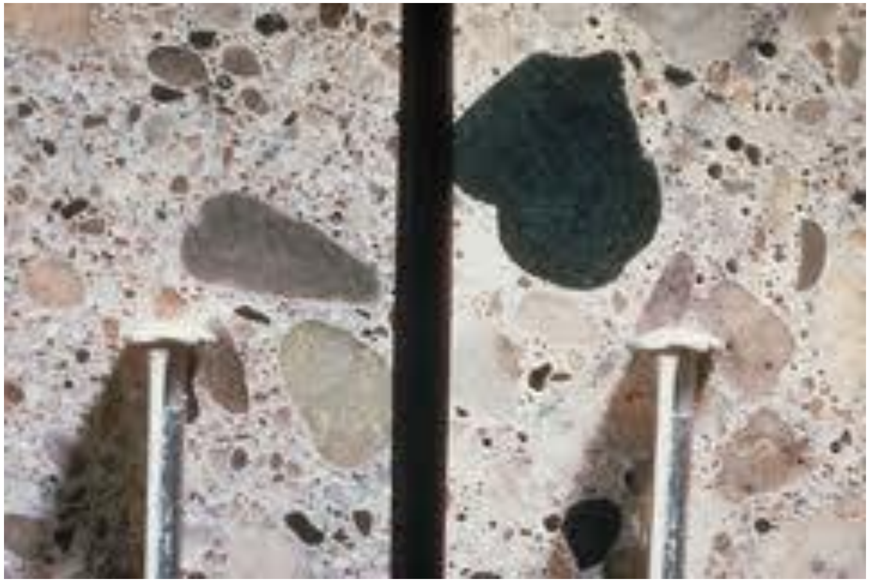
Control samples without air entrainment can easily be broken even by hand

The Entrained Air-void System

The volume of air required to give optimum freeze-thaw resistance is approximately 9% by volume of the mortar fraction of concrete.

In other words, the air content must be in the range of 4 to 8% by volume of concrete for satisfactory protection.

The actual amount depends on the D_{max} of the coarse aggregate because more paste is required for a concrete having lower maximum aggregate size and the air is actually entrained in the cement paste.

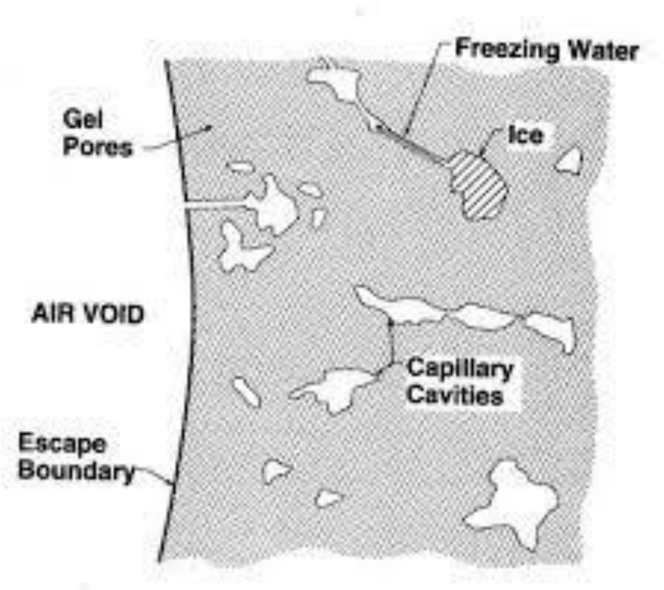


The Entrained Air-void System

The nature of the air-void system is also important.

The air-entraining admixture causes the mixing water to foam, and this foam is locked within the paste during hardening.

The critical parameter of the air-entrained paste is the spacing factor, which is defined as the average maximum distance from any point in the paste to the edge of a void. The spacing factor should not exceed 0.2 mm. The smaller the spacing factor, the more durable is the concrete. The voids should be small (0.05-1.25 mm diameter) to ensure that the required spacing factor is obtained at relatively low air contents.



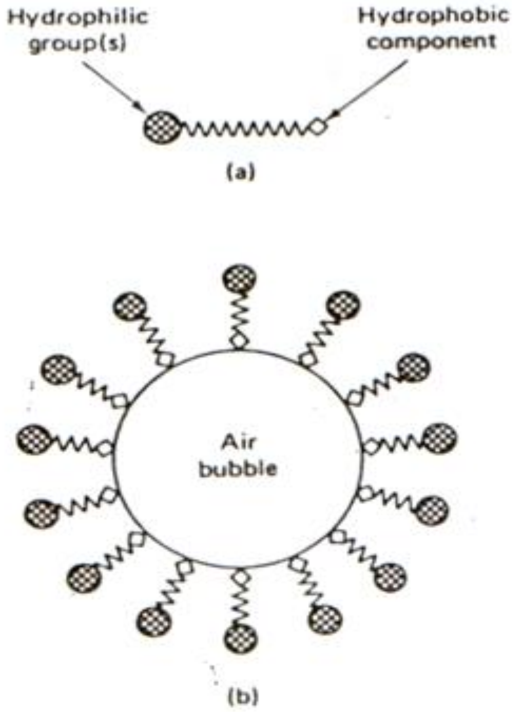
Air-entraining materials

The air-entraining agents contain compounds that will promote the formation of stable foam within concrete.

Bubble formation is normally a transient phenomenon. Foams can only be permanent if the surface tension of the water can be overcome.

Air-entraining admixtures contain surface-active agents which concentrate at the air-water interface, lower the surface tension so that bubbles can form, and stabilize them once they are formed.

Surface-active agents are molecules which at one end have chemical groups that tend to dissolve in water [hydrophilic = water loving groups], and which at the other end have water repelling groups [hydrophobia = water hating groups]



Effect of Air Entrainment on Fresh Concrete

Air entrainment improves the workability and cohesiveness of fresh concrete. Air-entrained concrete is more workable than a non-air-entrained mix at the same slump. Bleeding and segregation are considerably reduced in air-entrained concretes.

Keeping everything else constant, a 5% air entrainment increases the slump of concrete about 15 to 50 mm. The reason for improved workability is that air bubbles behave as low-friction, elastic, fine aggregate which reduces interactions between solid aggregate particles.

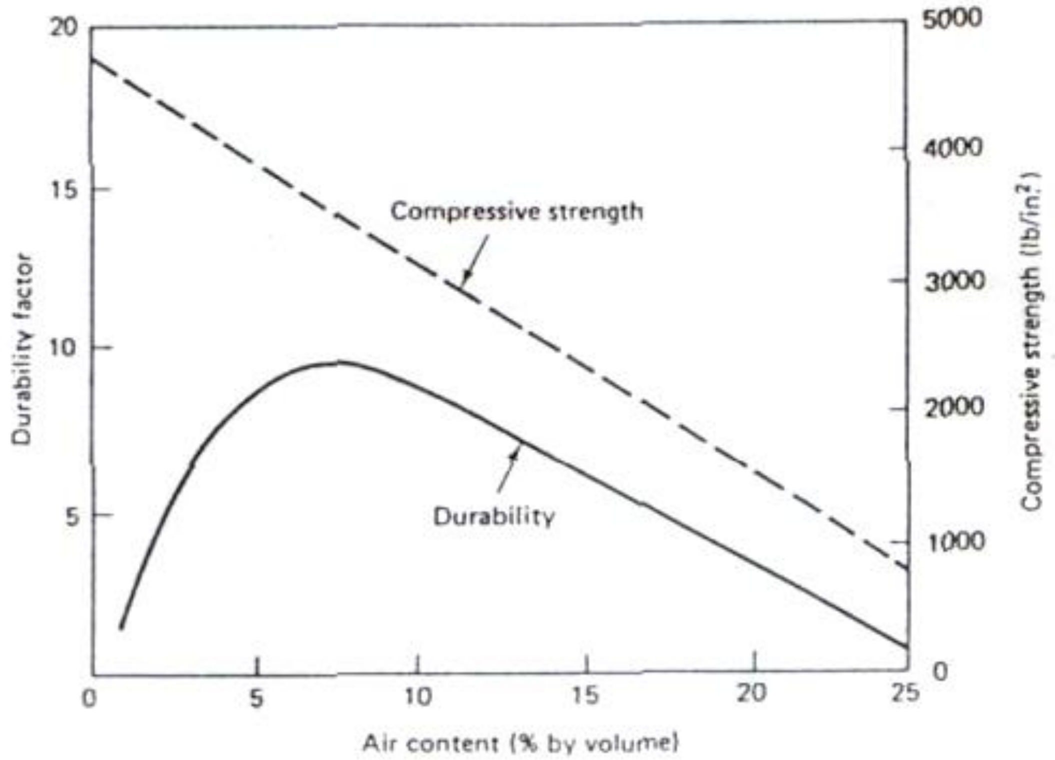
Water content of a concrete mix can be reduced 20-30 kg/m^3 for a 5% increase in air content. This partially offsets the strength reduction that accompanies the air-entrainment.

Effect of Air Entrainment on Hardened Concrete

The introduction of additional void space with air entrainment results in a 10-20% decrease in strength. However, air-entrainment may not affect the strength of lean concretes (cement content < 300 kg/m^3) since the adverse effect can be overcome by the reduction in W/C ratio due to increased workability.

Excessive amounts of air will drastically lower the strength and reduce the resistance to freezing. The high air contents generally indicate larger bubble sizes rather than smaller spacing factors.

Durability factor vs. air content



$$D.F = \frac{PN}{300}$$

P: percentage of initial dynamic modulus after N cycles

ACCELERATING ADMIXTURES

Set-accelerating admixtures: Normal or Rapid?

Very rapid setting agents are used for plugging leakages and for emergency repair where rapid development of rigidity is required. (shotcrete admixtures)

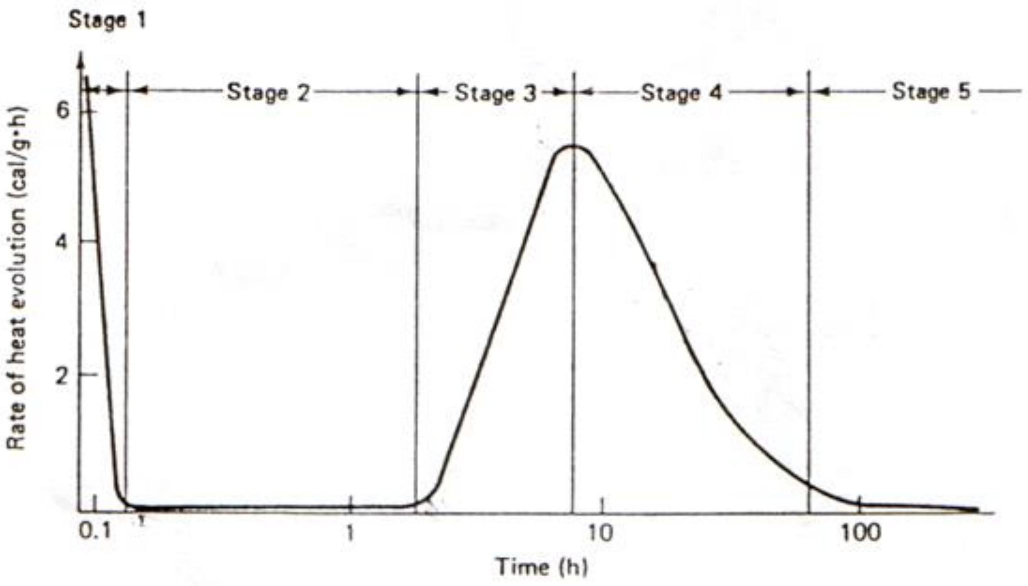
The conventional accelerators are used to speed up construction by permitting;

- early finishing,
- early removal of forms,
- early strength development.

Accelerators are beneficial during winter by overcoming the slower rate of development at low temperatures and by shortening the period for protection against freezing damage.

ACCELERATING ADMIXTURES

Accelerators increase the rate of hydration of C_3S by shortening the dormant period. They increase the early strength of concrete, the rate and the extent of this increase depends on the type and dosage of the accelerating agent. However, the late strength may be lower than the strength of concretes without any accelerating agent.



ACCELERATING ADMIXTURES

Accelerating agents may increase the rate of drying shrinkage and creep but not their ultimate value.

Accelerators usually help to produce smaller air bubble sizes and smaller spacing factors in air-entrained concrete which lead to a desireable air-void system for frost resistance of concrete.

RETARDING ADMIXTURES

Retarders can be used whenever it is desirable to offset the effects of high temperatures which decrease the setting time, or to avoid complications when unavoidable delays may occur between mixing and placing.

Prolonging the plasticity of fresh concrete can be used to advantage in placing mass concrete. Successive lifts can be blended together by vibration, with the elimination of cold joints that would occur if the first lift were to harden before the next were placed.

RETARDING ADMIXTURES

Retarders have exactly the opposite action that accelerators have: They decrease the rate of hydration of C_3S and elongate the dormant period. It must not be forgotten that when the retarder dosage exceeds a certain critical value, C_3S hydration will never proceed beyond stage 2 and the cement paste will never set.

The retarding power of an admixture increases when its addition to concrete is delayed a few minutes after the first addition of water. When a retarder is used, 1-day strength reduces. However, this difference is closed within seven days, unless an overdosage is used. Retarding admixtures increase the rate of drying shrinkage and creep but not the ultimate values.

WATER-REDUCING ADMIXTURES

A water-reducing admixture lowers the water requirement for a given slump. This property can be used to advantage in several ways. Achieving the desired slump with less water at a constant cement content means an effective lowering of the W/C ratio with consequent improvements in strength, water-tightness, and durability.

Alternatively, the desired slump may be attained without changing the W/C ratio by lowering the cement content.

Finally a water-reducing admixture may be used to increase slump without changing the water and cement contents, to facilitate difficult placements.

WATER-REDUCING ADMIXTURES

Water reducing admixtures are divided into two groups as

(1) plasticizers and

(2) superplasticizers.

The formers achieve 5-10% water reduction whereas the latters achieve 15-30%. The water reduction at any dosage of a water-reducing admixture depends on;

(a) cement fineness,

(b) mix proportions,

(c) temperature of concrete,

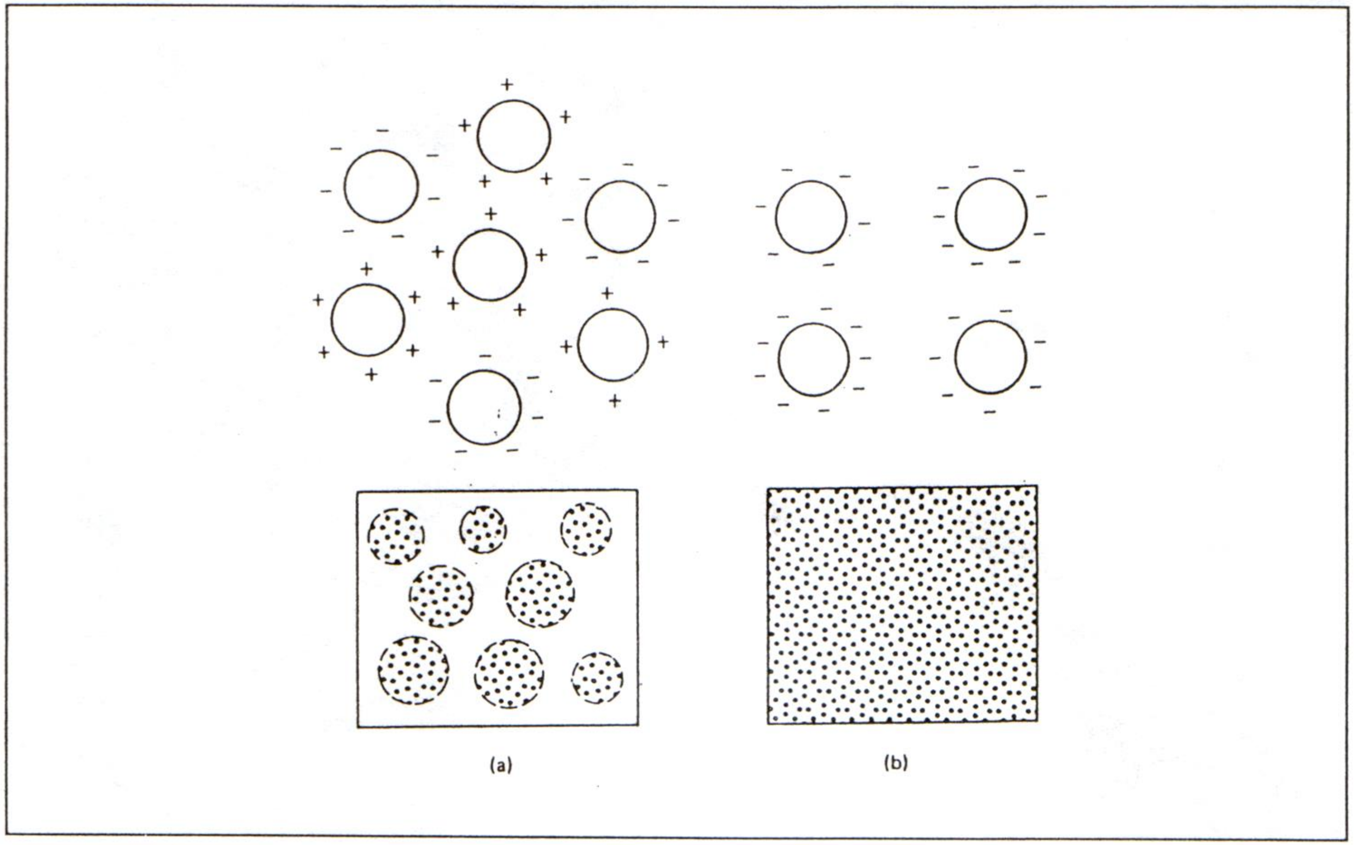
(d) time of addition.

WATER-REDUCING ADMIXTURES

The basis of water reduction for all three categories is that they are adsorbed at the solid-water interface. Cement particles carry charges on their surfaces, which may be positive or negative or both. The opposing charges on the adjacent particles exert electrostatic attractions causing particles to flocculate.

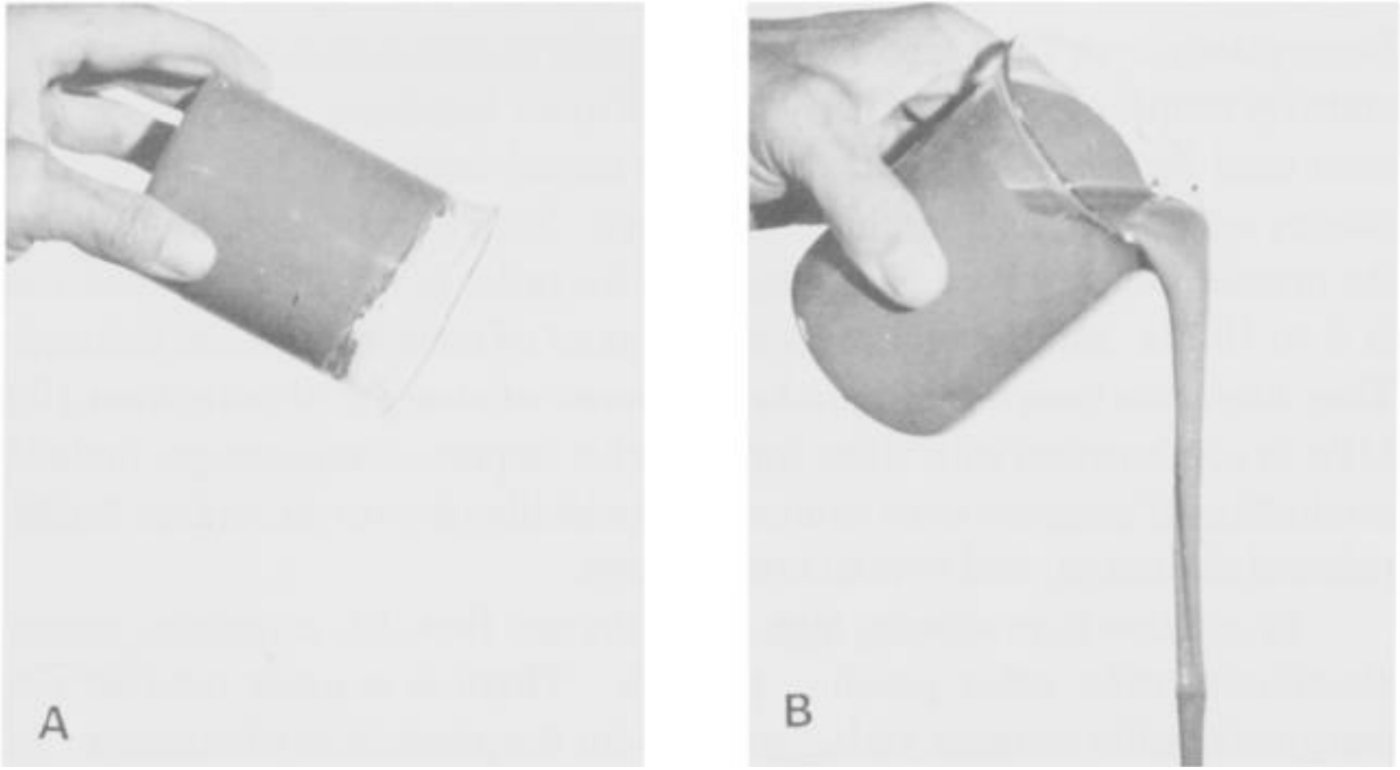
The molecules of water-reducing agents attach themselves to cement particles and either neutralize these charges or cause all surfaces to carry uniform charges of like sign. Thus, particles repel each other and remain fully dispersed in the mix.

Mechanism of Water Reducer Action



(a) (b)

Effects on the water–cement system



Effect of superplasticizer on cement.

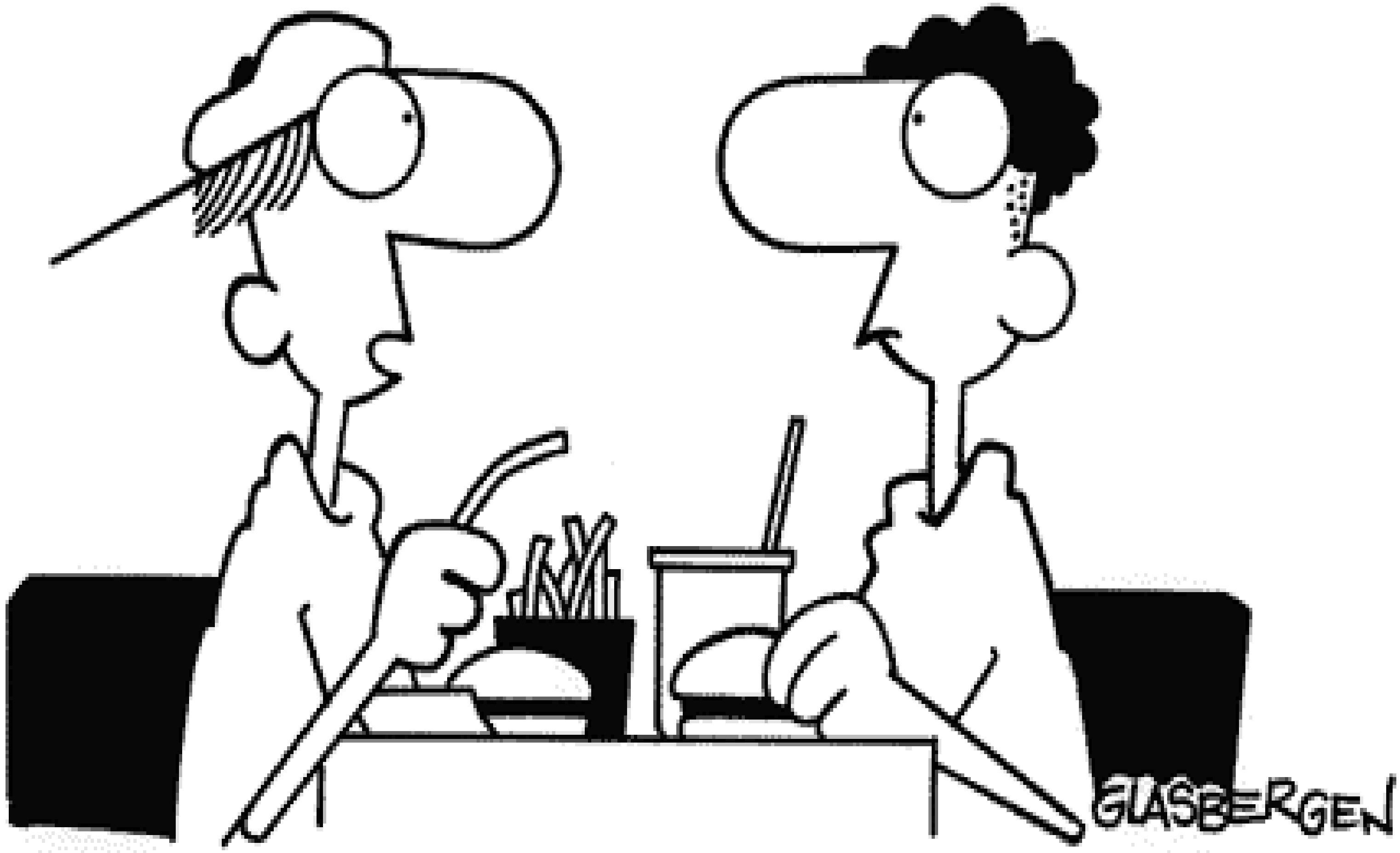
A: Cement + water.

B: Cement + water+ superplasticizer.

MISCELLANEOUS ADMIXTURES

There are many other types of admixtures that are commercially available. The consumption of these various materials added together is less than the amount used of any single type discussed so far. Some of these miscellaneous admixtures are as follows:

- Bonding admixtures: used for bonding new concrete surfaces to the old ones.
- Corrosion inhibitors
- Dampproofing admixtures
- Expansion-producing admixtures.



"I forgot to make a back-up copy of my brain, so everything I learned last semester was lost."

CONCRETE TECHNOLOGY

FRESH CONCRETE

The properties of fresh concrete are important because;

(1) they affect the choice of mixing, handling, and compaction equipments,

(2) they are responsible for the behavior of fresh concrete during these processes,

(3) they affect the properties of hardened concrete.

When making a concrete, following criteria must be considered;

(1) The long term requirements of hardened concrete such as strength, durability, and volume stability;

(2) Short term requirements, while the concrete is still in plastic state, which are generally named as "workability" requirements.

The fresh concrete must satisfy the following requirements:

(1) It must be easily mixed and transported

(2) It must be uniform throughout a given batch and between batches

(3) It should be capable of filling the forms completely and thoroughly

(4) It must be capable of being compacted properly without excessive effort

(5) It must not segregate during placing and compaction

(6) It must be capable of being finished properly.

WORKABILITY

It is often defined in terms of the amount of mechanical work, or energy, required to produce full compaction of concrete without any segregation.

RHEOLOGY

Rheology is the science dealing with the deformation and flow of materials under stress.

Basic Principles of Rheology-1

A liquid can not support a shear stress and flows irreversebly and continuously when a stress is applied; Viscosity is a characteristic property which determines the flow of any fluid. In other words, it is the resistance of a fluid during flow.

For an ideal liquid, the strain, γ, is related to the shear stress, τ, by Newton's Law:

$$\tau = \eta \frac{d\gamma}{dt}$$

where, η is coefficient of viscosity.

It is obvious from the above equation that this relationship is linear for Newtonian Liquids. Therefore, if concrete were approximated as a Newtonian liquid a single measurement of the stress, τ, and the rate of strain, dγ/dt, would describe the workability.

Basic Principles of Rheology-2

When solid particles are introduced into Newtonian liquids, viscosity increases. For heavily concentrated suspensions, as in the case of concrete in which the solid particles concentration in the liquid medium is very high, the coefficient of viscosity is determined by the following equation:

$$\eta = \eta_o (1 + C_1 \Phi + C_2 \Phi^2)$$

where η is the viscosity of the suspension, η_o is the viscosity of the pure liquid, and Φ is the volume fraction of the dispersed phase.

Basic Principles of Rheology-3

Besides the increased viscosity, the flow behavior is also changed. The fresh concrete has definite shear strength, τ_o, that must be exceeded to start flow (as the yield strength in metals). Thus, the rheology of fresh concrete can be described by

$$\tau - \tau_o = \eta \frac{d\gamma}{dt}$$

Furthermore, fresh concrete has a **thixotropic** character (a decrease in viscosity under increasing stress).

This characteristic becomes important during placing and compaction.

Basic Principles of Rheology-4

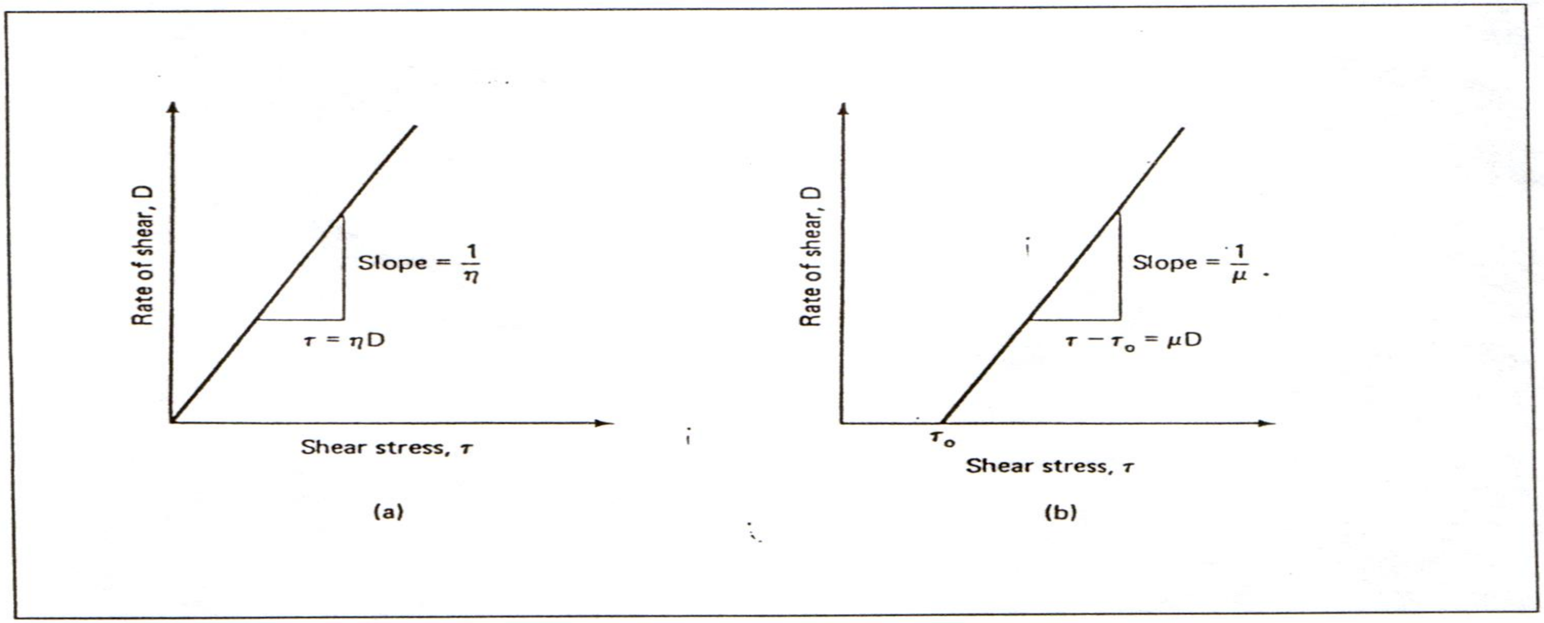


FIG. 4.1. Flow of (a) a Newtonian liquid and (b) a suspension

Finally, the behavior of fresh concrete can be reasonably approximated by a model named **Bingham Body** and therefore, two pairs of values of x and dy/dt must be determined to define the straight line of FIG.4.1.b

Concrete has some special properties compared to other materials;

(1) It is an agglomerate which is heterogeneous even at macroscopic scale;

(2) Its properties change with age and moisture movements;

(3) It has microcracks even before loading, and after loading, presence of microcracks are not considered unusual;

(4) It is basically made up of two rheological phases:

(a) the aggregate phase (skeleton) which is crystalline and more elastic

(b) the cement paste phase which is poorly crystalline or amorphous and more viscous;

(5) It can be both elastic and inelastic, and ductile and brittle.

Rheological Model for Fresh Cement Paste-1

Rheologically, fresh cement paste can be represented by Bingham Body. That is, after a certain value of yield stress, continuous deformation occurs.

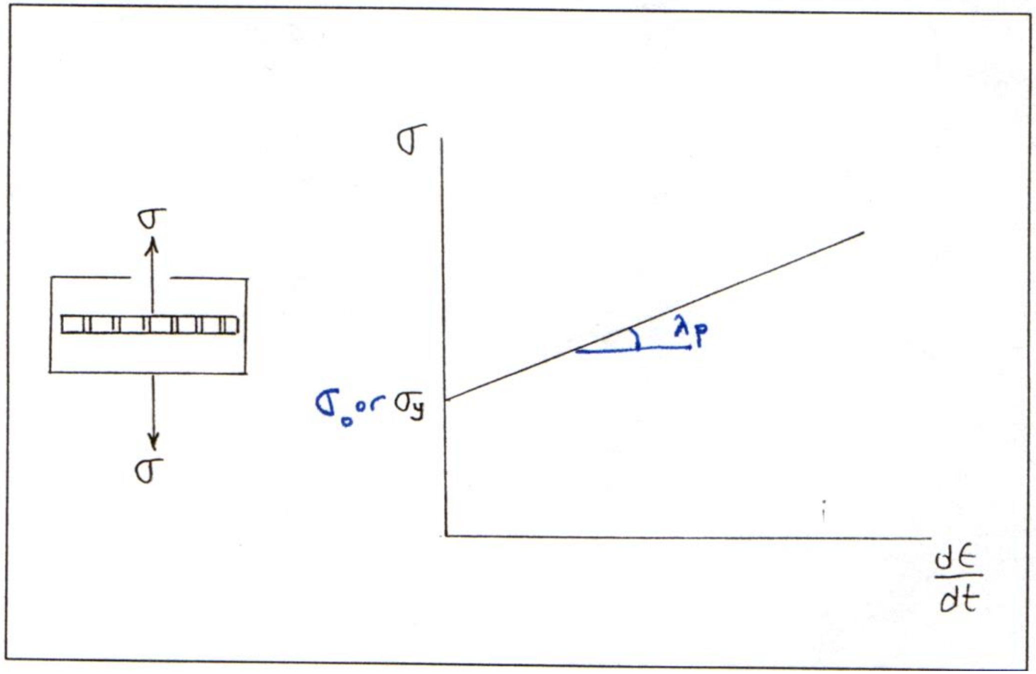


FIG. 4.2. Rheological model (Bingham Body) for fresh cement paste.

The behavior illustrated in FIG. 4.2. can be described by the following equation:

$$\sigma - \sigma_o = \lambda_p \frac{d\varepsilon}{dt}; \quad \lambda_p = 3\eta_p$$

for normal stresses and strains

where, λ_p is the coefficient of viscous fraction of the paste.

Rheological Model for Fresh Cement Paste-2

The flow behavior of fresh cement paste depends on

(a) W/C ratio;

(b) degree of hydration;

(c) particle size and particle size distribution of cement;

(d) volume concentration of solids in the paste;

(e) presence of admixtures;

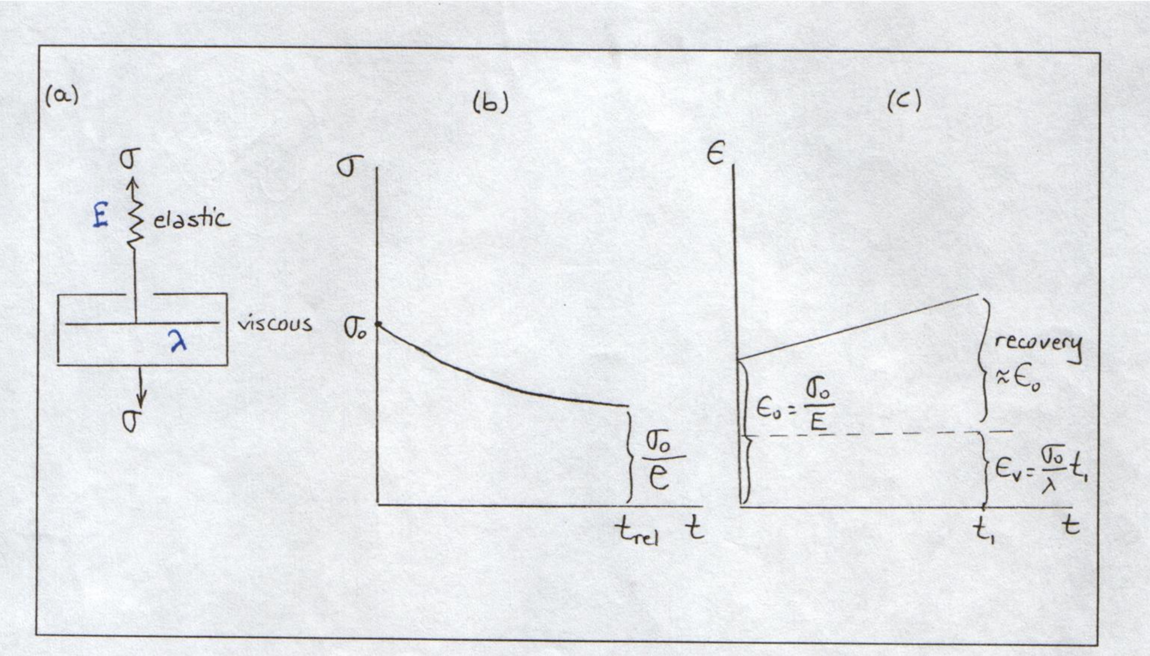
(f) temperature.

Rheological Model for Hardened Cement Paste-1

Hardened cement paste can be represented by **Maxwell** Model. That is, there is the combined action of elastic and viscous elements. The spring in the rheological models represents the elastic behavior and the dashpot represents the viscous behavior. The load on each element (spring and dashpot) is the same but the deflections differ.

$$\sigma_{spring} = \sigma_{dashpot}$$

$$\varepsilon_{spring} \neq \varepsilon_{dashpot}$$



Rheological model for hardened cement paste: (a) Maxwell body, b) constant strain condition, c) constant stress condition.

Rheological Model for Hardened Cement Paste-2

When the force is applied, the spring responds immediately by an amount

$$\varepsilon_1 = \frac{\sigma}{E}$$

while, at the same moment, the piston starts to move and the displacement of the piston at time t is given by

$$\varepsilon_2 = \int_0^t \frac{\sigma}{\lambda} dt$$

Since the total displacement is the sum of ε_1 and ε_2,

$$\varepsilon = \frac{\sigma}{E} + \int_0^t \frac{\sigma}{\lambda} dt$$

Rheological Model for Hardened Cement Paste-3

One important mode of behavior occurs when a load is suddenly applied to a Maxwell unit which is then held at a fixed elongation. If the strain is ε_o, the initial stress becomes $\sigma_o = E\varepsilon_o$. Since ε is constant, dε/dt=0. Therefore,

$$\frac{d\varepsilon}{dt} = \frac{\frac{d\sigma}{dt}}{E} + \frac{\sigma}{\lambda} = 0$$

Hence,

$$\sigma = \sigma_o e^{\frac{-E}{\lambda}t}$$

It means that if the body is left to itself under a constant strain, the stress gradually disappears **(relaxes). The Relaxation Time t_{rel}, is defined as the time required for the stress to decrease 1/e of its original value.**

Rheological Model for fresh mortar

Bingham's Body may be used to describe the viscoelastic behavior of fresh mortar. However the effects of fine aggregate should be considered.

Rheological Model for hardened mortar

Maxwell Model may be used.

Rheological Model for fresh concrete

Bingham's Body may be used as an approximation.

Rheological Model for hardened concrete

The viscoelastic behavior of hardened concrete is represented by the series combination of Maxwell and Kelvin Bodies.

$$\varepsilon_t = \varepsilon_e + \varepsilon_d + \varepsilon_v$$

$$\varepsilon_e = \frac{\sigma}{E_1},$$

$$\varepsilon_d = \frac{\sigma}{E_2}\left(1 - e^{\frac{-t}{\lambda_2 / E_2}}\right),$$

$$\varepsilon_v = \frac{\sigma}{\lambda_1} t$$

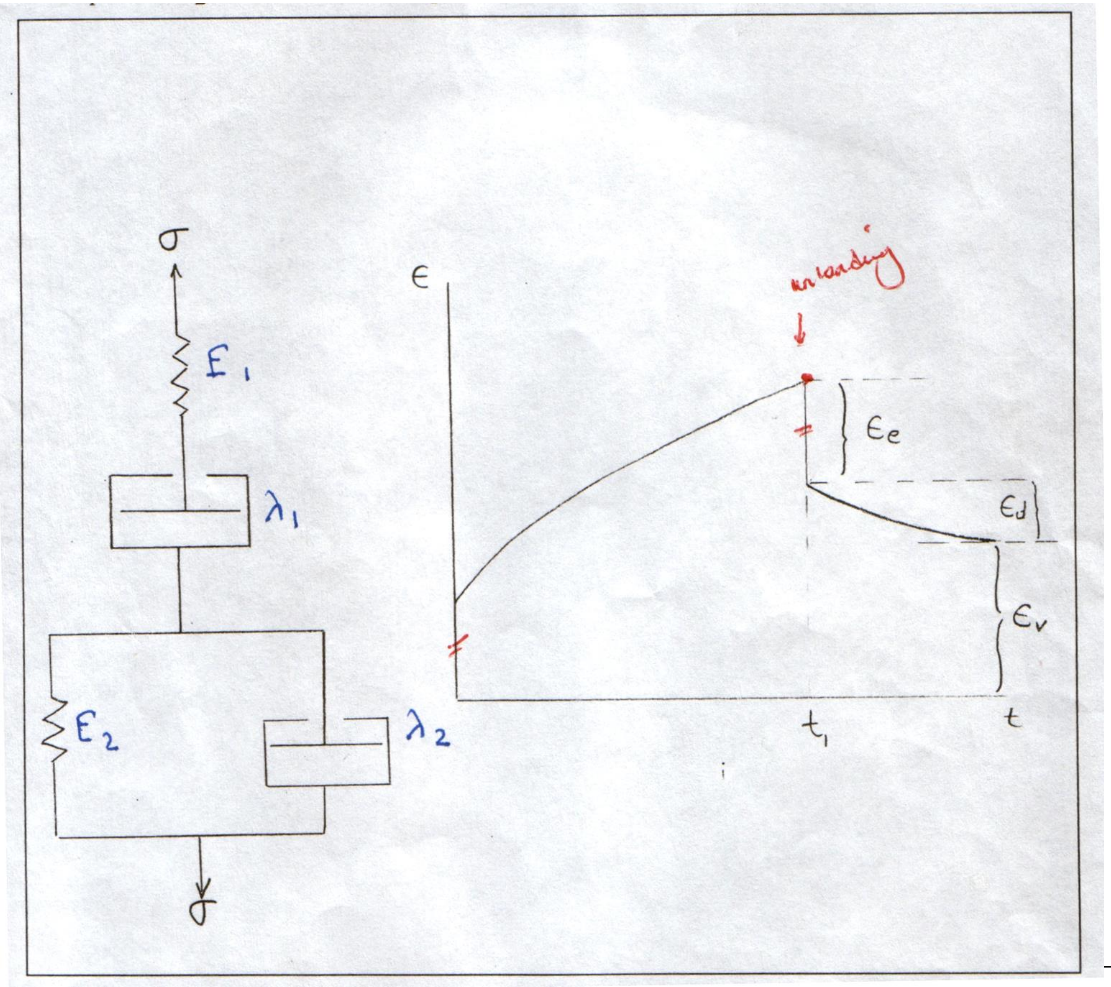


FIG 4.4 Representation of the viscoelastic behavior of hardened concrete by 4-Element Model

Factors Affecting Workability-1

Water content of the mix: Increasing the water content will increase the ease with which concrete flows. However, besides the disadvantage of reduced strength, increased water may lead to segregation and bleeding. For optimum workability, finer aggregate gradings require higher water contents due to increased surface area to be "**wetted**".

Aggregate content and fine and coarse aggregate proportions: For a constant water-cement ratio, an increase in the aggregate/cement ratio decreases workability; also more cement is needed when finer aggregate gradings are used. A deficiency in fine aggregate will result in a harsh mix which is prone to segregation and difficult to finish. On the other hand, an excess of fine aggregate will lead to a more permeable and less economical concrete, although the mix is easily workable.

Factors Affecting Workability-2

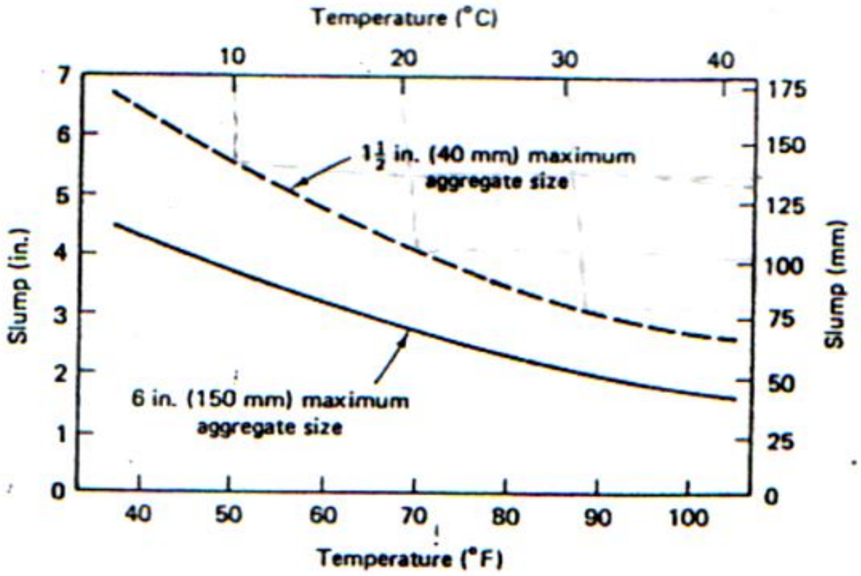
Shape and surface texture of aggregate particles: Generally, the more nearly spherical the aggregate particles, the more workable the resulting concrete will be. This is partly due to the fact that the spherical particles act as "**ball bearings**" while angular particles will have more interlocking and will therefore require more work to overcome the internal friction between the aggregate particles.

Besides, spherical particles have lower surface-to-volume ratio, and less mortar is needed to coat the particles. When flat and elongated particles are present in aggregate, the amount of sand, cement, and water must be increased. Also, smooth particles tend to increase workability whereas rough ones reduce it.

The porosity of aggregate may also affect workability. If the absorption is high, less water will be available for workability. Therefore, it is necessary to distinguish between the total water content of a concrete mix and the free (net) water content which is available to provide workability, and which determines the W/C ratio.

Factors Affecting Workabilitv-3

Time and temperature: As the ambient temperature increases the workability decreases. Higher temperatures will increase both the evaporation rate and the hydration rate. Therefore, in hot weather concreting more water should be used to maintain a constant workability.



During the period of plasticity there will be a steady decrease in workability with time. This is partially caused by the hydration of C_3S and C_3A which continues slowly even during the dormant period, and partially by the loss of water from the concrete through evaporation or absorption. This **slump loss** is approximately linear with time, although it is the greatest in the first 1/2 hour after mixing. Since the slump at the time of placement is important, the slump loss of concrete must be considered when making concrete mix design calculations. Slump loss increases when concrete temperatures are high.

Factors Affecting Workability-4

Cement characteristics: The cement characteristics are much less important in determining workability. However, the increased fineness (as in the case of high-early-strength cements) will reduce workability at a given W/C ratio because finer cements will require more water due to their high specific surface and because they also hydrate more rapidly.

If the cement is delivered to the job hot, there occurs a considerable loss of workability, due to more rapid hydration and evaporation of water. In addition, at temperatures higher than 60°C, flash set may occur.

Admixtures: Different types of admixtures have already been discussed. Mineral admixtures are sometimes used to supply additional fine material to harsh mixes. Air-entraining, water-reducing, and set-retarding admixtures improve workability.

Segregation -1

Segregation is the separation of the components of fresh concrete, resulting in a non-uniform mix. In general, this means the separation of some of the coarse aggregate particles from the mortar.

Segregation can be of two types:

(1) settling of heavy particles to the bottom of the fresh concrete,

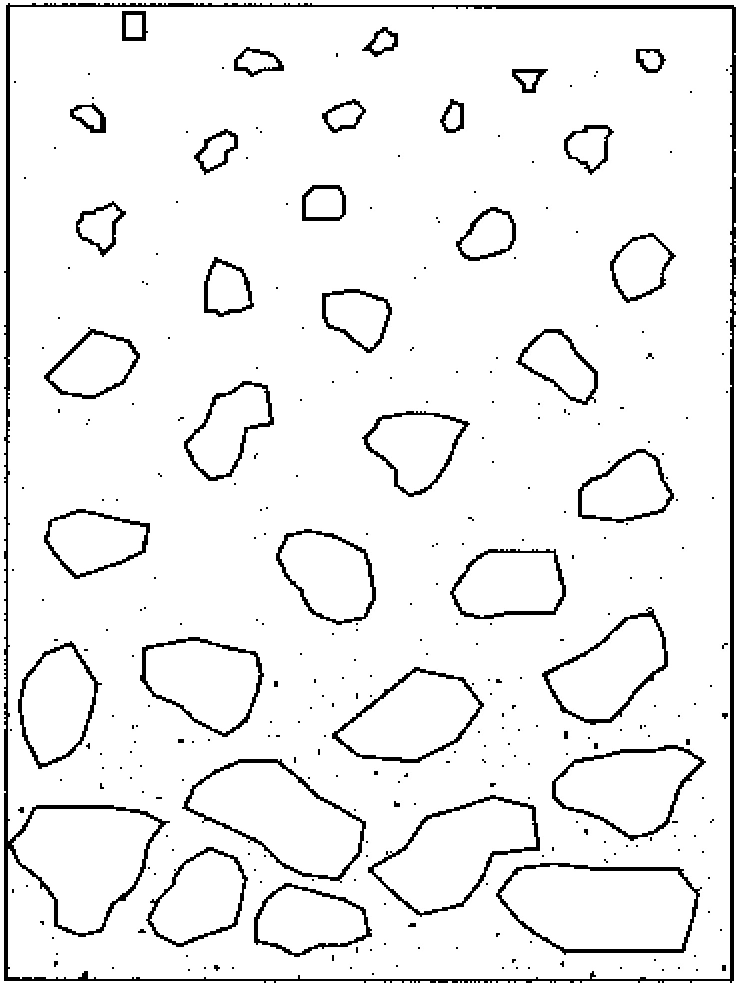
(2) separation of coarse aggregate particles from the concrete body by improper placing and compaction.

Segregation -2

The factors that contribute to increased segregation are as follows:

- Large maximum aggregate sizes (>25mm) and high proportions of large aggregate pieces
- High specific gravity of coarse aggregate compared to that of the fine aggregate
- Decreased amount of fines (sand and cement)
- Changes in the particle shape and texture from smooth, well-rounded to rough, irregular
- Mixes that are either too dry or too wet.

Segregation



Improperly consolidated Concrete

Bleeding -1

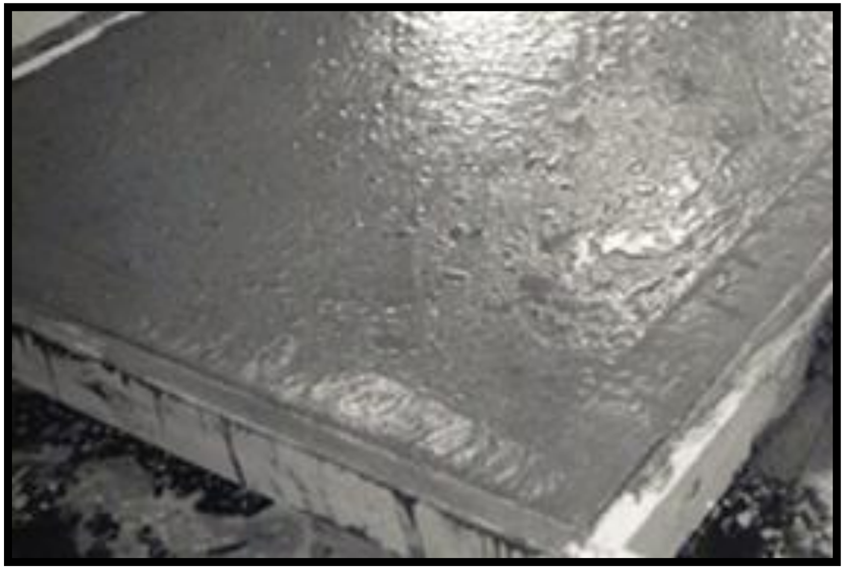
Bleeding is the tendency of water to rise to the surface of freshly placed concrete. Bleeding is caused by the inability of the solid ingredients to hold the mix water when they settle down.

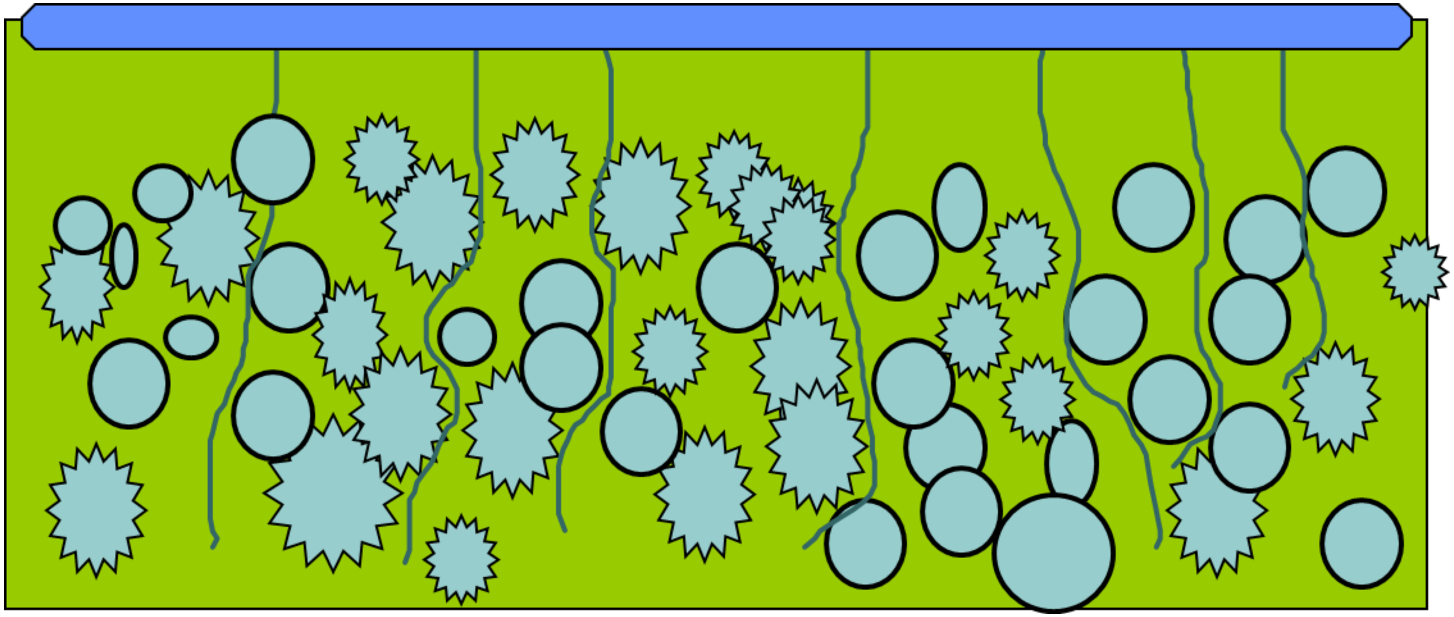
Undesirable effects:

1. As a result of bleeding, the top portion of concrete becomes overly wet, porous and weak, liable to disintegration by freezing.
2. Water rising to the surface carries fine particles of cement causing “**laitance**” which prevents the bond of subsequent layer of the concrete.
3. Water accumulates under the coarse aggregate and reinforcement, resulting in large voids in these zones which prevent formation of a bond between them and cement paste.
4. The capillaries formed upon bleeding increase the permeability and thus the danger of frost damage.

This can be avoidable by delaying the finishing operation until the bleeding water has evaporated.

Bleeding





Bleeding -2

If the bleed water evaporates more rapidly than the bleeding rate, plastic shrinkage cracks will form.

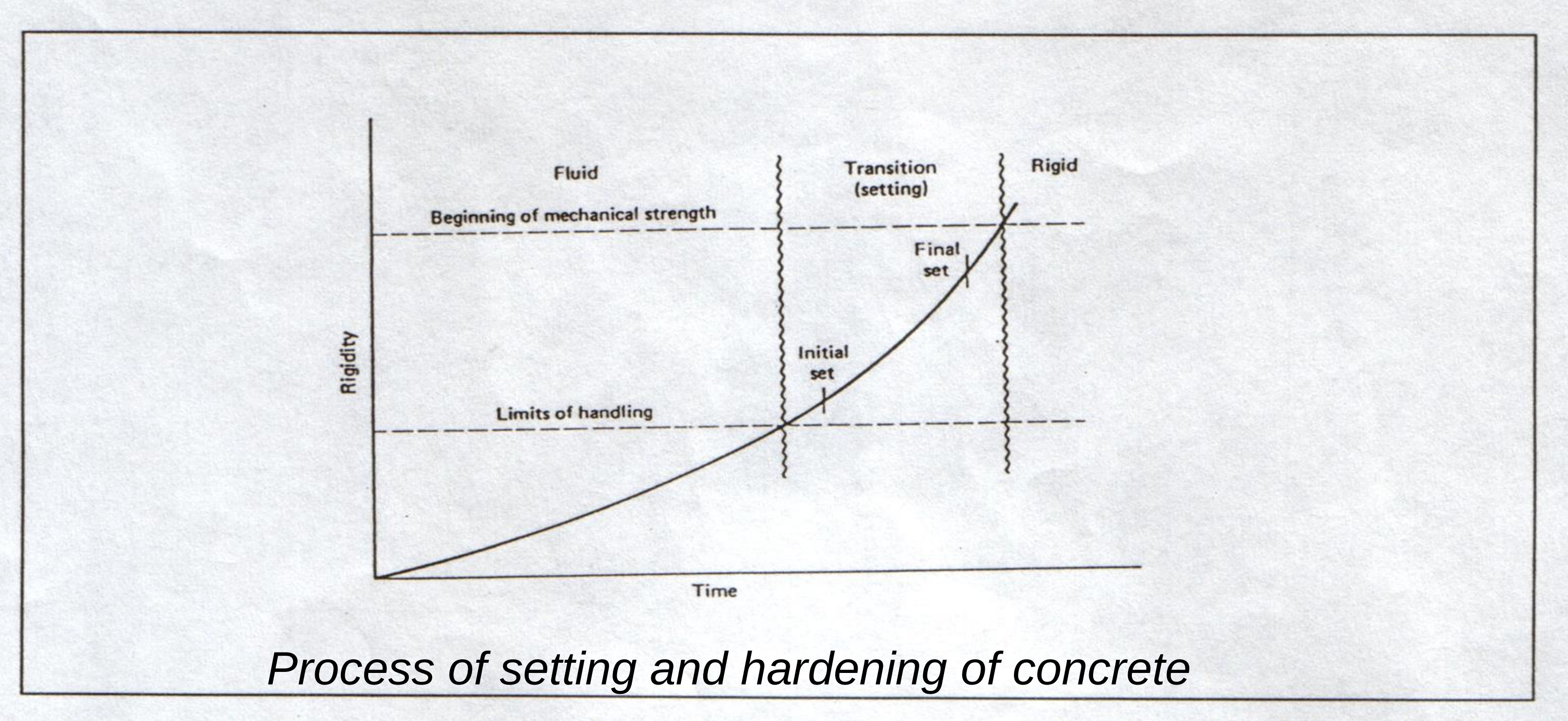
The tendency of bleeding depends largely on the properties of cement.

It can be reduced in a number of ways:

- reducing the water content of the mix (if possible)
- increasing fineness of the cement
- using finely divided mineral admixtures
- using high C_3A content or high alkali content cement
- using rich mixtures
- using air-entrainment admixture

SETTING OF CONCRETE

Setting is defined as the onset of rigidity in fresh concrete. It should not be mixed up with hardening which describes the development of measurable strength. So setting can be viewed as a transitional period between true fluidity and true rigidity.



Process of setting and hardening of concrete

Effect of Hydration on Setting

Effect of C_3S

The period of fluidity of a cementitious mixture corresponds to the dormant period of C_3S hydration.

Setting occurs when the dormant period ends. Initial set corresponds approximately to the beginning of stage 3 and final set corresponds to its midpoint.

Effect of C_3A and gypsum

In standard types of portland cements, C_3A has a minor role in determining the setting behavior. Gypsum present in the cement reacts with C_3A to help control the setting time.

Abnormal Setting-False Set

A cementitious system may stiffen rapidly a short time after mixing is completed. The fluidity of the system is restored by remixing and the cementitious system will then set normally. This phenomenon is sometimes called **plaster set** because it is generally caused by the formation of gypsum. When gypsum is ground with a hot clinker it may partially dehydrate to hemihydrate.

$$CaSO_4.2H_2O \xrightarrow{100\text{–}190^o C} CaSO_4 \cdot \frac{1}{2} H_2O + \frac{3}{2} H_2O \uparrow$$

When water is added to the cement the hemihydrate will rehydrate to gypsum and form a rigid mass. However, since there is small amount of gypsum in concrete, setting can easily be disrupted by further mixing.

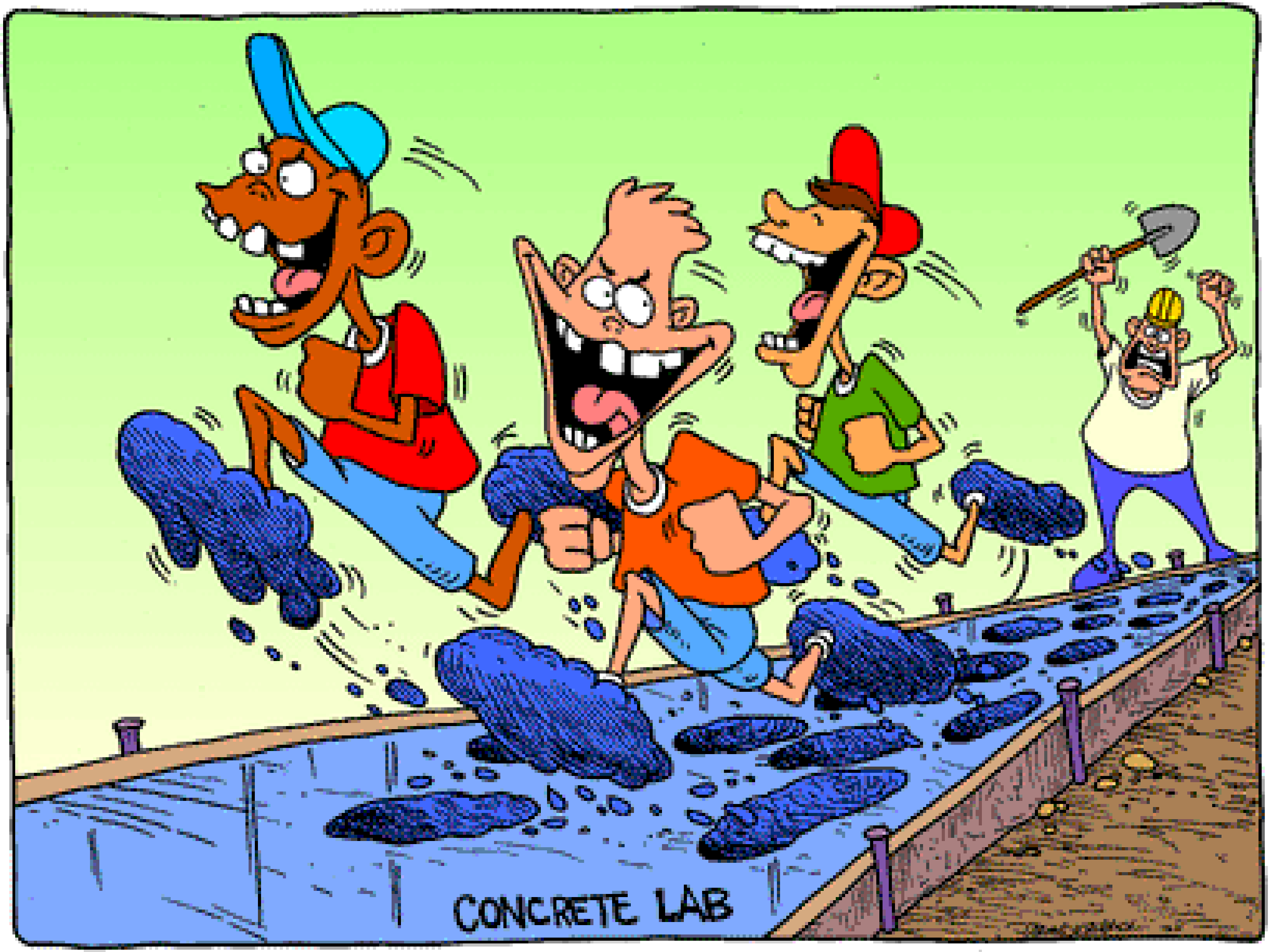
Another form of false set may result from the carbonation of alkali oxides in cement during storage. When the cement contacts with water C_3S starts to hydrate releasing calcium hydroxide. CH reacts with the alkali carbonates and a base exchange occurs resulting in $CaCO_3$. This precipitates and induces a rigidity to the cementitious system.

Flash Set

Flash set is the rapid development of rigidity in a freshly mixed cement paste, usually with the evolution of considerable heat, which rigidity cannot be overcome nor plasticity be regained by further mixing without the addition of water.

Flash set is due to the reaction of C_3A (more than the limited value) with water when there is not sufficient amount of gypsum. Upon this very rapid reaction, crystalline tricalcium aluminate hydrates form.

Amount of gypsum must be such that it will be used up to-at most-hardening, because expansion caused by ettringite can be distributed throughout the paste before hardening.


CONCRETE LAB

CONCRETE TECHNOLOGY

LECTURE NOTES
PROF. DR. KAMBİZ RAMYAR

TURKISH STANDARD (TS 802) METHOD OF MIX DESIGN

Step 1. Determination of Maximum Aggregate Size:

The maximum size of aggregate should be chosen such that it will be smaller than either 1/5 of the smallest dimension of the beams and columns or 1/3 of the slab thickness or 3/4 of the clear spacing between the reinforcement. A recommendation for maximum aggregate sizes that can be used for certain concrete members are given in TABLE TS-1.

TABLE TS-1. Maximum Aggregate Sizes for Various Concrete Members

Smallest dimension of the member (cm) ↓	Maximum size of the aggregate (mm)			
Member →	Reinforced walls, beams, and columns	Heavily reinforced slabs	Unreinforced or slightly reinforced members	Unreinforced walls
6-14	16	16	32	16
15-29	32	32	63	32
30-74	63	63	63	63

Step 2: Determination of the Grading of Aggregate:

The grading limits according to the maximum size of aggregate are given in FIGs TS-1, TS-2, TS-3, and TS-4. The grading curves falling in regions 3 and 4 are acceptable. If the working conditions persist, it is possible to use grading curve falling in the region 2, however, it is not permitted to use the aggregate falling in the region 5.

Since the aggregates will be introduced into mixer in two or three size groups, TABLE TS-4 can be used to determine these size groups.

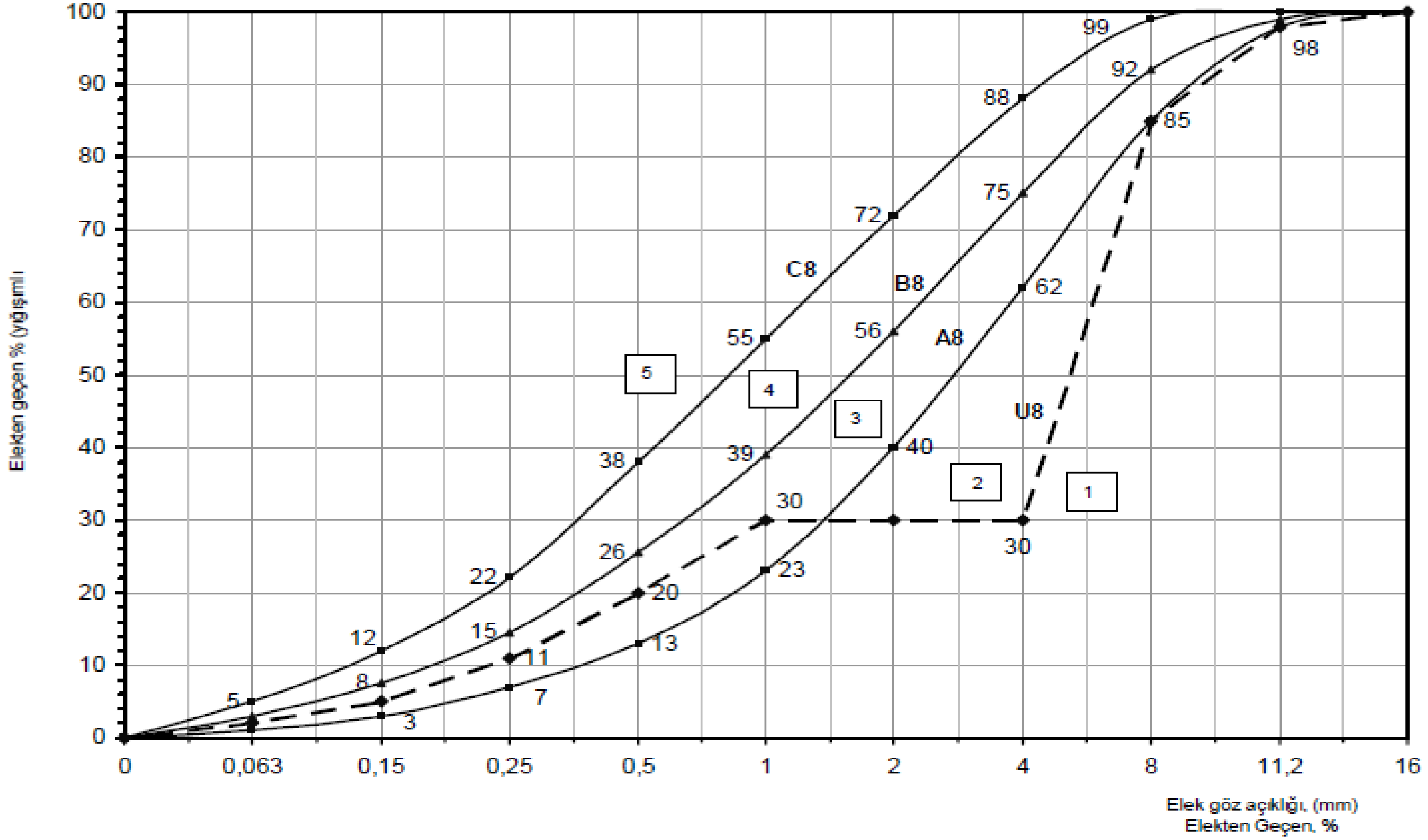


Figure TS-1. Grading limits for concrete having D_{max} = 8 mm

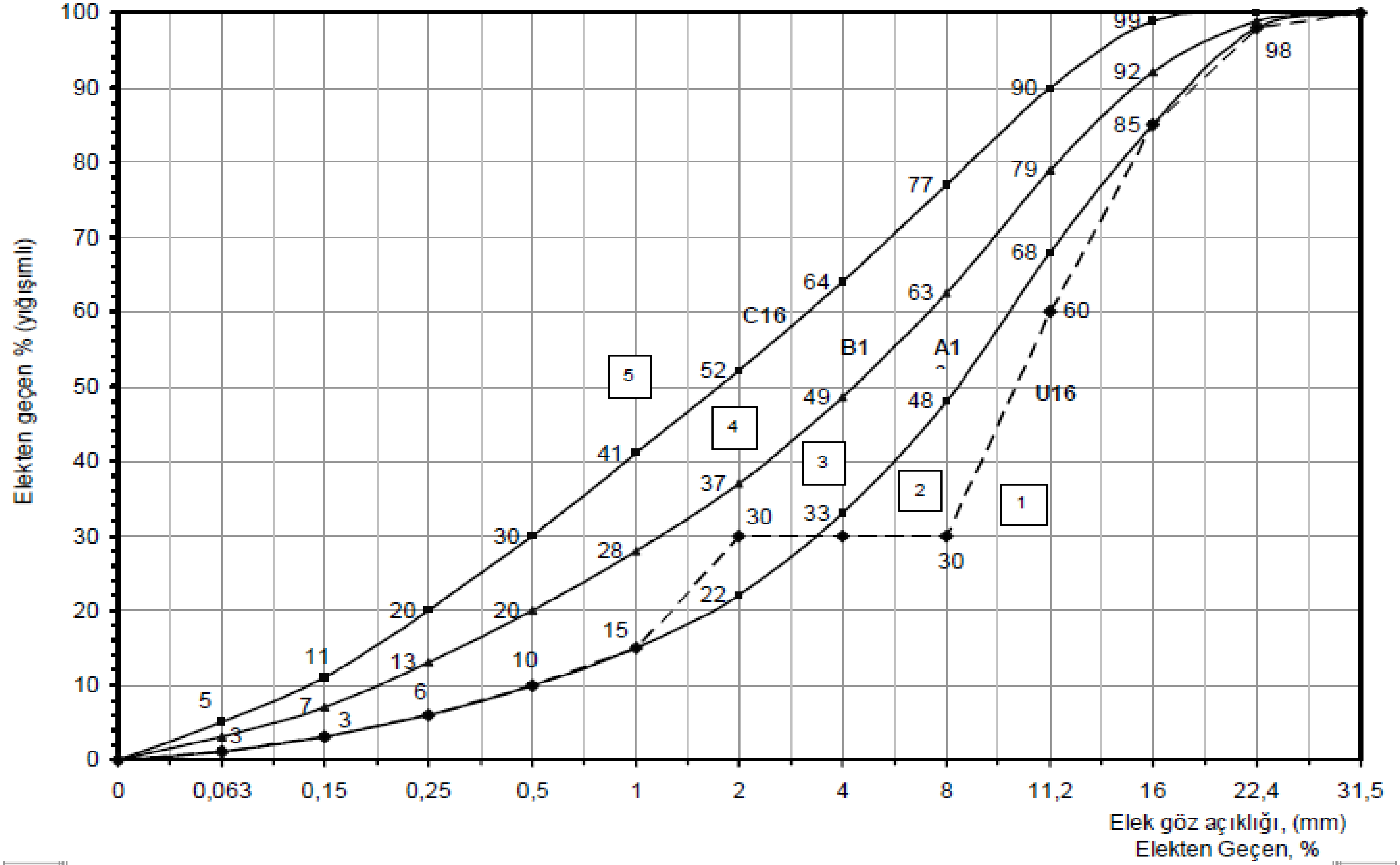


Figure TS-2. Grading limits for concrete having D_{max} = 16 mm

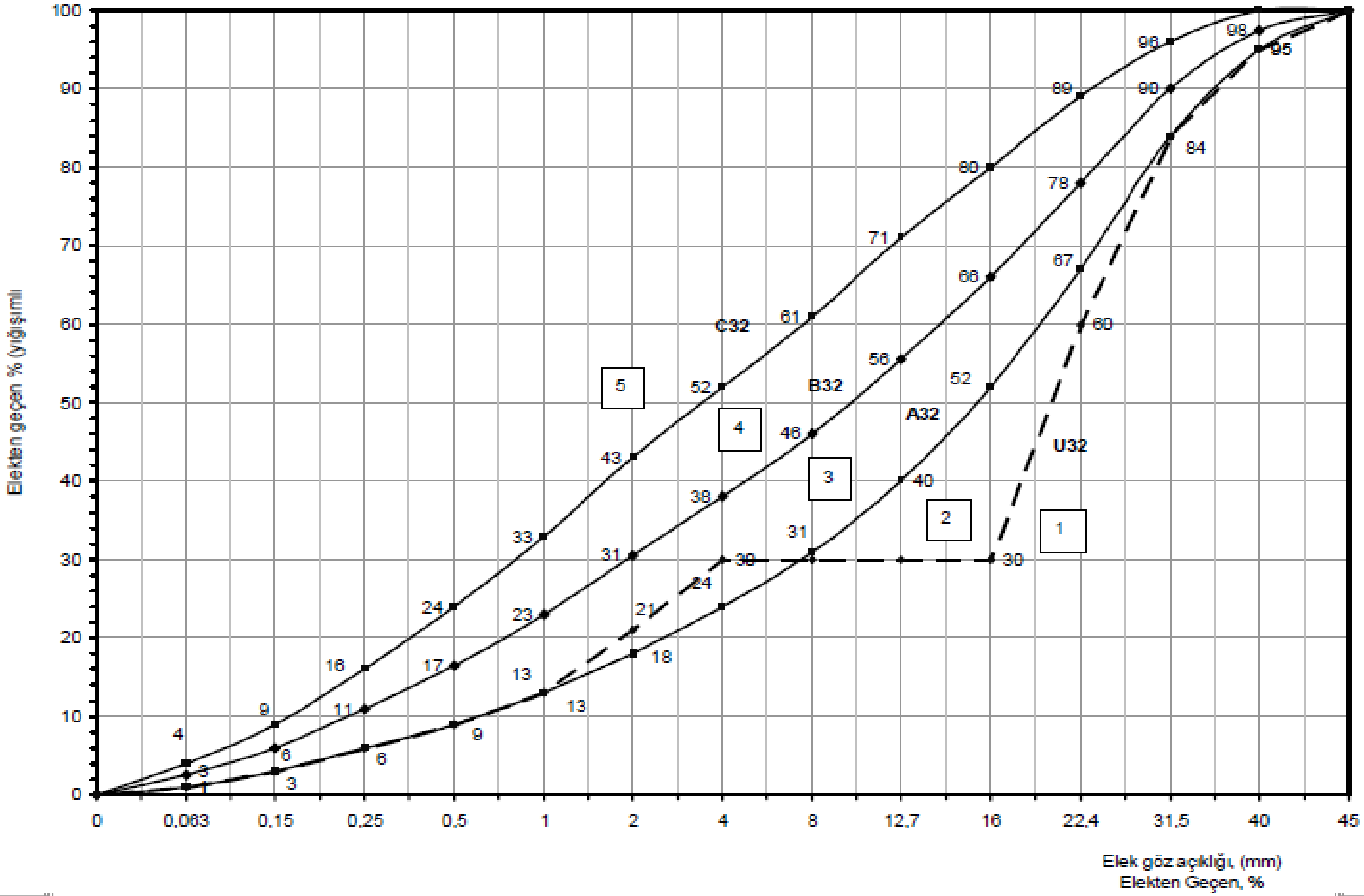


Figure TS-3. Grading limits for concrete having D_{max} = 32 (31.5) mm

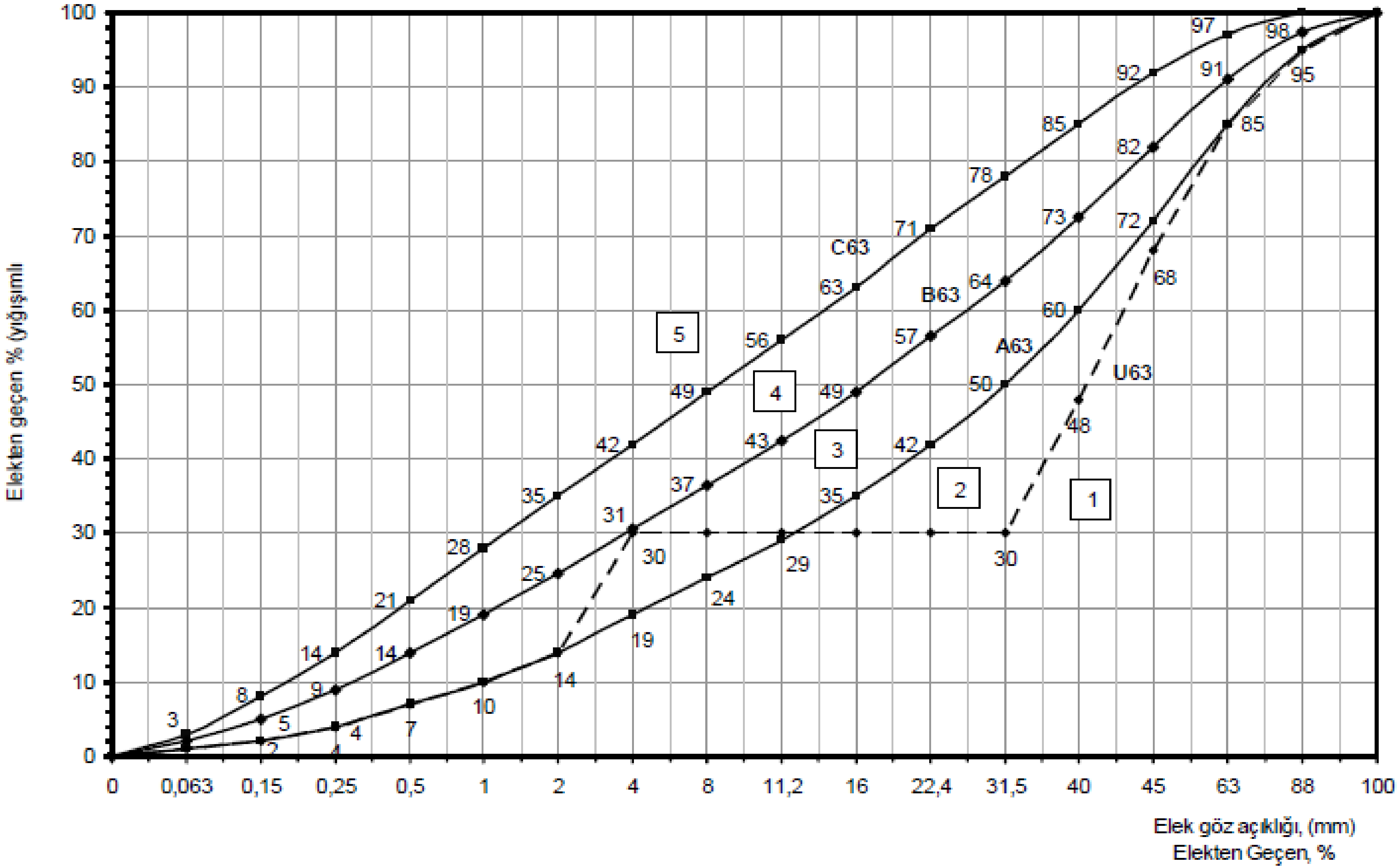


Figure TS-4. Grading limits for concrete having D_{max} = 63 mm

TABLE TS-4. Size Grouping of Concrete Aggregates

Concrete class	Maximum aggregate size of the mixture (D_{max})												
	8			11.2			16			22.4			
	Aggregate size classification												
	1	2	3	1	2	3	1	2	3	1	2	3	4
C16/20 C20/25 C25/30	0/4	4/8	-	0/4	4/11.2	-	0/4	4/16	-	0/4	4/11.2	11.2/ 22.4	-
	0/2	2/4	4/8	0/2	2/4	4/11.2	0/4	4/8	8/16	0/2	2/4	4/ 11.2	11.2/ 22.4
C30/37 C35/45 C40/50 C45/55 C50/60	0/2	2/4	4/8	0/4	4/11.2	-	0/4	4/8	8/16	0/4	4/11.2	11.2/ 22.4	-
				0/2	2/4	4/11.2	0/2	2/4	4/16	0/2	2/4	4/ 11.2	11.2/ 22.4

TABLE TS-4. Size Grouping of Concrete Aggregates

Concrete class	Maximum aggregate size of the mixture (D_{max})													
	32(31.5)				45					63				
	Aggregate size classification													
	1	2	3	4	1	2	3	4	5	1	2	3	4	5
C16/20 C20/25 C25/30	0/4	4/11.2	11.2/32	-	0/4	4/22.4	22.4/45	-	-	0/4	4/16	16/32	32/63	-
	0/4	4/8	8/16	16/32	0/4	4/11.2	11.2/22.4	22.4/45	-	0/4	4/11.2	11.2/22.4	22.4/63	-
C30/37 C35/45 C40/50 C45/55 C50/60	0/4	4/11.2	11.2/32	-	0/4	4/11.2	11.2/22.4	22.4/45	-	0/4	4/16	16/32	32/63	-
	0/2	4/8	4/11.2	11.2/32	0/2	2/4	4/11.2	11.2/22.4	22.4/45	0/4	4/8	8/16	16/32	32/63

The grading limits for concrete to be transported and placed by means of a concrete pump are given for fine aggregate and mixed aggregate in FIGs TS-5 and TS-6, respectively. In addition, a pumpable concrete should include fine aggregate having fineness modulus between 2.30 to 3.10.

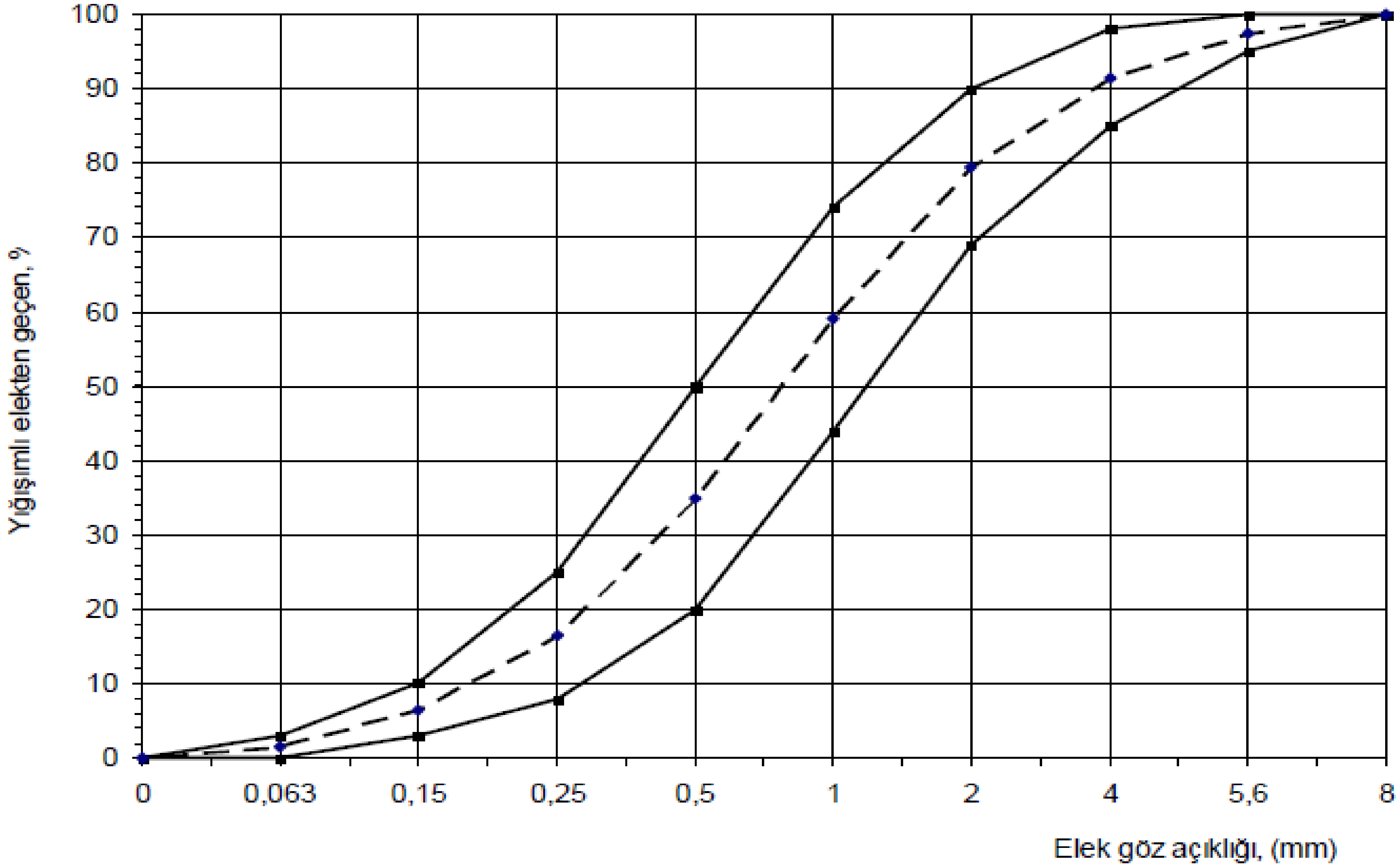


Figure TS-5. Grading limits for fine aggregate recommended for concrete to be placed by means of concrete pump

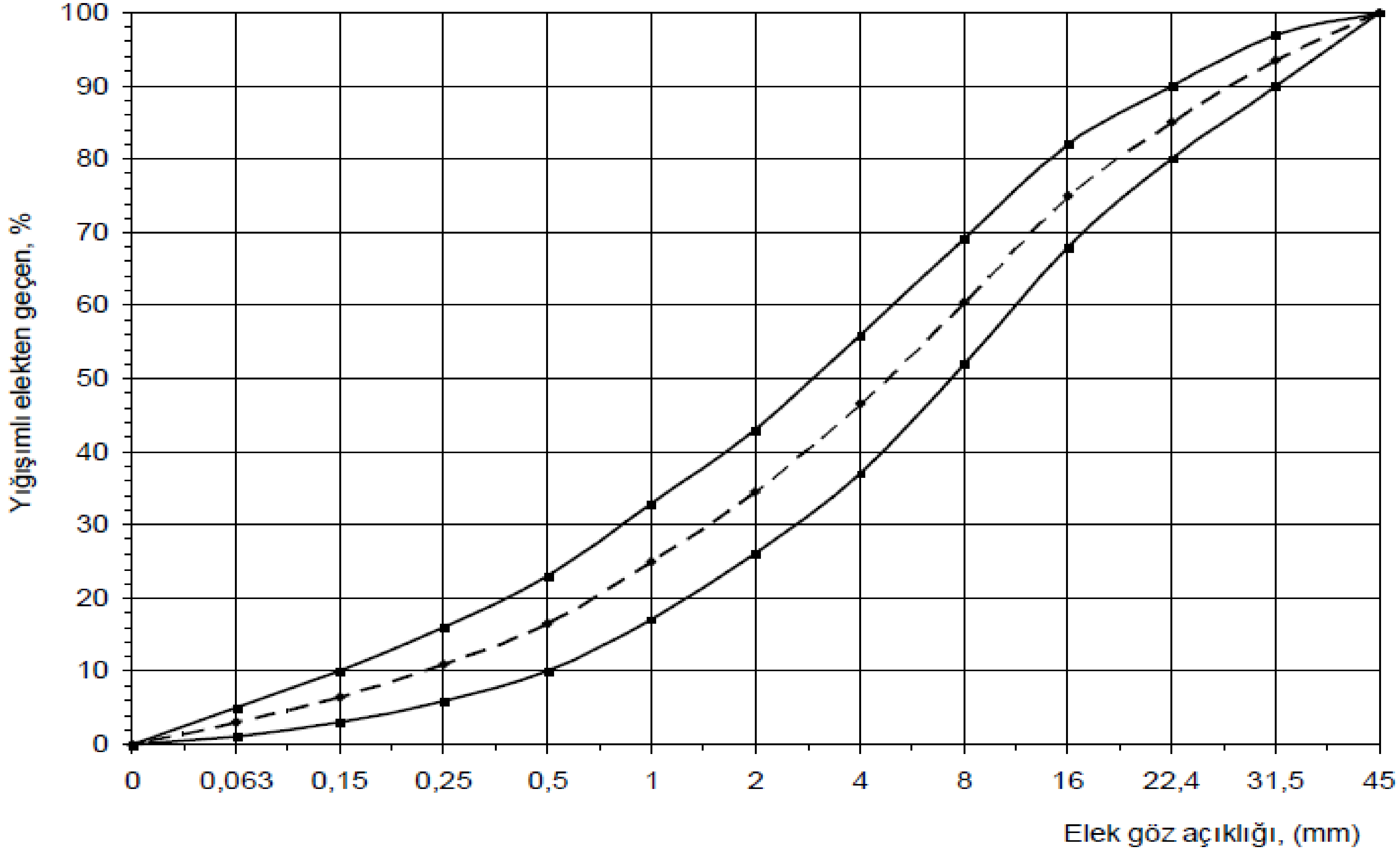


Figure TS-6. Grading limits for mixed aggregate recommended for concrete to be placed by means of concrete pump

Step 3. Choosing W/C Ratio:

For the determination of W/C ratio, the concrete strength class as well as the limits given for different exposure conditions in TS EN 206-1 should be considered.

28-day compressive strength values on which the choice of W/C ratio will be based on are given in Table TS-5.

TABLE TS-5. Design strength (fcm) and the minimum and average strength value requirements for specimens

Concrete Class	Characteristic Strength, fck, (MPa)		Target Strength, fcm (MPa)		
	Cylinder (150x300)	Cube (150x150)	If st. dev. is known	If st. dev. is unknown	
				Cylinder	Cube
C 14/16	**14**	**16**	$f_{cm} = f_{ck} + 1.48\sigma$	**18**	**20**
C 16/20	16	20		20	24
C 18/22	18	22		22	26
C 20/25	**20**	**25**		**26**	**31**
C 25/30	25	30		31	36
C 30/37	30	37		36	43
C 35/45	**35**	**45**		**43**	**53**
C 40/50	40	50		48	58
C 45/55	45	55		53	63
C 50/60	50	60		58	68
C 55/67	55	67		63	75
C 60/75	60	75		68	83
C 70/85	70	85		78	93
C 80/95	80	95		88	103
C 90/105	90	105		98	113
C100/115	100	115		108	123

Table TS-6 gives the W/C ratio values for both air-entrained and non air entrained mixtures, these values are plotted in FIG TS-8.

TABLE TS-6 Relationship Between W/C Ratio and 28-day Cylinder Compressive Strength

28-day Strength* (MPa)	W/C Ratio (by weight)	
	Non-air-entrained concrete	**Air-entrained concrete**
45	0.37	-
40	0.42	-
35	0.47	0.39
30	0.54	0.45
25	0.61	0.52
20	0.69	0.60
15	0.79	0.70

* Assumes 150x300 mm cylinder specimens

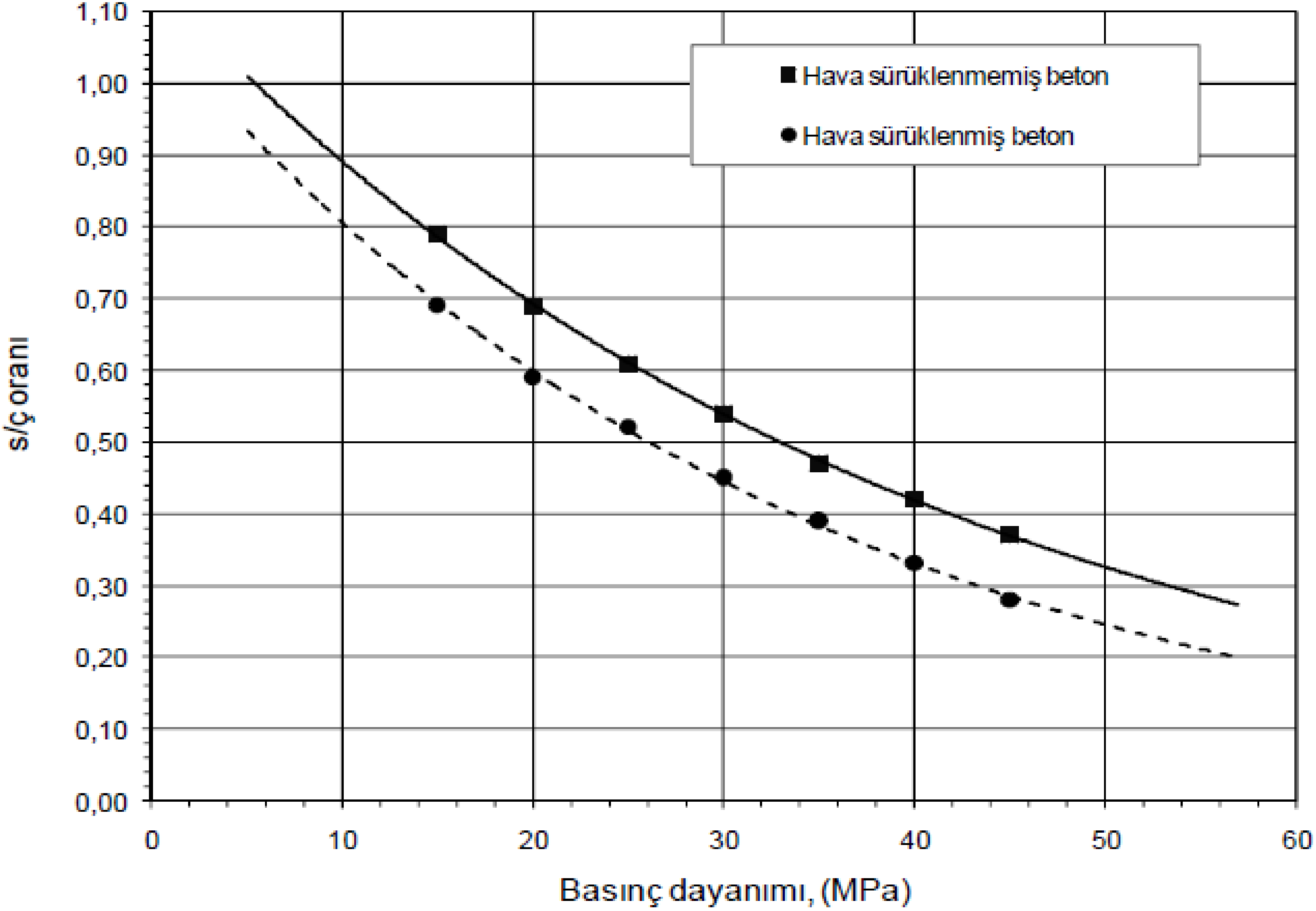


Figure TS-8. W/C ratio strength relationship according to TABLE TS 6.

Step 4. Determining the Water Content:

Water content of the mixture depends on consistency, D_{max} and the presence of chemical admixtures (especially air entraining or water reducing agents). FIGS TS-9 to TS-12 give the approximate values for different slump values and aggregate types. The variations in water content are possible by the use of chemical admixtures.

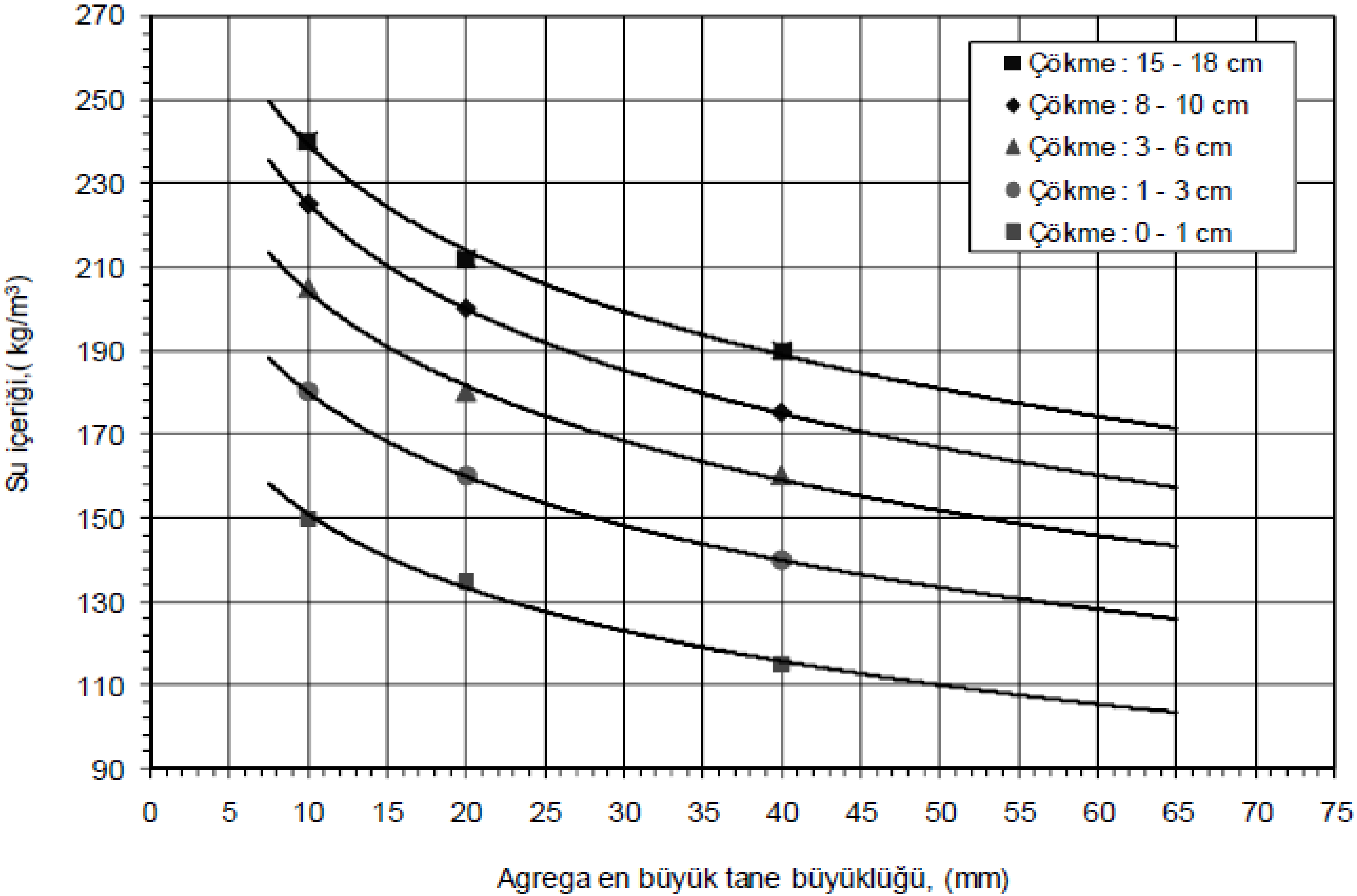


Figure TS-9. Determination of water content for non air entrained concrete designed without chemical admixture and with natural graded aggregates

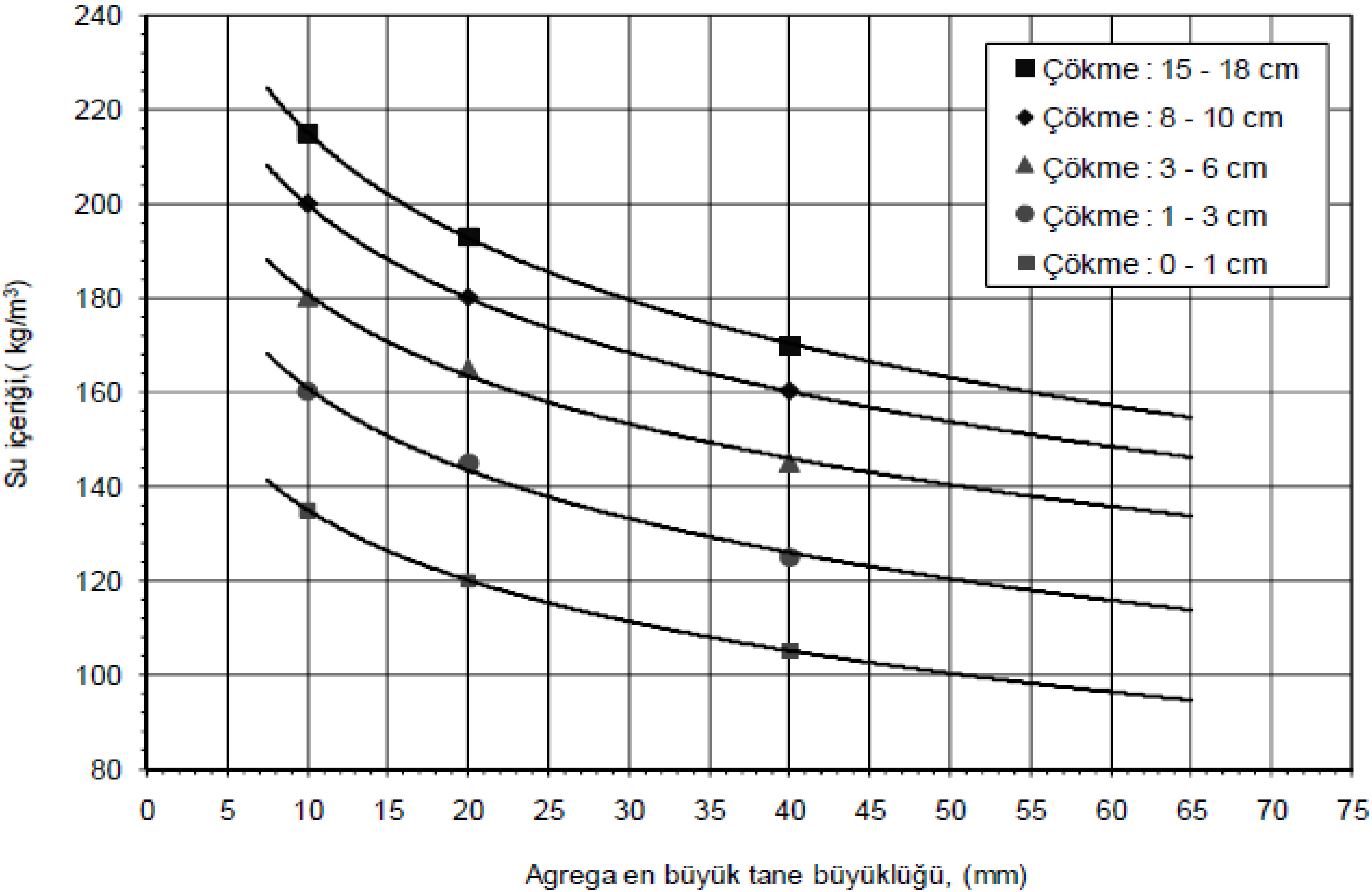


Figure TS-10. Determination of water content for air entrained concrete designed without chemical admixture and with natural graded aggregates

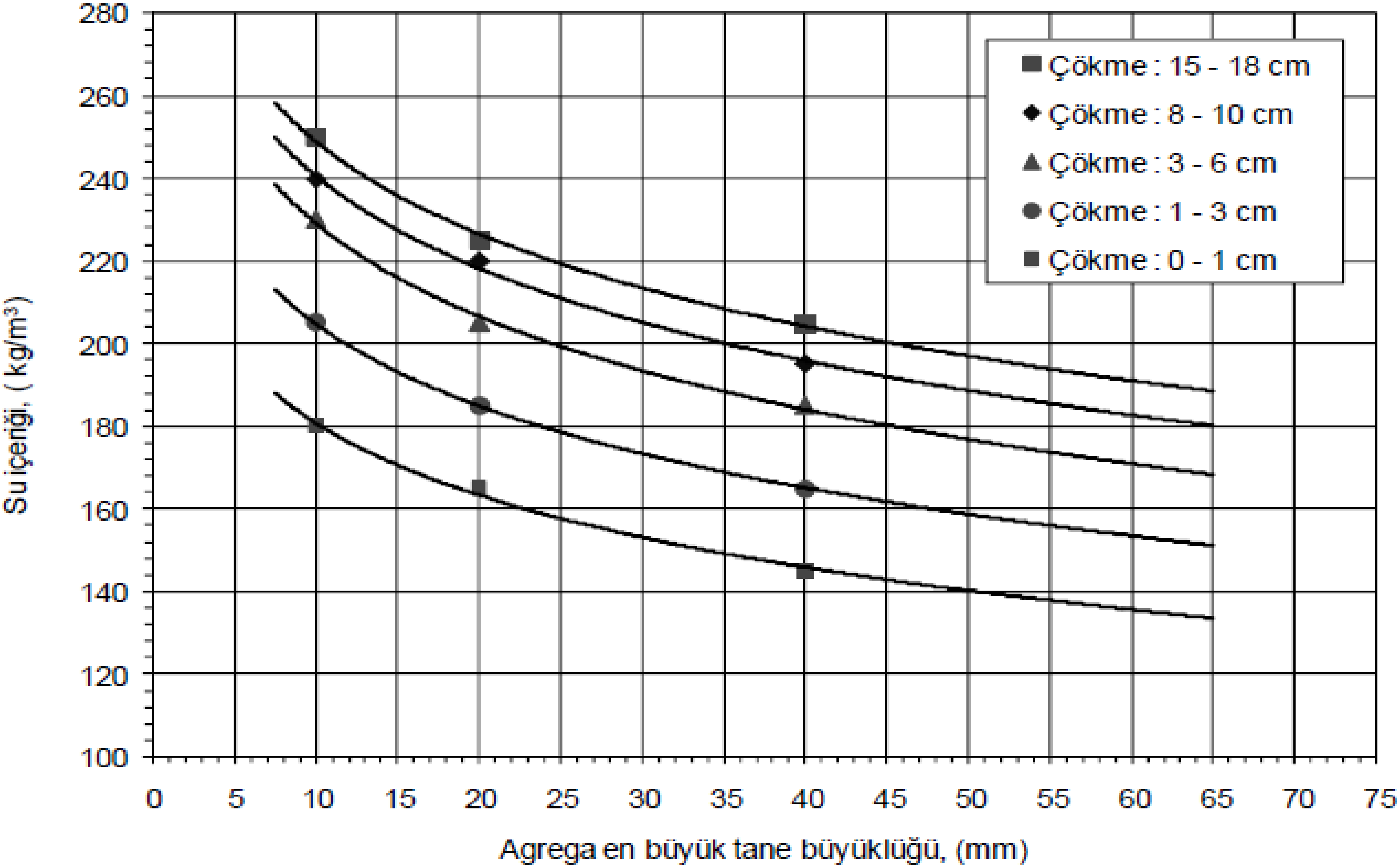


Figure TS-11. Determination of water content for non air entrained concrete designed without chemical admixture and with crushed aggregates

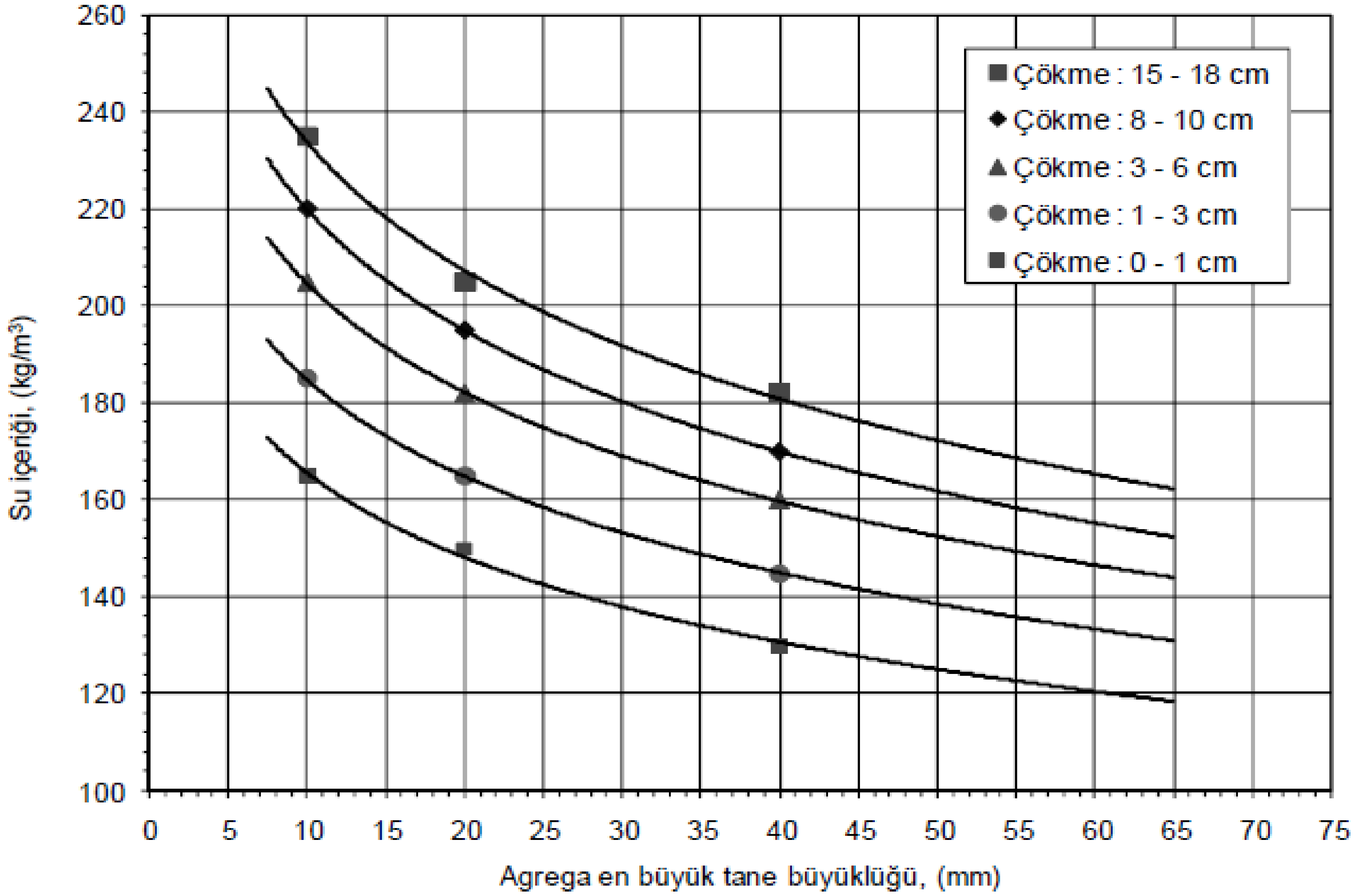


Figure TS-12. Determination of water content for air entrained concrete designed without chemical admixture and with crushed aggregates

Step 4. Determining the Air Content:

The air content of concrete should be determined according to the weather conditions and D_{max}

(see FIG TS 13).

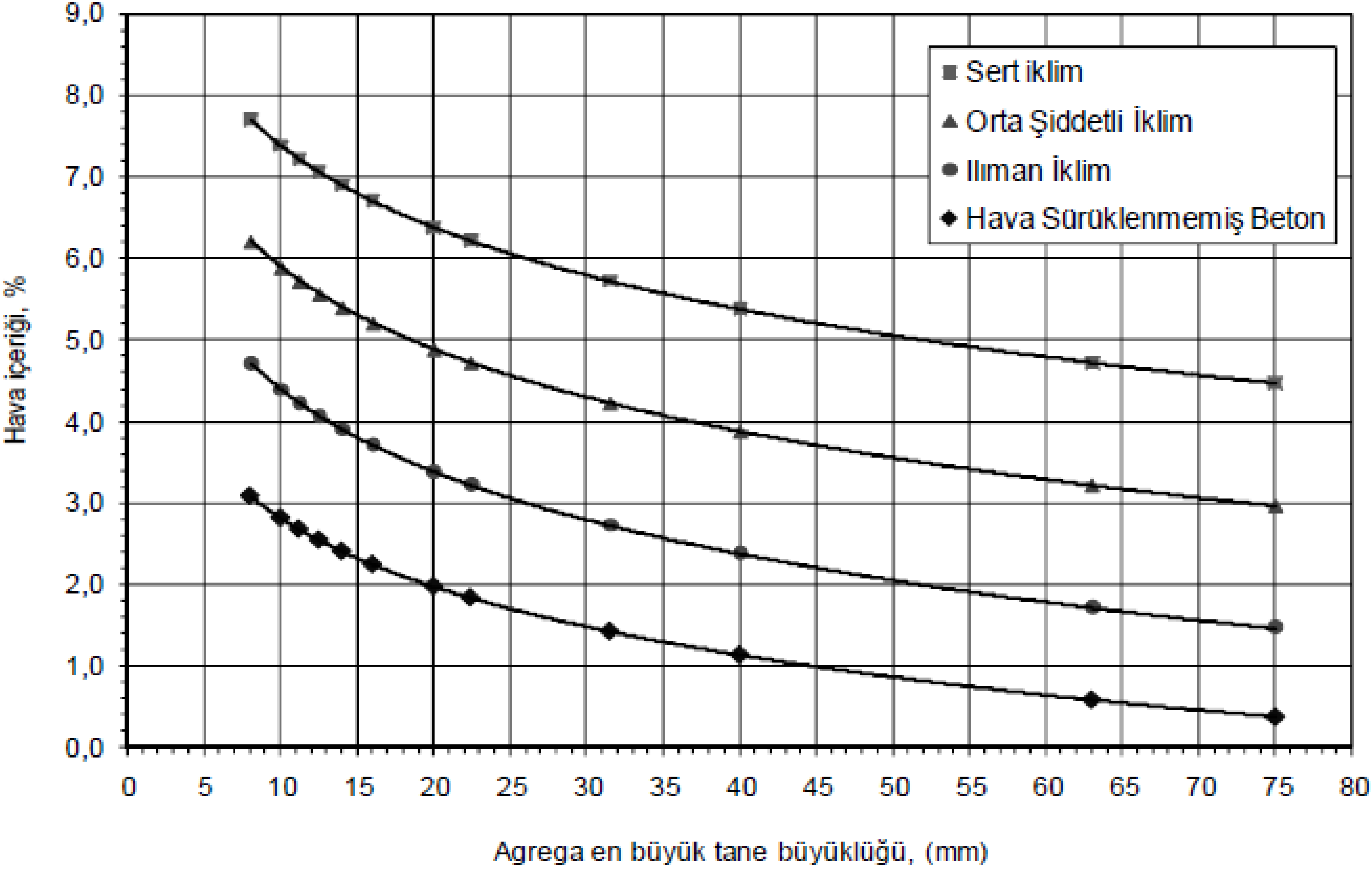


Figure TS-13. Determination of air content

Step 5. Determining the Slump:

The slump values should conform to TS EN 206-1 standard. It is possible to achieve slump values with less water by the use of chemical admixtures.

Recommended slump values for various structure types are given in TABLE TS-7.

TABLE TS-7. Recommended Slump Values for Various Types of Constructions

Type of Construction	Slump Value	
	Maximum	Minimum
Reinforced foundations	80	30
Plain footings, underwater foundations, caissons, substructure walls, channel linings	80	30
Beams, slabs, reinforced walls, columns	100	50
Tunnel walls	50	20
Pavements, bridge piers, tunnel floors	50	20

Calculation Method (TS EN 802)

- The amount of concrete ingredients for 1m^3 compacted concrete can be calculated by;

$(c/\rho_c) + (p/\rho_p) + (k/\rho_k) + w + (W_a/\rho_a) + 10xA = 1000dm^3$

c: is the amount of cement in the mixture (kg)

p: is the amount of mineral admixture (pozzolan) in the mixture (kg)

k: is the amount of chemical admixture in the mixture (kg)

ρ_c: cement density (kg/dm^3)

ρ_p: pozzolan density (kg/dm^3)

ρ_k: chemical admixture density (kg/dm^3)

w: volume of water in the mixture (dm^3)

W_a: amount of aggregate in the mixture (kg)

ρ_a: average specific mass of the aggregate (kg/dm^3)

A: total air content of concrete (%)

Note that if chemical admixture is used it should be assumed that at least half of the admixture is water and this amount should be subtracted from the w value.

Determination of cement content

After w/c ratio is determined, the amount of cement to be added to the mixture becomes;

$c = w/ (w/c)$

The density of cement –if not measured- can be taken from Table TS-9.

TABLE TS-9 Recommended Density Values for Various Types of Cements

Cement Type	Specific mass
CEM I 42.5N and CEM I 42.5R	3.10
CEM II/A 42.5R, CEM II/B 42.5R	3.00
CEM III/A, CEM III/B	2.95
CEM IV, CEM V	2.85

Determination of water and air content: Use the related tables and graphics.

Determination of aggregate content:

$$V_a = (W_a / \rho_a) = 1000 - ((c/\rho_c) + (p/\rho_p) + (k/\rho_k) + w + A)$$

$$\bar{\rho}_a = \frac{1}{\frac{x_1}{\rho_{a1}} + \frac{x_2}{\rho_{a2}} + \frac{x_3}{\rho_{a3}} + \ldots\ldots + \frac{x_n}{\rho_{an}}}$$

Here the ρ_a value is the average specific mass of the aggregates, then it is possible to calculate the total amount of aggregate in the mixture; and finally, the amounts of each size fraction can be determined by multiplying their percentages with the total amount of aggregate.

$$W_a = V_a \times \rho_a$$

- If the density of the aggregates have not been determined by tests, it is possible to use the values given in Table TS-10.

TABLE TS-10 Recommended Specific Gravity Values for Various Types of Aggregates

<table>
<tr><th colspan="2">Fine Aggregate (0/4)</th></tr>
<tr><td><i>Rock type</i></td><td><i>Specific gravity</i></td></tr>
<tr><td>Quartz sand</td><td>2.64</td></tr>
<tr><td>Dense limestone sand</td><td>2.70</td></tr>
<tr><th colspan="2">Coarse Aggregate (>4)</th></tr>
<tr><td><i>Rock type</i></td><td><i>Specific gravity</i></td></tr>
<tr><td>Granite</td><td>2.62</td></tr>
<tr><td>Gneiss</td><td>2.67</td></tr>
<tr><td>Limestone</td><td>2.70</td></tr>
<tr><td>Diabase</td><td>2.85</td></tr>
<tr><td>Diorite</td><td>2.90</td></tr>
</table>

Trial mixture: The slump, unit weight, air content and yield of fresh concrete should be determined and the real amounts of ingredients in the concrete mixture should be calculated. For every $1 m^3$ of concrete an alteration of 2.5 kg of water corresponds to 10 mm of slump change, and for every 1% of alteration in the air content, the mix water can be changed by 3 kg/m^3. Any change in the design values also changes the yield, thus the values should be re-corrected after any change in the design values.

EXAMPLE:

Concrete will be designed for a reinforced concrete column having minimum mold thickness of 25 cm and concrete cover of 35 mm. The concrete will be subjected to freezing-thawing –air entrained concrete- and the concrete will have a strength class of C25/30.

Materials

Cement: CEM I 42.5R (ρ_c: 3.15 kg/dm^3)

Aggregates:

$AC_{CA1(16/32mm)} = 0.5\%$ $AC_{CA2(4/16\ mm)} = 0.8\%$ $AC_{FA}=1.5\%$

$TM_{CA1(16/32mm)} = 1\%$ $TM_{CA2(4/16\ mm)} = 1.2\%$ $TM_{FA}=3.5\%$

$\rho_{CA1(16/32mm)SSD} = \rho_{CA2(4/16mm)SSD} = 2.80$ kg/dm^3 $\rho_{FA\text{-}SSD} = 2.65$ kg/dm^3

$D_{max} = 31.5$ mm

Aggregate gradation is given in Figure 14, and the amounts for each aggregate was chosen as; CA1: 28%, CA2: 25%, FA: 47%.

Fineness modulus of fine aggregate is determined as 2.82.

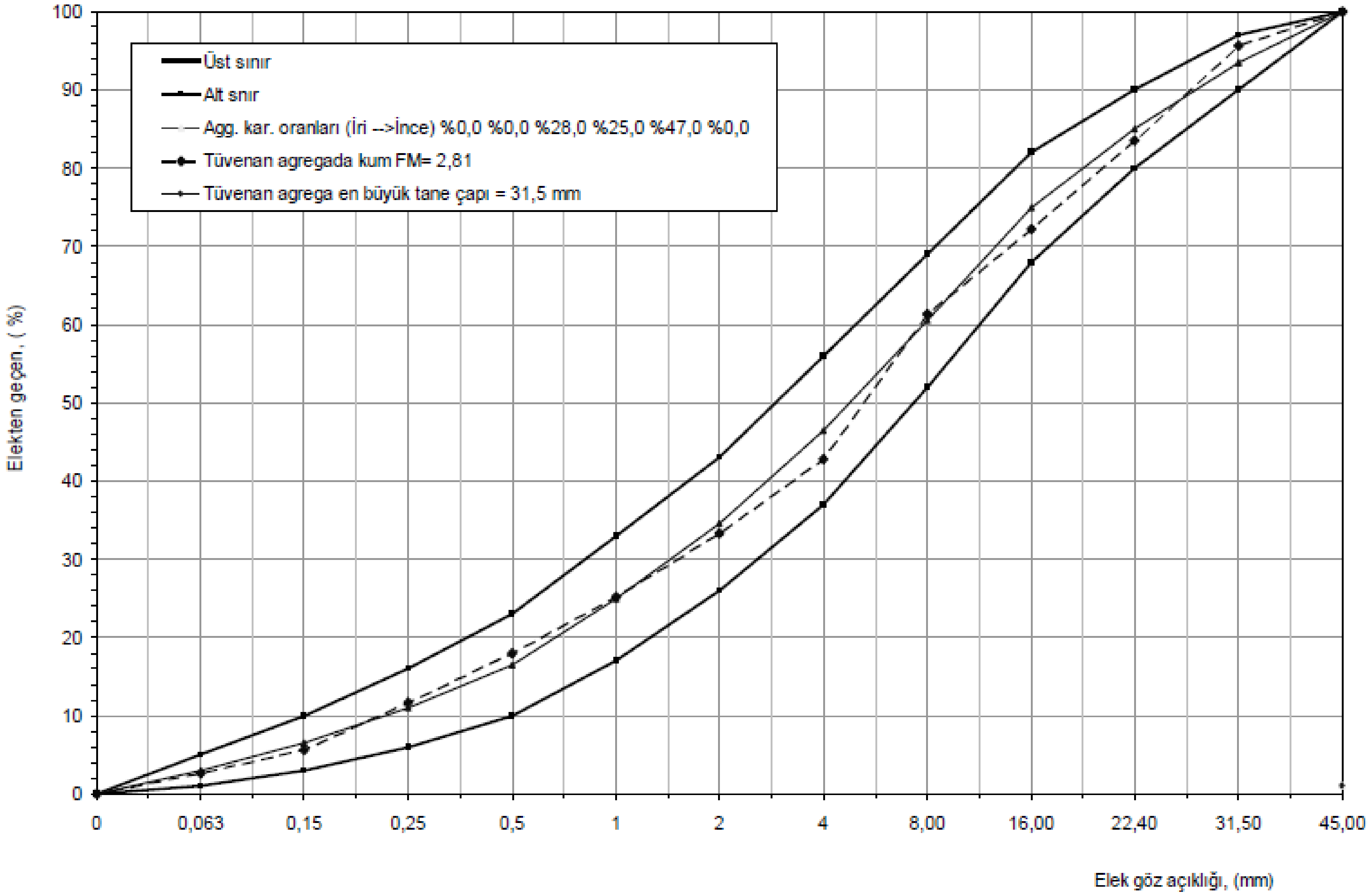


Figure 14: Gradation of the aggregates conform the limits given in Figure TS-6.

SOLUTION:

The structure (column) will be exposed to frequent freeze-thaw cycles, thus, air entrainment is going to be applied. According to TS EN 206-1, the concrete exposing to such conditions should have maximum w/c ratio of 0.50, concrete class should be at least C30/37, minimum cement content should be 320 kg/m^3 and air content at least 4%.

From strength p.o.w the required strength level is C25/30 concrete; however, durability conditions necessitate a higher strength level, C30/37.

TABLE TS-5. Design strength (fcm) and the minimum and average strength value requirements for specimens

Concrete Class	Characteristic Strength, fck, (MPa)		Target Strength, fcm (MPa)		
	Cylinder (150x300)	Cube (150x150)	If st. dev. is known	If st. dev. is unknown	
				Cylinder	Cube
C 14/16	14	16	$f_{cm} = f_{ck} + 1.48\sigma$	18	20
C 16/20	16	20		20	24
C 18/22	18	22		22	26
C 20/25	20	25		26	31
C 25/30	25	30		31	36
C 30/37	30	37		36	43
C 35/45	35	45		43	53
C 40/50	40	50		48	58
C 45/55	45	55		53	63
C 50/60	50	60		58	68
C 55/67	55	67		63	75
C 60/75	60	75		68	83
C 70/85	70	85		78	93
C 80/95	80	95		88	103
C 90/105	90	105		98	113
C100/115	100	115		108	123

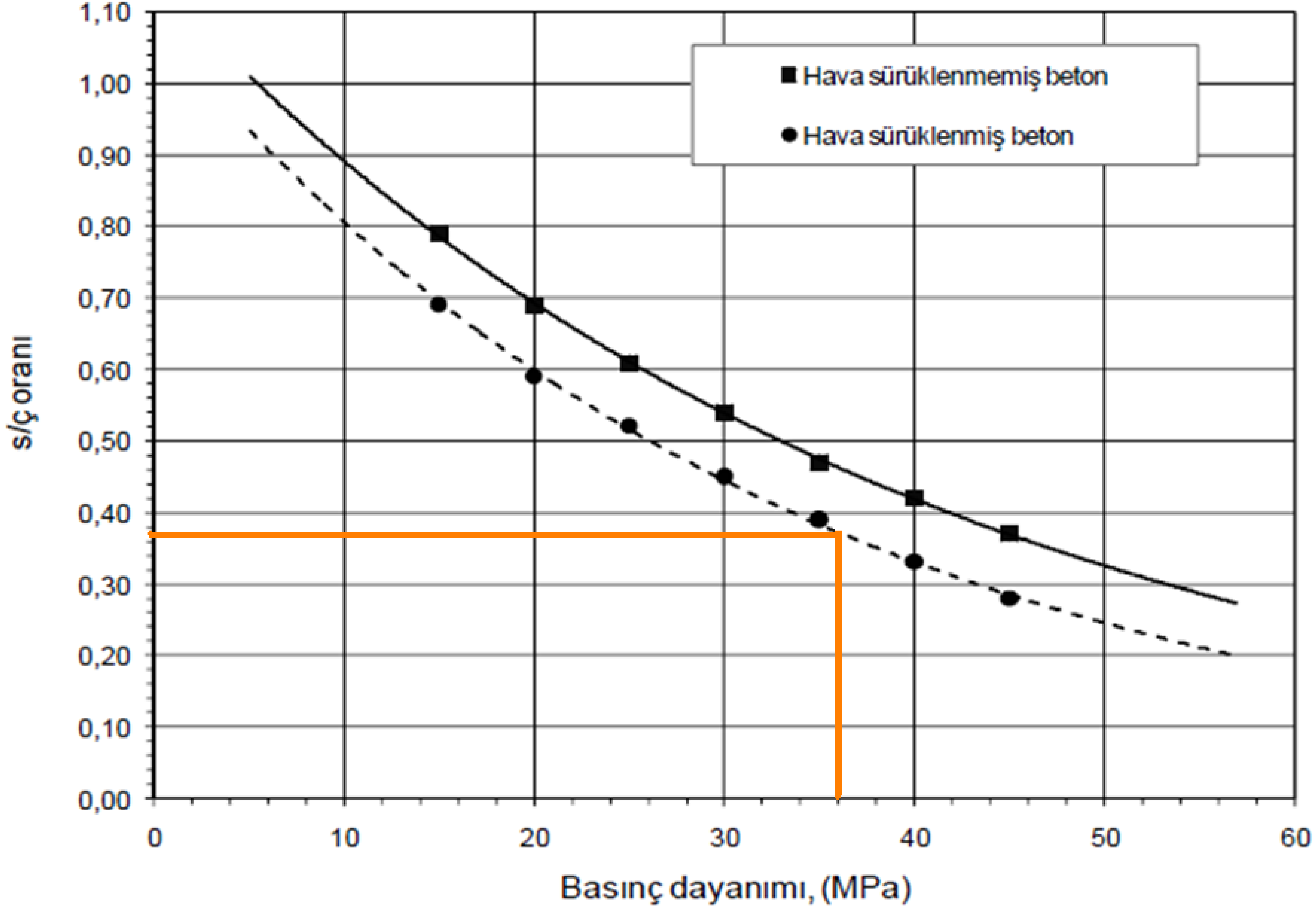


Figure TS-8. W/C ratio strength relationship according to TABLE TS 6.

For this strength level (C30/37) 28 day old characteristic cylinder strength (f_{ck}) is 36 MPa (Table TS 5), and the corresponding w/c ratio can be read from the plot as 0.38. This value can be accepted as it is less than 0.50 limit (TS EN 206-1).

Slump value

S3 is chosen for workability and pumpability. Previous experience shows that to obtain this slump value with optimum water content, 1.5% by weight of cement a HRWRA is necessary. Specific gravity of HRWRA is 1.15.

Class	**Slump , mm**
S 1	10 – 40
S 2	50 – 90
S 3	100 – 150
S 4	160 - 210
S 5	≥ 220

Water content

- Depending on the slump value and D_{max}, as first approach the water content is found as 190 kg/m^3 (Figure TS 12). However, an addition of HRWRA may lead to about 15% reduction in water, i.e. 160 kg/m^3.

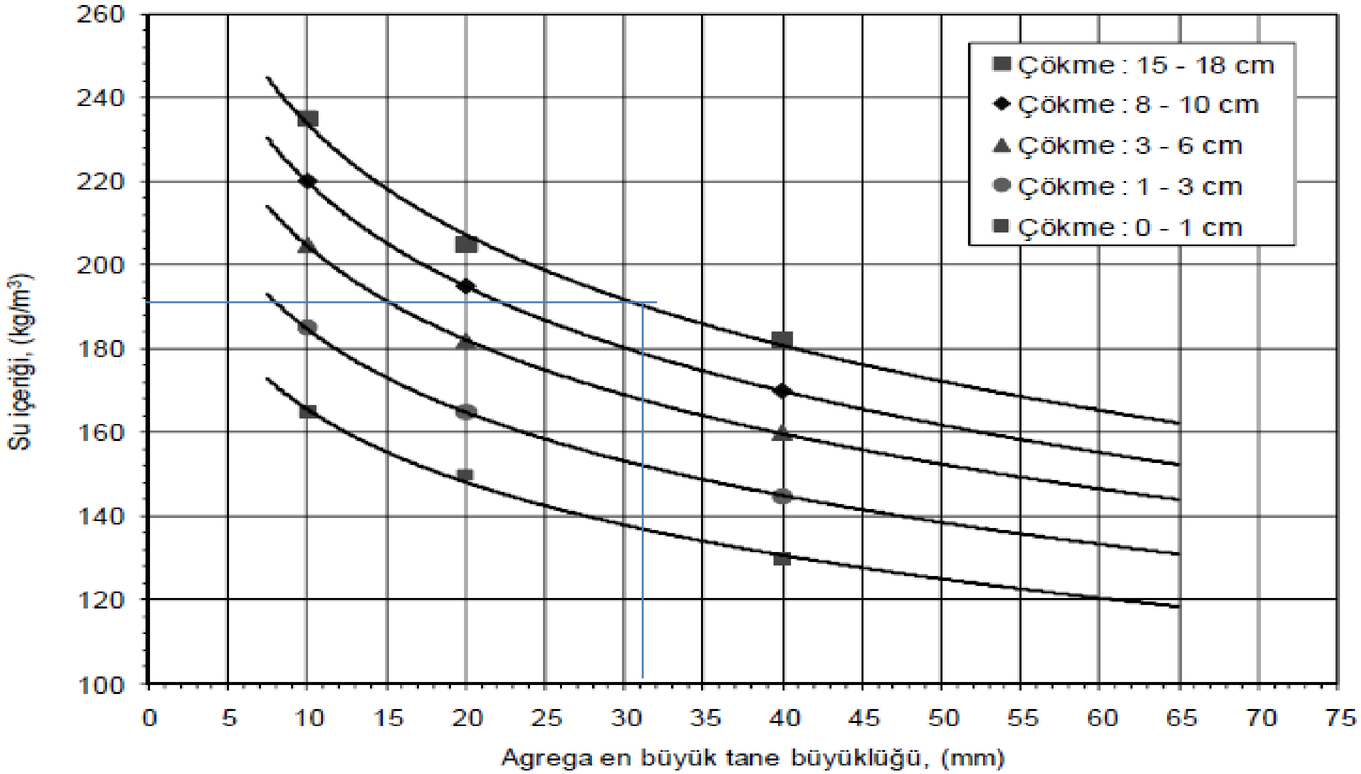


Figure TS-12. Determination of water content for air entrained concrete designed without chemical admixture and with crushed aggregates

Air Content

Air content is read from Figure TS 13 as apprximately 5%. The amount of air-entraining admixtures (AEE) was determined as 0.05% by weight of cement from the past experience.

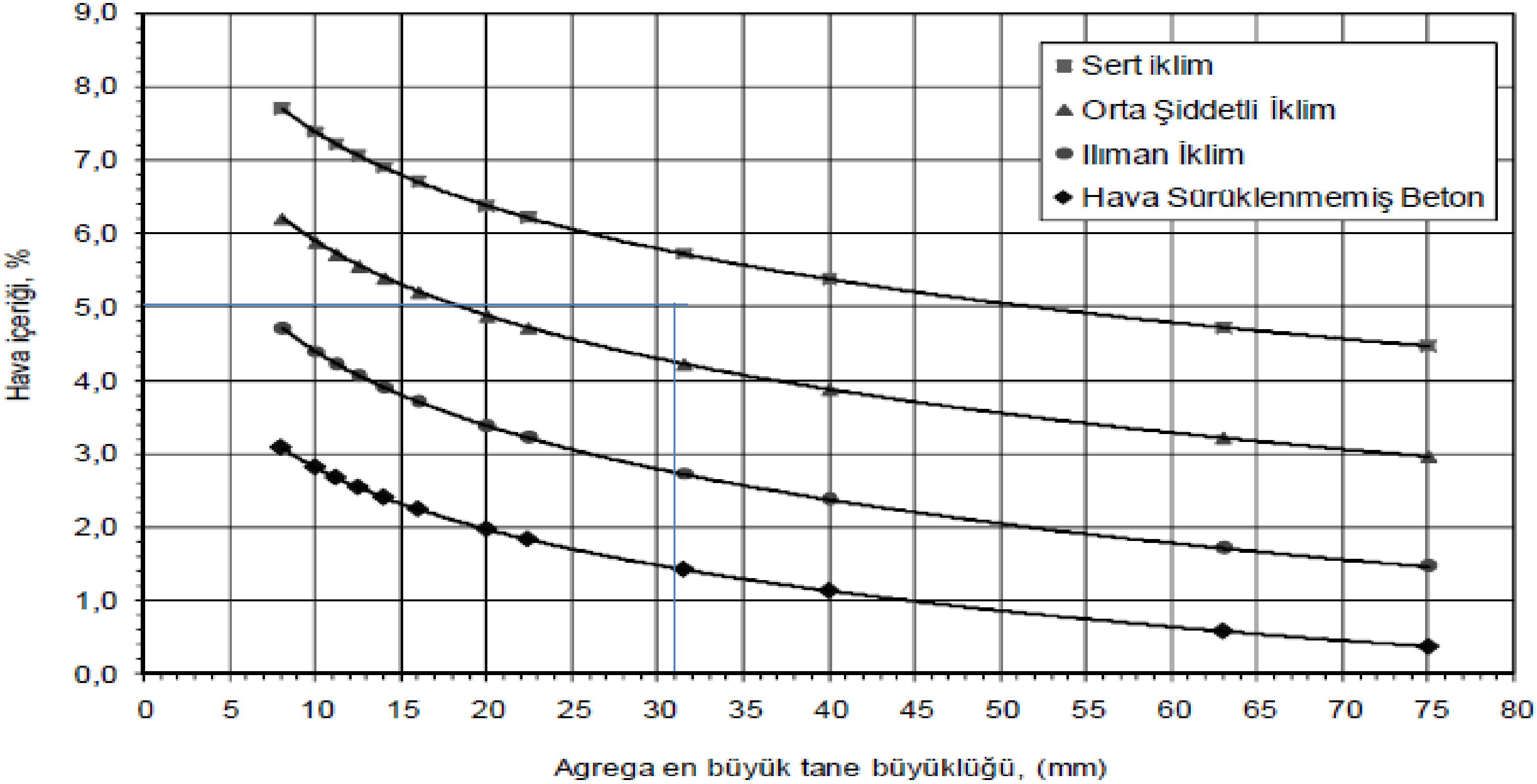


Figure TS-13. Determination of air content

Cement Content

W=160 kg/m^3 and w/c = 0.38,

Thus c= w/(w/c) = 160/0.38=420 kg/m^3 (this value is higher than the minimum cement content required in TS EN 206-1).

Cement Volume

In 1m^3 concrete V_c = 420/3.15 = 133.3 dm^3

Chemical Admixture Content

The cement dosage is 420 kg/m^3 and the amount of chemical admixture in 1m^3 concrete;

HRWRA: 420 x 1.5 /100 = 6.3 kg

AEA: 420x0.05/100 = 0.21 kg

and their volume is;

V_{HRWRA}: 6.3/1.15 = 5.48 dm^3

V_{AEA}: 0.21/1.01 = 0.20 dm^3

Aggregate Content

$V_a = W_a/\rho_a = 1000 – (c/\rho_c + w+A+V_{HRWRA} + V_{AEA})$

$W_a/\rho_a =$

$1000 – (133.3+160+5x10+5.48+0.20) =651.1dm^3$

Average specific mass of aggregate, $\rho_{a},_{SSD}$

$\rho_{a\,=}$ 1/(0.28/2.8+0.25/2.8+0.47/2.65)=2.727 kg/dm^3

Total amount of aggregate (W_a);

$W_a = V_a \times \rho_a$ = 651.1x2.727 = 1775.5 kg

SSD FA (0/4) = 1775.5 x 0.47 = 834.5 kg

SSD CA1 (16/32)= 1775.5 x 0.28 = 497.1 kg

SSd CA2 (4/16)= 1775.5 x 0.25 = 443.9 kg

Water: 160 kg/m^3

Cement: 420 kg/m^3

High-range water-reducer: 6.3 kg/m^3

Air-entraining admixture: 0.21 kg/m^3

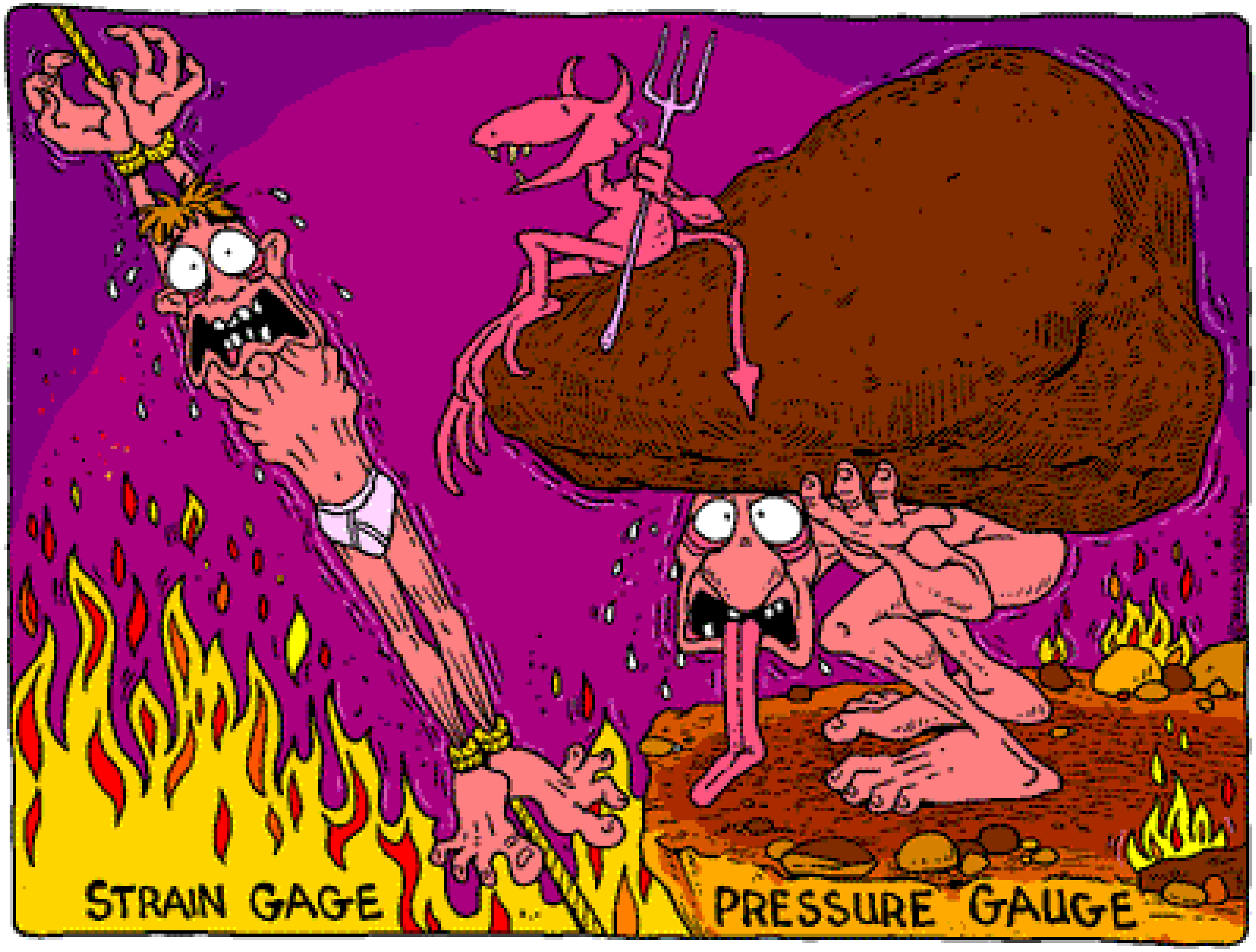
0/4 SSD: 834.5 kg/m^3

4/16 SSD: 443.9 kg/m^3

16/32 SSD: 497.1 kg/m^3

TOTAL: 2362 kg/m^3

HW: Considering half of the HRWRA as water, make the necessary moisture conditions for this concrete mixture.


STRAIN GAGE
PRESSURE GAUGE

6. STRENGTH OF CONCRETE

The strength of concrete appears to be a good index, whether direct or inverse, of most of the other properties of practical significance.

Stronger concretes are:

- stiffer,
- more water-tight,
- more resistant to weathering and destructive agencies.

On the other hand, they usually exhibit higher drying shrinkage and lower extensibility, hence are more liable to cracking.

Strength tests are;

- relatively simple to make,
- form the basis for the common use of strength in specifying and controlling quality,
- useful for evaluating the effects of variable factors such as materials, proportions, manufacturing equipments, methods, and curing conditions.

Nature of Strength-1

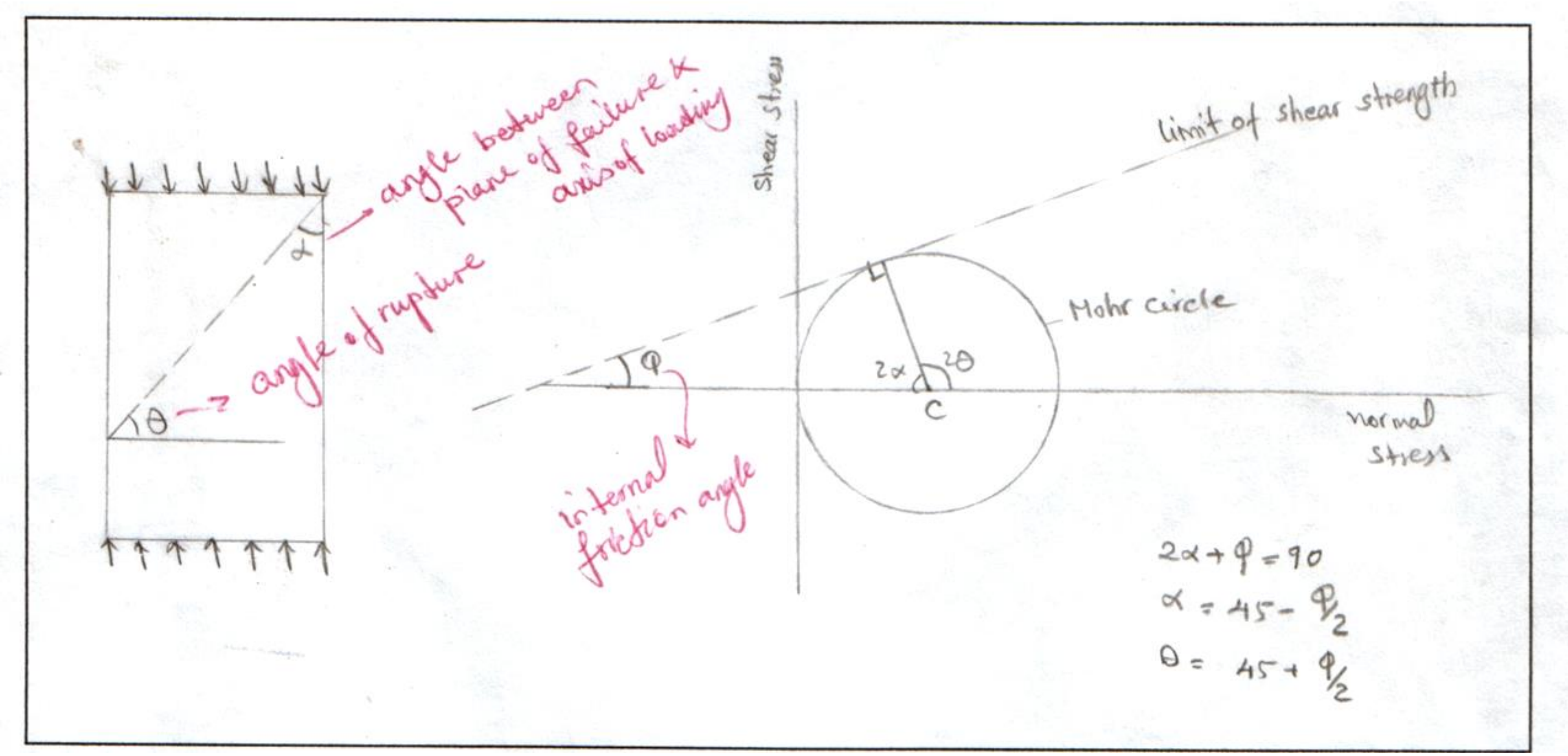
Definition of strength with regard to concrete is unit force (stress) required to cause rupture.

Rupture may be caused by applied tensile stress, by applied shear stress, or by compressive stress.

A brittle material such as concrete is much weaker in tension and in shear than in compression, and failures of concrete specimens under compressive load are essentially shear failures on oblique planes.

Nature of Strength-1

Since the resistance to failure is due to both cohesion and internal friction, the angle of rupture is not 45° but is a function of the internal friction angle ϕ; the angle α which the plane of failure makes with the axis of loading is equal to $45 - \phi/2$. The internal friction angle for concrete is approximately 20°.



FACTORS AFFECTING STRENGTH

Strength of concrete has three components:

(1) Strength of the cement paste, → porosity
(2) Strength of the aggregate, → generally no problem
(3) Bond between the paste and the aggregate. → praticle shape and D_{max}

Strength of Paste as Related to Concrete Strength-1

Powers' **gel/space ratio law** states that the strength of Portland Cement mortars is directly proportional to the increase in gel/space ratio regardless of age, original W/C ratio, or type of cement. The gel/space ratio, X, is the ratio of hydration products volume to the space available for these products.

$$X = \frac{2.1\alpha C V_c}{\alpha C V_c + V_o + A_o}$$

X: gel space ratio

C :weight of cement

Vc *:* specific volume of cement $\boxed{\left(\frac{1}{specific gravity}\right)}$

α *:* fraction of cement that has hydrated

Vo: volume of mixing water

Ao : volume of air

Strength of Paste as Related to Concrete Strength-2

It has been found that the relationship between compressive strength (σ_c) and the gel/space ratio can be written as

$\sigma_c = AX^n$ where, A is a constant representing the strength of the gel at X =1.0

n is a constant having values in the range of 2.6 to 3.0 depending on the characteristics of cement.

This relationship can be written as

$\sigma_c = 235 X^3$ MPa

for portland cement concretes.

For a given cement, strength of the paste depends on

(1)cement content (C),

(2)water content (Wo),

(3)age (α),

(4)air content (Ao).

Water-Cement Ratio

Even though the strength of concrete is dependent largely on the capillary porosity or gel/space ratio, the quantities are not easy to measure or predict. Therefore, they are not suitable for practical purposes.

Fortunately, capillary porosity of a properly compacted concrete at any degree of hydration is determined by the W/C ratio.

Therefore, in practice, the strength of a properly compacted concrete can be assured by specifying the W/C ratio.

Effect of the Water/Cement Ratio on Strength

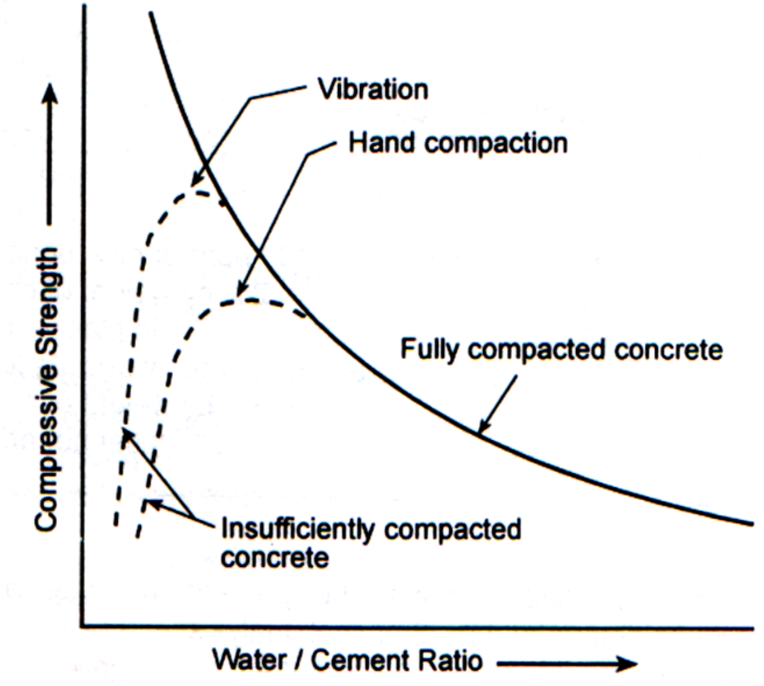
The strength of a properly cured concrete at a given age depends mainly on:

- W/C ratio

- degree of compaction

According to Abrams:

$$\sigma = \frac{K_1}{K_2^{W/C}}$$



σ = Strength of the concrete

w/c = Water/cement ratio

K_1, K_2 = Empirical constants

The W/C ratio affects the capillary porosity of the cement paste. The higher the W/C ratio of the paste, the higher the capillary porosity and the lower the strength.

Time-1

Gain of strength with time is basically related with the increase in gel/space ratio due to the increase of α value. However the rate of strength gain is also dependent on:

(1) characteristics of the cement,

(2) curing conditions,

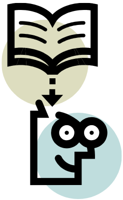
(3) W/C ratio of the mix.

Coarser cement particles (~25μm) → late strength (>28 days)

Finer cement particles (~5μm) → early strength (<7 days)

Low-W/C ratio mixes gain strength more rapidly than high-W/C ratio mixes.

Time-2



In practice, it is common to obtain 7-day as well as 28-day compressive strength tests. Thus, it becomes possible to extrapolate the 28-day strength from 7-day (or other) strengths.

These values are obtained for w/c= 0.52 concretes cast into 100 mm cubic molds and cured at 20°C:

Age of Concrete	Age Factor	
	Range	Average
3 days	0.39-0.63	0.51
7 days	0.59-0.86	0.72
14 days	0.77-1.00	0.88
28 days (1 month)	1.00	1.00
2 months	1.01-1.25	1.11
4 months	1.03-1.34	1.17
8 months	1.04-1.41	1.23
1 year	1.11-1.52	1.28
2 year	1.12-1.49	1.31
5 year	1.12-1.76	1.39

W/C Ratio vs. Compressive Strength

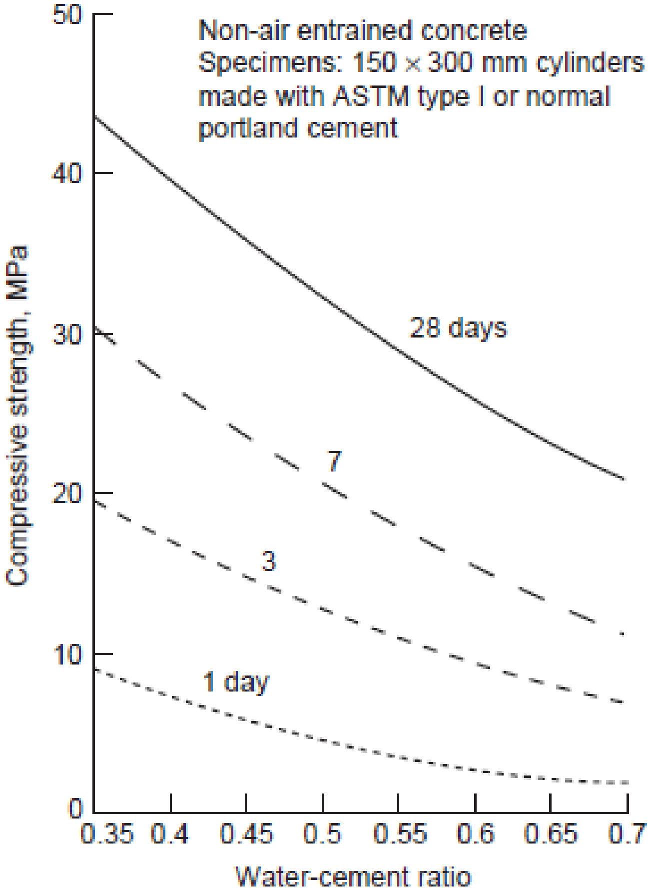


Figure 3-3 Influence of the water-cement ratio and moist curing age on concrete strength. (From *Design and Control of Concrete Mixtures*, 13th ed., Portland Cement Association, Skokie, Ill., p. 6, 1988.)

Compressive strength of concrete is a function of the water-cement ratio and degree of cement hydration. At a given temperature of hydration, the degree of hydration is time dependent and so is the strength.

Maturity-1

The hydration of cement is greatly affected by both the time and the temperature of hydration, so the gain in strength of concrete is also largely controlled by these two factors.

The concept of maturity which is defined as some function of the product of curing time t and curing temperature T.

The maturity function that best correlates with the strength of concrete is the "Nurse-Saul Expression«, an emprical equation:

$$\text{maturity } (°\text{C} \bullet \text{days}) = \Sigma\, a_t\, (T+10)$$

where a_t is the time of curing in days and T is the temperature in °C.

Maturity-2

There are certain limitations to the use of maturity for predicting concrete strengths:

1. Maturity functions do not take into consideration the effect of humidity conditions.
2. Proposed functions can not be applied to mass concrete, because the rate of heat loss from such concrete is much less than that from normal members. In other words, only the ambient heat is considered but the effect of heat of hydration is ignored.
3. Maturity functions are not applicable to very low maturities.
4. Type of cement, W/C ratio, etc. are not considered in these functions.
5. Accelerated curing may lead to contradictory results.

Maturity-3

Nevertheless, in spite of these limitations, the maturity concept may be useful **when trying to establish the strength of concrete in a structure at some previous time. This may be done by measuring the core strengths at some time and then using the maturity functions to estimate the strength at some earlier time.**

Also, the maturity concept can be used **to estimate the appropriate time for form removal when concreting at lower-than-normal temperatures.**

Effect of Curing Conditions on Concrete Compressive Strength

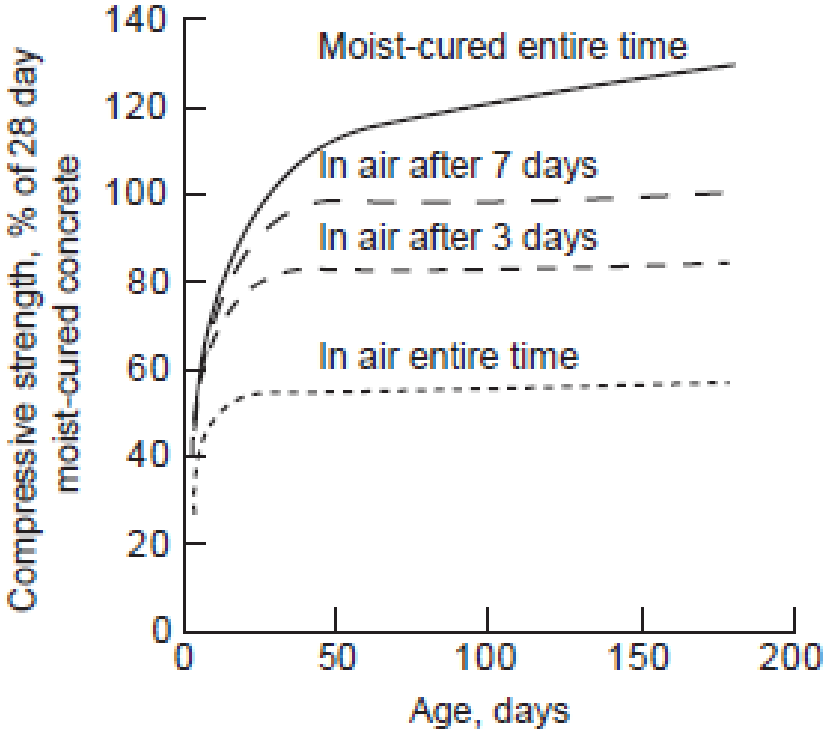


Figure 3-8 Influence of curing conditions on strength. (From *Concrete Manual*, 8th ed., U.S. Bureau of Reclamation, 1981.)

The curing age would not have any beneficial effect on the concrete strength unless curing is carried out in the presence of moisture.

Aggregate Properties

Below aggregate properties increase the water requirement of concrete;

- Aggregates with bad gradation,
- Odd shaped and rough textured particles,
- Smaller Dmax
- Presence of excessive amounts of clay lumps or very fine particles in the aggregate (also affects bonding),

An increase in water content leads to a decrease in the strength of concrete.

The concrete aggregates should be hard, strong and clean. Weak and friable aggregates cause low strength concretes.

Maximum Size of Aggregate

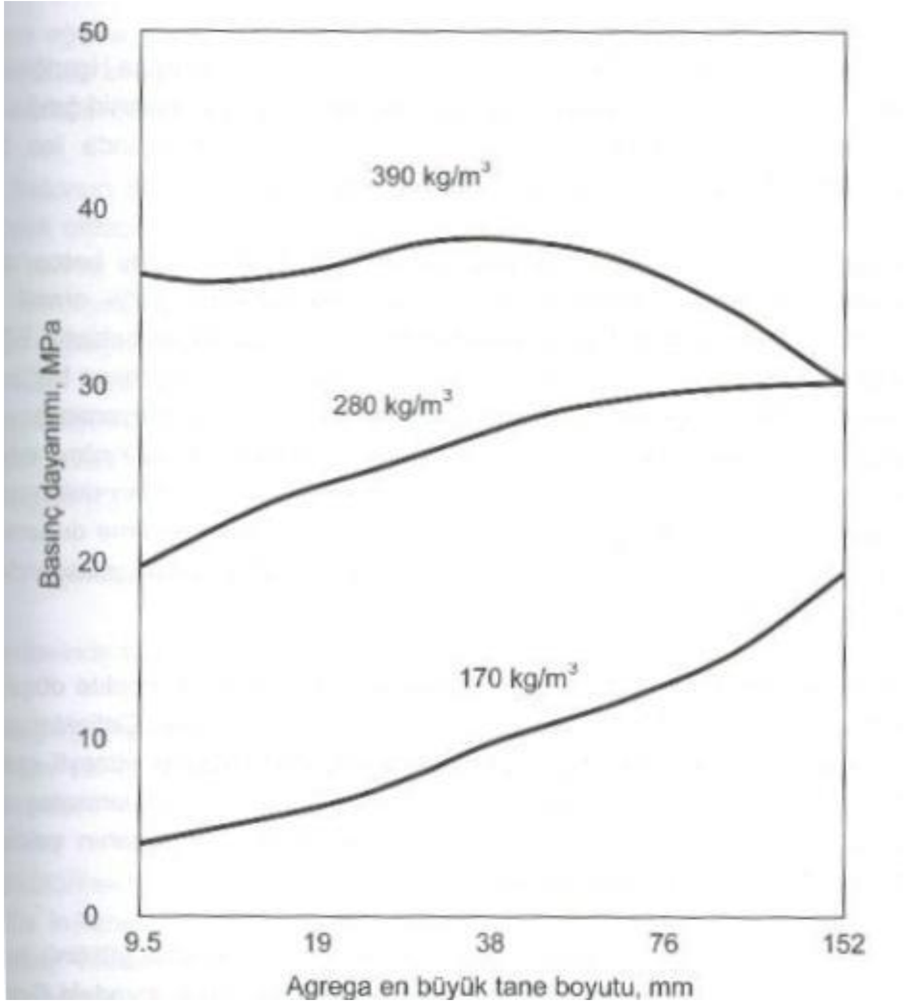
In practice, maximum size of aggregate is limited by member dimensions and minimum reinforcement bar spacing. Below these structural limitations, various maximum sizes can be used.

For smaller aggregate sizes, bonding surface between the aggregate and the paste is larger, therefore the interfacial bond is stronger. However, in this case, water requirement for a specified workability becomes higher and paste gets weaker.

On the other hand, larger aggregate particles provide more restraint on volume changes in the paste and thus may induce additional stresses in the paste, which tend to weaken the concrete. This effect is offset, however, by the reduced water content necessary to achieve a given workability.

In general, there is an optimum maximum aggregate size to give the highest strength.

Dmax vs. Compressive Strength



Rate of Loading

Slow or rapid rates of loading may give misleading strength results. Therefore, standards always specify ranges for loading rates. For compressive strength testing this rate is ~ 2 kgf/cm^2 /sec. Loads applied slower than standard rates cause creep and reduce the apparent strength.

Properties of Cement

Compound composition and fineness of cement affect the rate and amount of the C-S-H gel production upon hydration.

As the gel production increases, the capillary porosity in the cement paste decreases.

Therefore, the strength and durability increase.

Quality of Mixing Water

Excessive amount of impurities in mixing water may affect setting time, strength and durability of the concrete adversely.

The type and amount of impurities in mixing water can be determined by chemical analysis.

A method for investigating water quality;

- Cast concretes with suspected water and good water,
- Compressive strength of the concrete made with the suspected water ≥ 90% of the strength of concrete made with the good quality water.

MIXING OF CONCRETE

Mixing time should be sufficient to produce a uniform concrete. The time of mixing depends on the type of mixer and also to some properties of fresh concrete.

- Undermixing → non-homogeneity
- Overmixing → danger of water loss, breakage of aggregate particles

Effects of Mixing, Transportation, Placing and Consolidation on Strength

Improper transportation and placing may lead to segregation, which in turn may adversely affect the strength.

Fresh concrete placed in molds or forms needs to be properly consolidated in order to remove the excess entrapped air.

Inadequate compaction ⟶ large number of voids

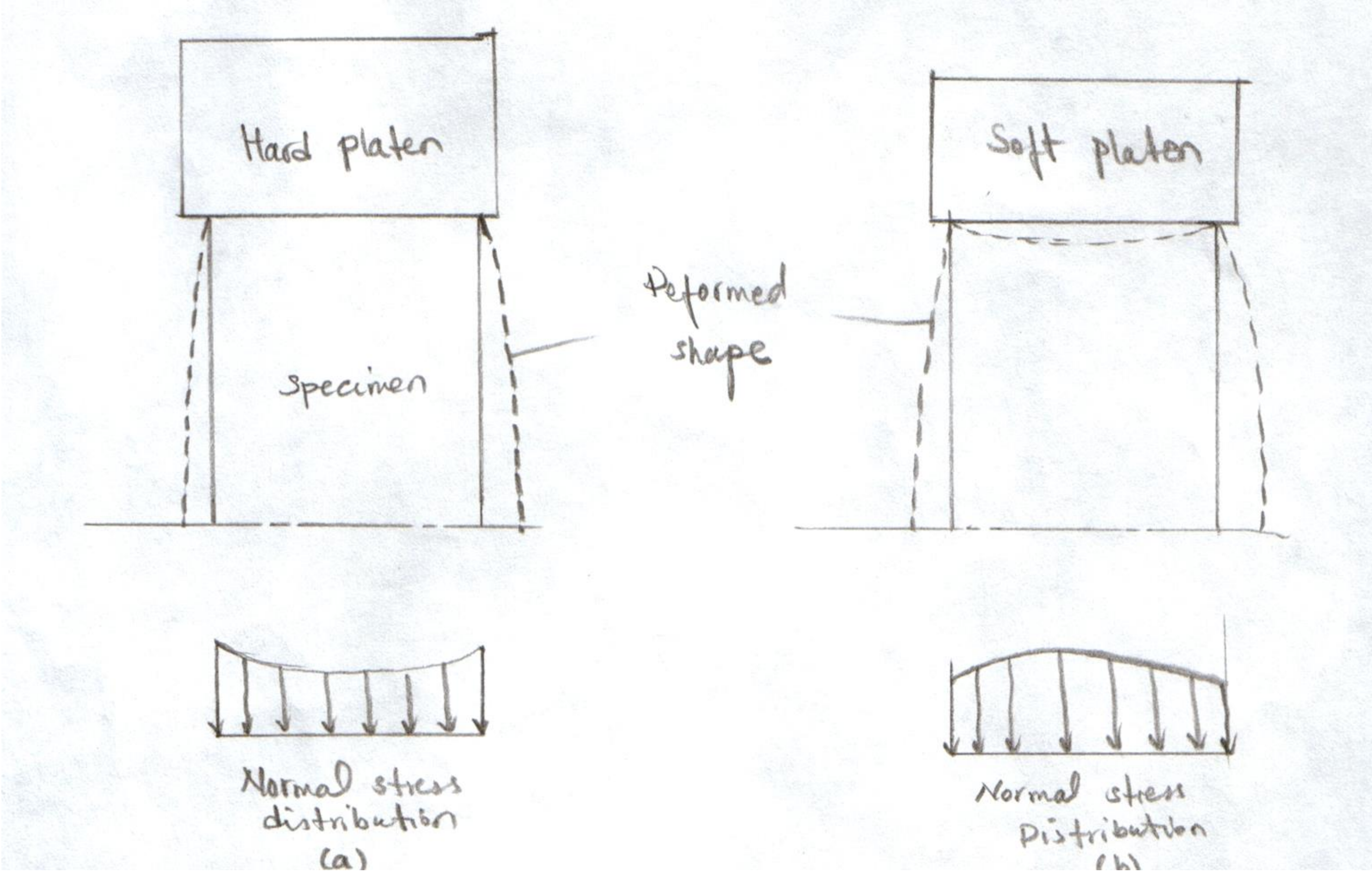
Over vibration ⟶ segregation

In both cases, strength decreases!

COMPRESSIVE STRENGTH-1

Compressive strength is the most significant strength for concrete since the concrete members are primarily designed for compressive loads. Furthermore, some reliable correlations exist between the compressive strength and other strengths and properties of practical significance.

Certain characteristics of the testing machine may affect the compressive strength of concrete. Testing machines may be classified as **"hard" (very rigid machines)** and **"soft"** (less **rigid machines).** In very soft machines, the energy stored in the machine is released as the specimen begins to fail; this additional energy will cause greater crack propagation and failure at lower loads than with very rigid machines, which cannot release their energies as easily.



Specimen deformation and normal stress distribution for (a) hard and (b) soft platens

COMPRESSIVE STRENGTH-2

The difference in the properties of the steel platen of the test machine and those of the concrete specimen will lead to certain discrepancies at the "ends" of the specimen. In other words, concrete near the steel platen will be in lateral compression. This is called the **"end effect".**

END EFFECT

ε_{XS} : lateral strain in steel

ε_{XC} : lateral strain in concrete

E_S : Modulus of elasticity of steel; $\sim 2\times10^6$ kgf/cm^2

E_C : Modulus of elasticity of concrete; $\sim 2\times10^5$ kgf/cm^2

μ_S : Poisson's ratio of steel; ~0.3

μ_C : Poisson's ratio of concrete; ~0.15

at the steel platen concrete specimen interface

$$\varepsilon_x = -\mu \frac{\sigma_y}{E}$$

$$\frac{\varepsilon_{xc}}{\varepsilon_{xs}} \cong \frac{-0.15\dfrac{P}{A(2\times10^5)}}{-0.30\dfrac{P}{A(2\times10^6)}} \Rightarrow \varepsilon_{xc} \cong 5\varepsilon_{xs}$$

∴ concrete near steel platens will be in lateral compression.

L/D RATIO (H/D Ratio)

End effect diminishes at a certain distance from the ends. When the length of the specimen is ≥1.7 times its diameter, the effect of the distorted region is eliminated. This is the reason why length-to-diameter ratio (l/d) of 2 is used.

When, $l/d<1.5$, the apparent strength will be higher because of the end effect. If $2.5>l/d>1.5$, true strength with ±5% error will be achieved. The apparent strength will be lower due to buckling and bending when $l/d>2.5$.

Influence of L/D Ratio on Concrete Strength

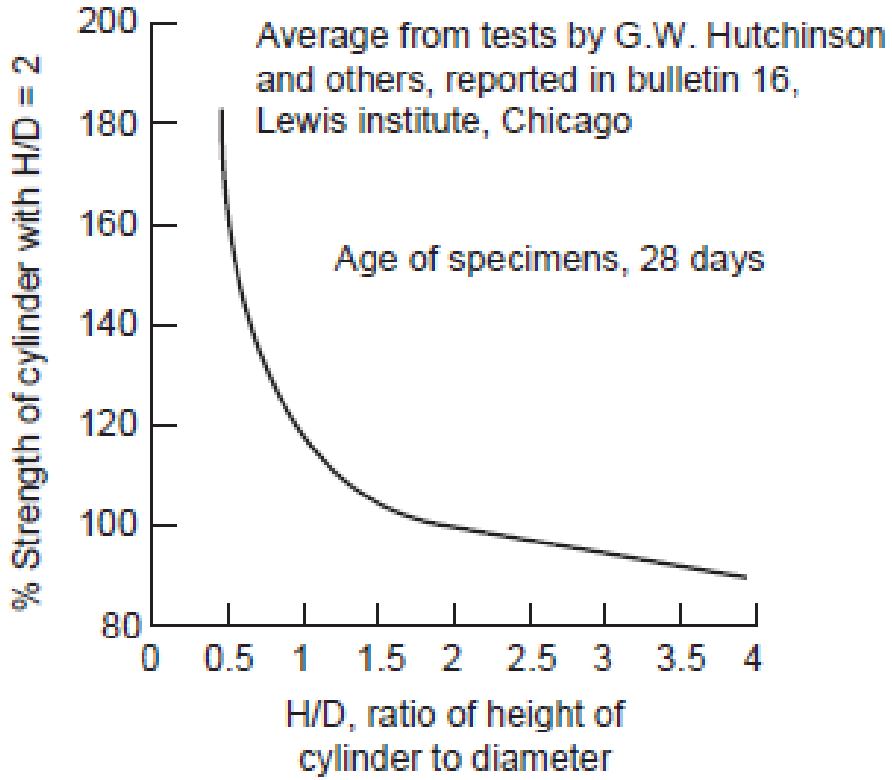


Figure 3-11 Influence of varying the length/diameter ratio on concrete strength. (From *Concrete Manual*, U.S. Bureau of Reclamation, pp. 574–575, 1975.)

SIZE EFFECT

- As most of the other real materials, larger concrete specimens give lower apparent compressive strength.
- However, after a certain large size, “size effect” becomes negligible.

Table 6.2 Factors for Converting Apparent Strength to Standard Strength

Cube specimen dimensions (mm)	Factor to convert to 200mm cube strength
80	1.18
120	1.12
150	1.06
200	1.00
250	0.94

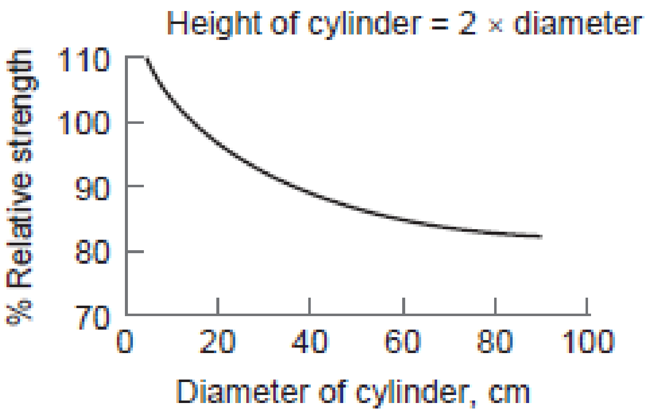
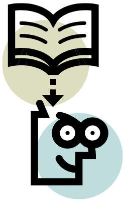


Figure 3-10 Influence of the specimen diameter on concrete strength when the height-diameter ratio is equal to 2. (From *Concrete Manual*, U.S. Bureau of Reclamation, pp. 574–575, 1975.)

Specimen geometry can affect the laboratory test data on concrete strength. The strength of cylindrical specimens with a slenderness ratio (H/D) above 2 or a diameter above 30 cm is not much influenced by the size effects.

CUBES OR CYLINDERS ?



In different parts of the world, either cylinder specimens (150x300 mm) or cube specimens (200 mm) are used as standard compressive strength specimens. The relationship between cylinder strength and cube strength is given by,

$$\sigma_{cube} = k\sigma_{cyl}$$

where, k is a constant depending on the cement content of the mix.

Cubes are more practical to test, but for at least research purposes, cylinders are preferred in most countries.

Compressive strength test on cylindrical specimens

Compressive strength test on cube specimens

COMPRESSIVE STRENGTH EQUATIONS-1

1. Abrams:

$$R = \frac{A}{B^{\frac{w}{C}}}$$

where,

R: strength of concrete

W: weight of water in the batch

C: weight of cement in the batch

A and B: constants

COMPRESSIVE STRENGTH EQUATIONS-2

2. Bolomey:

$$R = K\left(\frac{C}{W} - 0.5\right)$$

where K is a constant which depends on

- strength of cement
- aggregate shape and gradation,
- method of compaction,
- curing conditions,
- specimen size and shape.

The average K values were calculated as 147 and 186 for 7 and 28 days.

COMPRESSIVE STRENGTH EQUATIONS-3

3. Graf:

$$R = \frac{r_c}{b}\left(\frac{C}{W}\right)^2$$

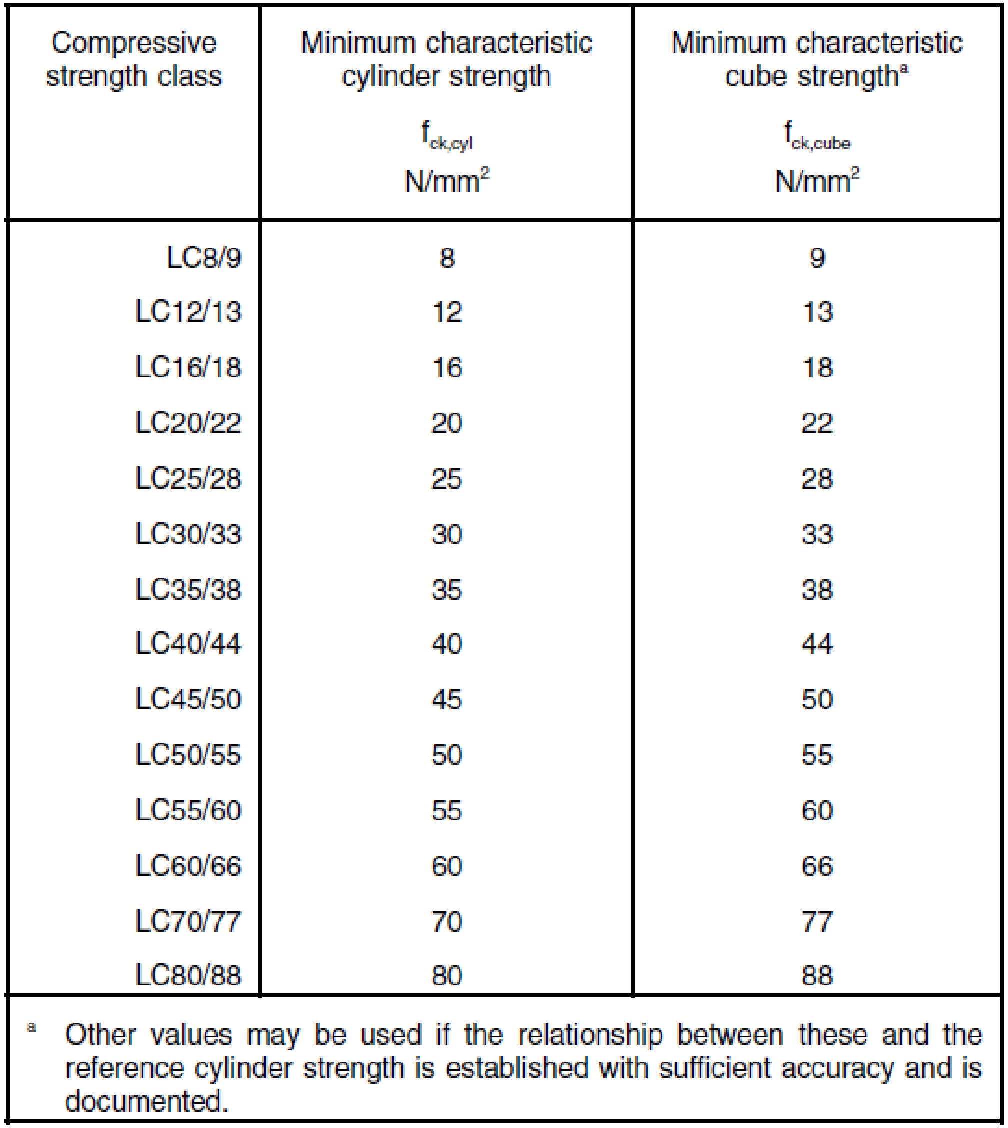
where, r_C = cement strength at a given age

b = a constant ranging between 4.3 and 12.4

Concrete Strength Classes for Normal weight and heavy weight concrete – TS EN 206-1

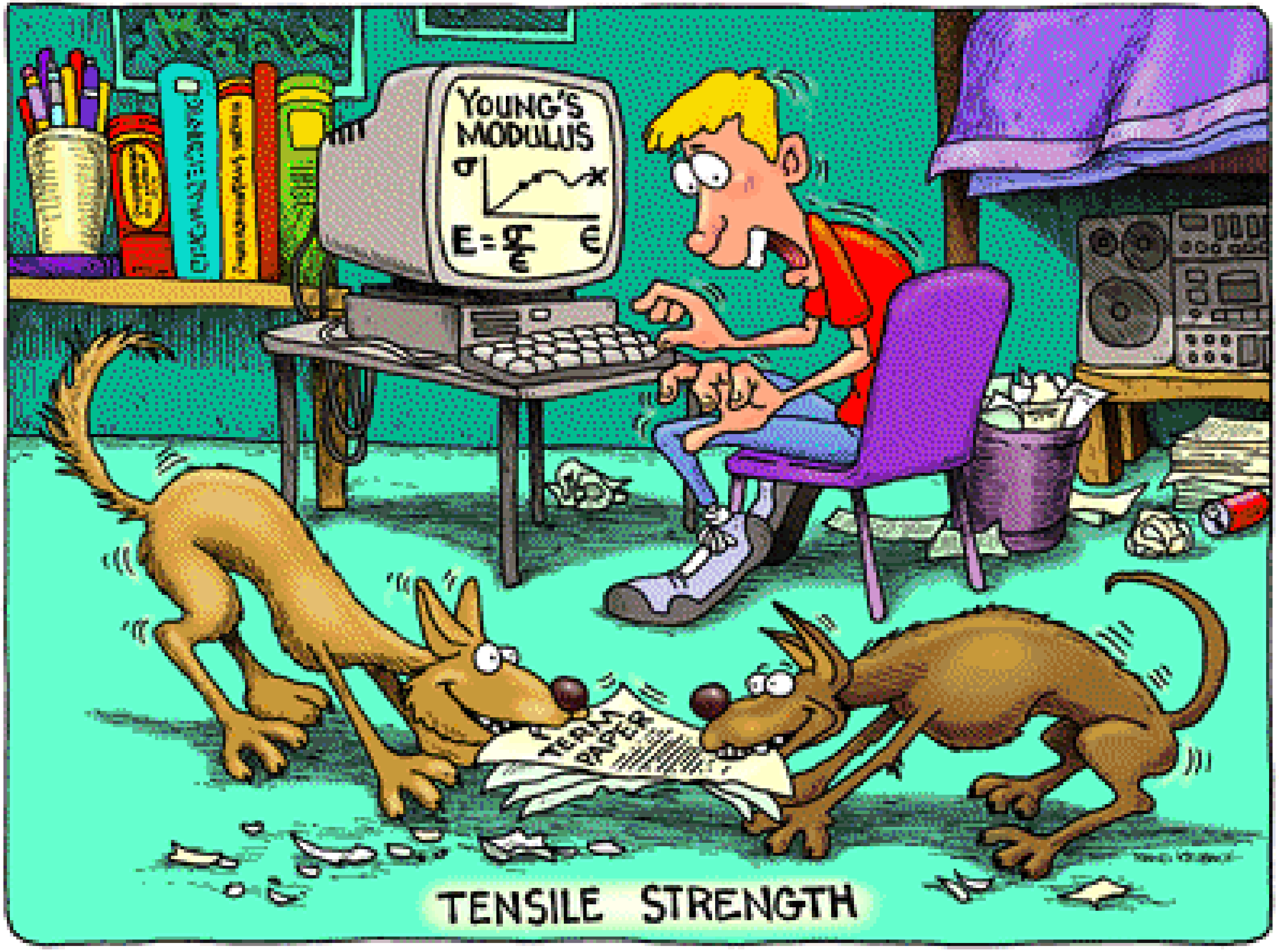
Compressive strength class	Minimum characteristic cylinder strength $f_{ck,cyl}$ N/mm^2	Minimum characteristic cube strength $f_{ck,cube}$ N/mm^2
C8/10	8	10
C12/15	12	15
C16/20	16	20
C20/25	20	25
C25/30	25	30
C30/37	30	37
C35/45	35	45
C40/50	40	50
C45/55	45	55
C50/60	50	60
C55/67	55	67
C60/75	60	75
C70/85	70	85
C80/95	80	95
C90/105	90	105
C100/115	100	115

Concrete Strength Classes for lightweight concrete – TS EN 206-1



Compressive strength class	Minimum characteristic cylinder strength $f_{ck,cyl}$ N/mm^2	Minimum characteristic cube strength[a] $f_{ck,cube}$ N/mm^2
LC8/9	8	9
LC12/13	12	13
LC16/18	16	18
LC20/22	20	22
LC25/28	25	28
LC30/33	30	33
LC35/38	35	38
LC40/44	40	44
LC45/50	45	50
LC50/55	50	55
LC55/60	55	60
LC60/66	60	66
LC70/77	70	77
LC80/88	80	88

[a] Other values may be used if the relationship between these and the reference cylinder strength is established with sufficient accuracy and is documented.

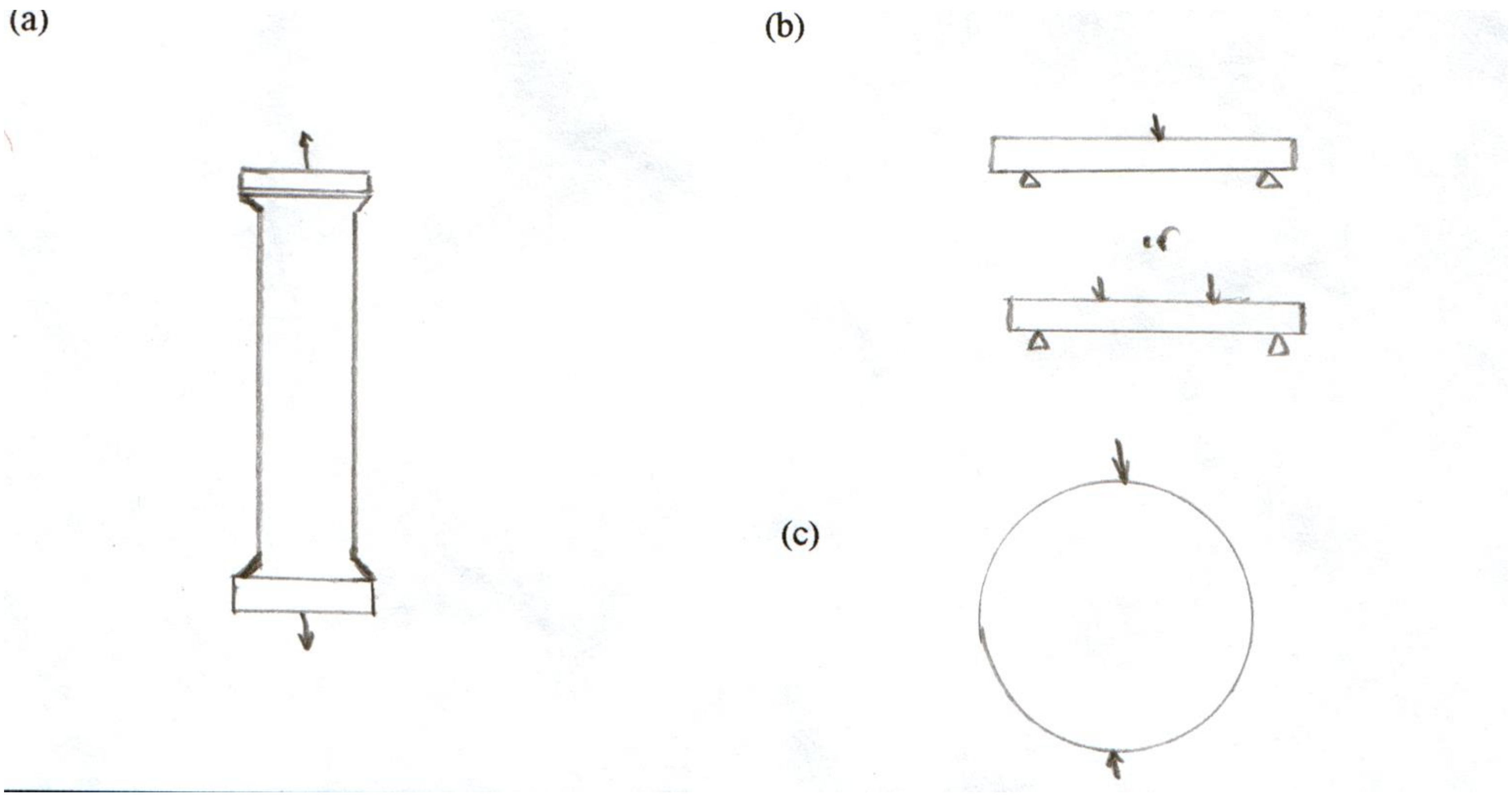

YOUNG'S MODULUS
σ
E = σ/ε
ε
TERM PAPER
TENSILE STRENGTH

TENSILE STRENGTH OF CONCRETE

Tensile strength of concrete is not as important as the compressive strength. However, it still deserves attention since the crack arresting is not very effective under tension and these cracks increase the corrosion of the reinforcement bars.

There are three methods of determining the tensile strength of concrete:

1. Direct tension
2. Tension under flexural loading
   a) center-point loading
   b) third point loading
3. Split tension



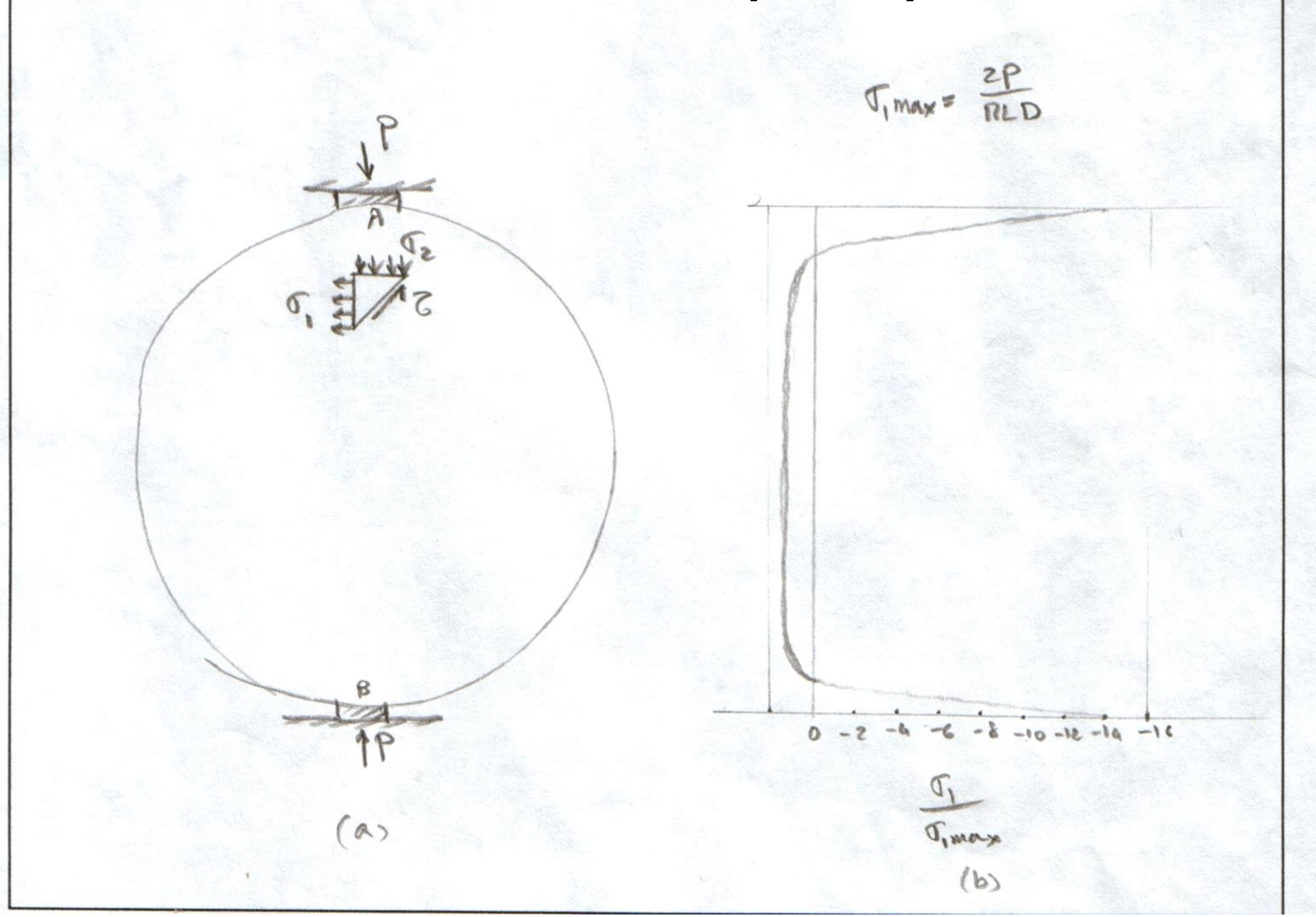
Methods of tension testing: (a) Direct tension; (b) Flexural tension; (c) Split tension.

METHODS OF TENSION TESTING

Among these methods, direct tension gives the most correct results as long as it is carried out with extreme care. However, this method has certain difficulties from application point of view. It is not commonly used.

There are, two common methods for estimating the tensile strength of concrete through indirect tension tests: **The split tension test** is carried out on standard cylinder specimens, tested on its side in diameter compression.

Stress distribution in split-cylinder test-1



(a)

(b)

Split tensile strength

vertical compression,

$$\sigma_c = \frac{2P}{\pi LD}\left[\frac{D^2}{r(D-r)} - 1\right]$$

horizontal tension,

$$\sigma_t = \frac{2P}{\pi LD}$$

the horizontal tensile strength tested on cube specimens is

$$\sigma_t = \frac{2P}{\pi a^2}$$

In general, it has been found that the values of tensile strength obtained from splitting tests tend to be about 15% higher than values obtained from direct tension tests.

Flexural Tension Tests-1

The other way of estimating the tensile strength is by **flexural** tests. Flexural tension tests are carried out in two ways:

(1) Center point loading, where the load is applied at the center of the span length

(2) Third point loading, where the load is applied at two points 1/3 of the distance away from the supports.

In both of these cases the strength is calculated

From

$$\sigma = \frac{Mc}{I}$$

where M is the maximum moment, c is the distance to the neutral axis; I is the moment of inertia.

Flexural Tension Tests-2

Among these two methods, third-point loading is more common and gives statistically more meaningful results. In this method, the theoretical maximum tensile strength, or **Modulus of Rupture,** σ_r *is* calculated as follows

$$\sigma_r = \frac{Pl}{bd^2}$$

where P is the maximum load applied; L is the span length; b is the specimen width; and d is the specimen depth.

This equation holds true if the specimen breaks between the two interior loading points (i.e., in the middle third of the specimen). If the beam breaks outside these points by not more than 5% of the span length, the following equation should be used:

$$\sigma_r = \frac{3Pa}{bd^2}$$

where a is the average distance between the point of fracture and the nearest support. The results of tests where failure occurs even closer to the supports are discarded.

Flexural Tension Tests-3

Flexural tensile strength test tends to overestimate the true tensile strength by about 50%. However it is still a useful test for conditions where concrete is to be used under flexure, such as highways, airport runways, etc.

Estimation of Concrete Strength by Cores

TS EN 13791: 2010 Assessment of in-situ compressive strength in structures and precast concrete components

TS EN 12504-1:2002 Testing concrete in structures – Part 1: Cored specimens. Taking, Examining and Testing in Compression

Why Cores are Taken?

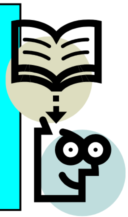
- To determine the compressive strength of concrete in structure which is still in service and to compare the result with the relevant standards.

- To comment on the correctness of placement, compaction and curing operations that were applied to the concrete.

- To determine whether compressive strength loss happened due to some reasons, etc., overloading, fire, freeze-thaw, other durability problems.

- To investigate the mezo-micro structure of concrete in order to find the evidence of ASR, carbonation, etc.

TS EN 13791
Assessment of in-situ compressive strength in structures and precast concrete components



This standard applies where cores are used to;

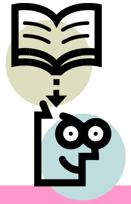
- Determine the conformability of concrete strength in the structure,
- Evaluate the old structures that was damaged or in need of modification/redesign
- Solve the conflictions about concrete quality,

Evaluation of the results of indirect test methods (rebound hammer, pull-out, ultrasonic pulse velocity, e.g.) correlated with core strength tests.

Determination of Compressive Strength

- l/d :1 and d:100 mm core str. = 150 mm standard cube str.
- l/d: 2 and d:100-150 mm core str = 150x300 cylindrical str
- See the relevant standard (TS EN 13791) for statistical evaluation of results.

How to take cores from the structure?



1. Cores should be taken at least from 14 day old concretes,

2. Cutting the reinforcement should be avoided while taking cores,

3. H/D ratio should never be less than 1,

4. At least 100 mm or 150 mm diameter cores should be taken, if core diameter is less than 100 mm, number of core samples should be doubled. Cores less than 50mm should never be used for strength determination,

5. Aggregate D_{max} / core diameter should be at least 1/3.

6. Core caps should be cutted parallel to each other,

7. 3 - 5 mm thick capping should be made with a suitable material,

8. Cores are kept under water at room temperature for 2-3 days in order to bring to saturated condition just before the application of strength test,

10. Compressive strength test is applied at a speed of 1.4-3.5 kgf/cm^2/sec.

Factors Affecting Core Strength;

Samples size (diameter and height);

size ⇧ strength ⇩

Presence of reinforcement

reinforcement present → strength ⇩

Rate of loading; rate ⇧ strength ⇧

Direction of coring; horizontal → strength ⇩

vertical → strength ⇧

Age of conrete; age ⇧ strength ⇧

Moisture conditions of concrete; moisture ⇧ strength ⇩

Other testing conditions..


P.18
P.2
P.21

CONCRETE TECHNOLOGY

STRESS-STRAIN RELATIONS and ELASTIC CONSTANTS

STRESS-STRAIN RELATIONS AND ELASTIC CONSTANTS

Concrete is not a truly elastic material. The nonlinearity of the stress-strain curve is obvious even at the origin. This fact creates certain problems in determining the elastic constants for concrete. Nevertheless, there are certain approximate methods of determining these constants.

DETERMINATION OF MODULUS OF ELASTICITY

Experimental methods;

(1) Initial Tangent Method: The slope of the tangent to the curve at origin.

(2) Tangent Method: The slope of the tangent to the curve at 15, 25, 35, or 50% of the ultimate strength.

(3) Secant Method: The slope of the secant drawn from the origin to a point on the stress-strain curve corresponding to 15, 25, 35, or 50% of the ultimate strength.

DETERMINATION OF MODULUS OF ELASTICITY

In the ACI Building Code the relationship used is:

$E = 0.043\rho^{\frac{3}{2}}\sqrt{\sigma_c}$ MPa where,

E is the secant Modulus at ~ 40% of the ultimate strength in MPa; ρ is the unit weight of concrete in kg/m^3; and σ_c is the compressive strength of standard cylinder specimen in MPa.

In the British Code of Practice, Modulus of Elasticity is;

$E = 9.1\rho\sqrt[3]{\sigma_c}$ GPa where,

σ_c is the compressive strength of cube specimens.

CEB uses the following expression:

$E = 9.5(\sigma_c + 8)^{1/3}$ GPa where,

σ_c is the nominal design strength.

OTHER ELASTIC CONSTANTS OF CONCRETE

Poisson's Ratio, μ, can be conveniently determined by direct strain measurements in uniaxial compression. It generally falls in the range of 0.15 to 0.20.

$$\mu = -\frac{\varepsilon_{lateral}}{\varepsilon_{longitudinal}}$$

Shear Modulus, G, is not obtained by direct measurement. Rather, it is calculated from the elastic relationship

$$G = \frac{E}{2(1+\mu)}$$

Bulk Modulus, K, is similarly determined by calculation:

$$K = \frac{E}{3(1-2\mu)}$$

EQUATIONS FOR ESTIMATING THE STRESS-STRAIN CURVES

1. Woellmy Parabola:

$$\sigma = \sigma_u \frac{\varepsilon}{\varepsilon_{\max}}\left(2 - \frac{\varepsilon}{\varepsilon_{\max}}\right)$$

where σ_u is the ultimate strength

ε_{max} : maximum strain

2. Smith - Young Equation:

$$\sigma = \sigma_u \frac{\varepsilon}{\varepsilon_{\max}} e^{\left(1-\frac{\varepsilon}{\varepsilon_{\max}}\right)}$$

EQUATIONS FOR ESTIMATING THE STRESS-STRAIN CURVES

3. Desayi - Krishnan Equation:

$$\sigma = \frac{E\varepsilon}{1+\left(\frac{\varepsilon}{\varepsilon_o}\right)^2}$$

where, ε_o *is the strain corresponding to* σ_u*,*

E is the initial tangent modulus

For practical purposes, ε_o *can be taken as 0.02% and* ε_{max} *can be calculated from*

$$\varepsilon_{\max} = 0.0004 - \frac{\sigma_c}{6.5x10^6}$$

where, σ_c *is the ultimate strength in kgf/cm*^{2}

EQUATIONS FOR ESTIMATING THE STRESS-STRAIN CURVES

The relationship between axial, lateral and volumetric strains in concrete under uniaxial compressive loading is given by the following equation:

$$\varepsilon_v = \varepsilon_a(1-2\mu) = \varepsilon_a - 2\varepsilon_l$$

where, ε_a *,* ε_l*, and* ε_v *are axial, lateral and volumetric strains, respectively.*

Stress-strain relationship for concrete

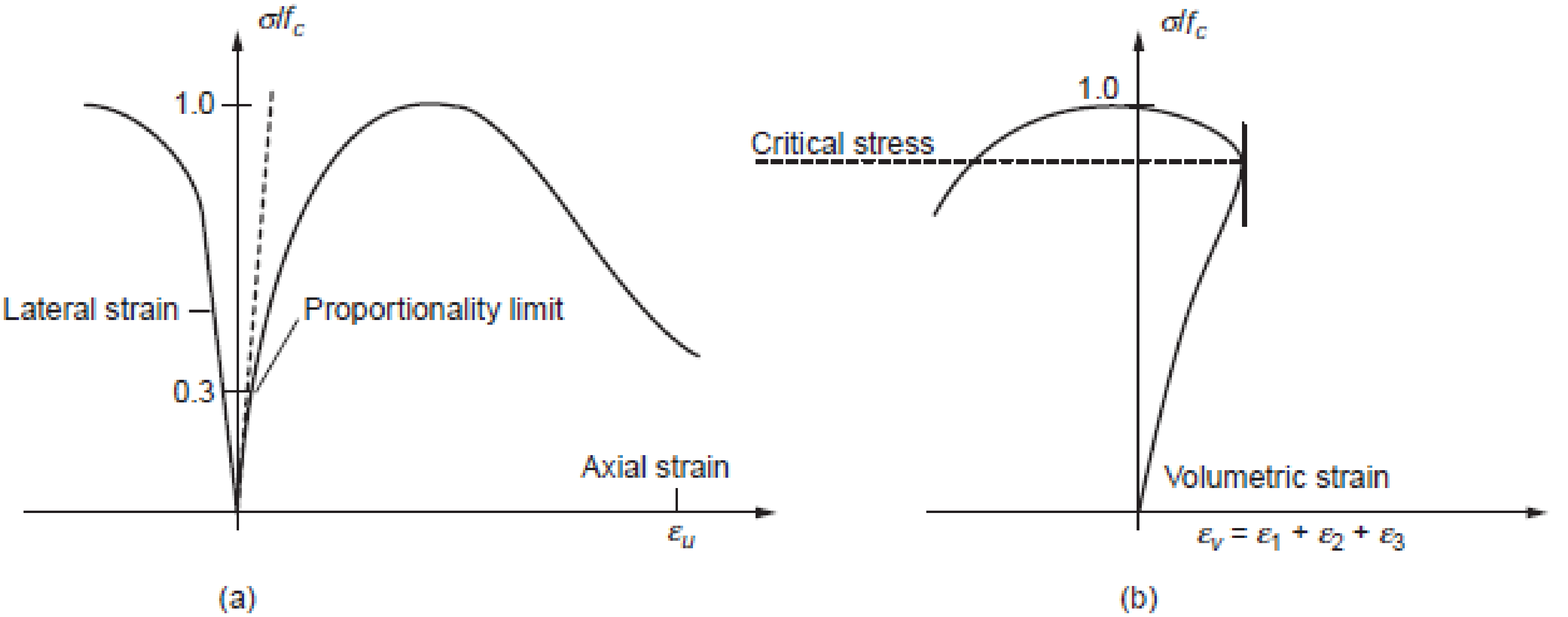
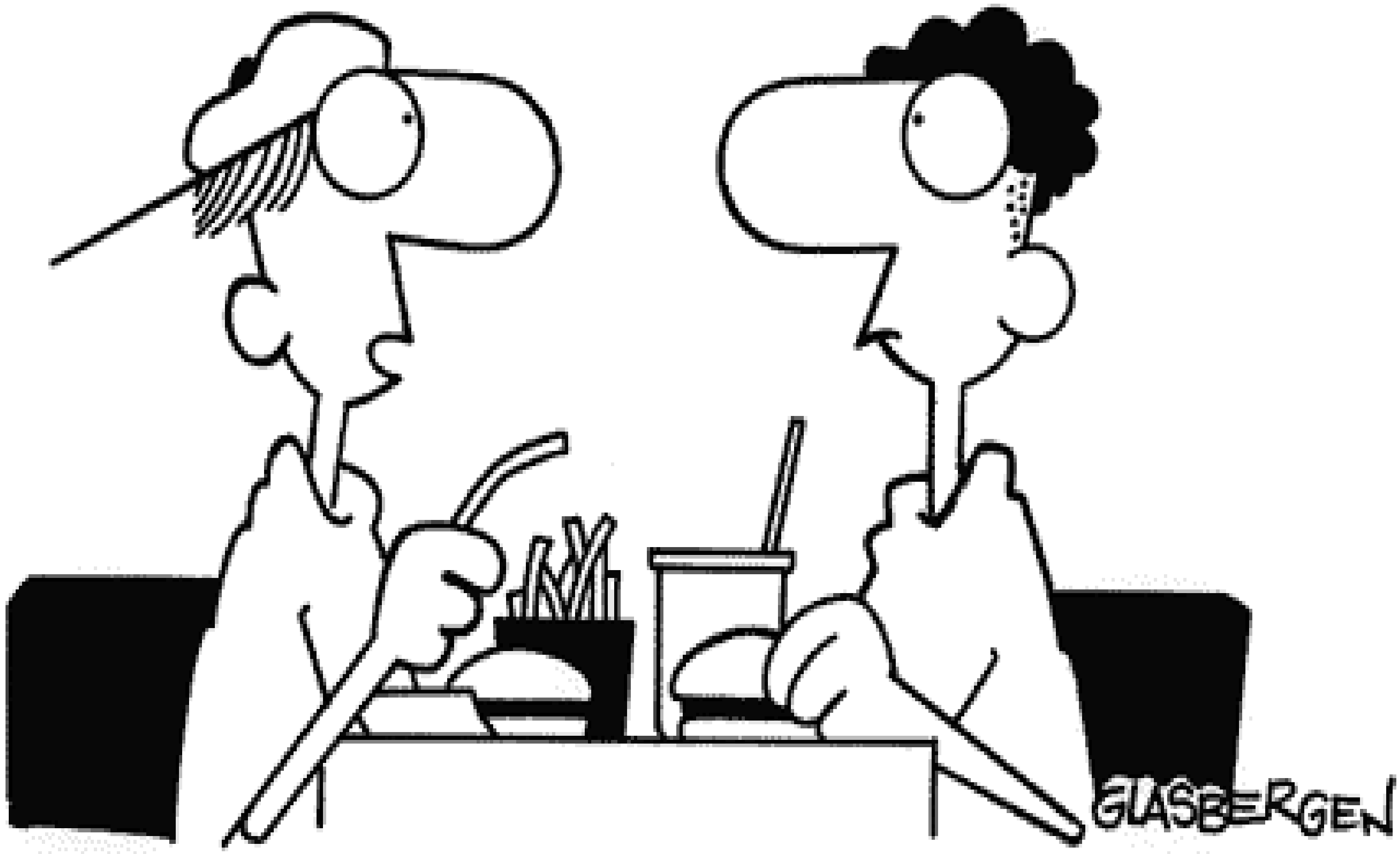


Figure 3-13 Typical plots of compressive stress vs. (*a*) axial and lateral strains, and (*b*) volumetric strains. (From Chen, W.F., *Plasticity in Reinforced Concrete*, McGraw-Hill,, New York, p. 20, 1982.)



"I forgot to make a back-up copy of my brain, so everything I learned last semester was lost."

CONCRETE TECHNOLOGY

LECTURE NOTES-CHAPTER 9
QUALITY CONTROL

QUALITY CONTROL

There are numerous factors involved in the production of a high quality concrete:

- materials,
- proportioning,
- handling and placing,
- curing,
- testing, etc.

It should, therefore, come as no surprise that concrete, in common with other engineering materials, is inherently a *variable* material.

That is, tests on nominally identical samples of concrete will show some variation in properties.

Factors that contribute to this variability may be grouped as follows:

(1) *Materials.* This includes variability in the cement; grading, moisture content, mineral composition, physical properties, and particle shape of the aggregate; and admixtures used.

(2) *Production.* This involves the type of batching plant and equipment, the method of transporting the concrete to the site, and the procedures and workmanship used to produce and place concrete.

(3) *Testing.* This includes the sampling procedures, the making and curing of test specimens, and the test procedures used.

The term **quality assurance** is frequently used in the construction industry. Quality assurance refers collectively to all of the steps taken to ensure adequate confidence that the concrete will perform satisfactorily in service.

Quality control applies to each action used to measure the properties of the concrete, or its components, and to control them within the established specifications.

MEASUREMENT OF VARIABILITY - 1

Before discussing the quality control, it is necessary to define the concept of variability. It has been found that the distribution of concrete strength can best be approximated by the **Normal (Gaussian) Distribution.**

Such a distribution is defined by two parameters: **the mean, μ and standard deviation,** s.

The equation of the normal distribution curve is

$$y = \frac{1}{s\sqrt{2\pi}} e^{\left[\frac{-(x-\mu)^2}{2s^2}\right]}$$

If we represent the values of strength by **x,** then the mean is

$$\mu = \frac{\sum x}{n}$$

MEASUREMENT OF VARIABILITY - 2

The standard deviation is a measure of dispersion or variability of the values. It can be calculated by

$$s = \sqrt{\frac{\sum (x-\mu)^2}{n-1}}$$

or

$$s = \sqrt{\frac{\sum x^2 - (\sum x)^2 / n}{n-1}}$$

The **variance** is defined as s^2, and the **coefficient of variation, V,** as

$$V = \frac{s}{\mu} x 100\%$$

MEASUREMENT OF VARIABILITY - 3

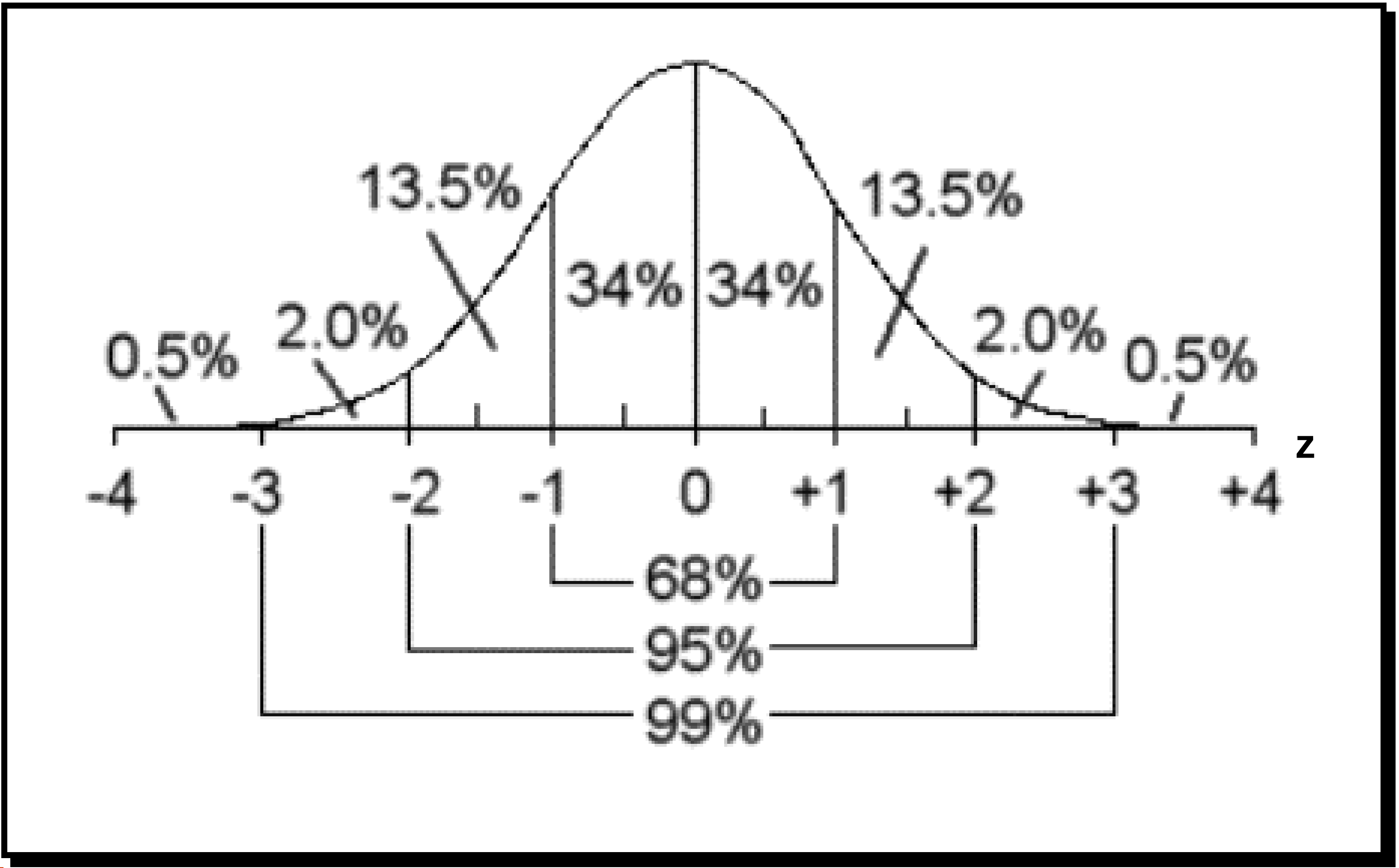
It is possible to "normalize" the equation of normal distribution by making the substitution

$$z = \frac{x - \mu}{s}$$

The value z represents the number of standard deviations that x is away from the mean. Therefore, the normal distribution equation can be rewritten as

$$y = \frac{1}{s\sqrt{2\pi}} e^{-\frac{z^2}{2}}$$

When the function described by this equation is graphed, it gives a bell-shaped curve.


13.5%
13.5%
34%
34%
2.0%
2.0%
0.5%
0.5%
z
-4
-3
-2
-1
0
+1
+2
+3
+4
68%
95%
99%

Properties of normal curve

1. The maximum ordinate is at $z = 0$.
2. The curve is symmetrical about $z = 0$.
3. Points of inflection exist at ±1 standard deviation from the mean. That is, at $z=\pm l$.
4. The total area under the normalized curve is 1.
5. The area under the curve between any two points is the probability of occurrence of a value in that interval. That is, the probability of a value lying within 1 standard deviation of the mean is 68.27%; within 2 standard deviations from the mean, 95.45%; within 3 standard deviations from the mean, 99.73%.
6. If we know the standard deviations s_1, s_2 ..... s_n of all the components that contribute to the total standard deviation, the total standard deviation is given by $s^2 = s_1^2 + s_2^2 + \ldots + s_n^2$

Statistical approach to concrete-1

1. Concrete structures can not be designed on the basis of mean strength. Otherwise, this would mean that 50% of the concrete placed would have strengths below the design value. On the other hand, since concrete strengths are normally distributed, it can not be insisted that all concrete strengths be above the design value. Therefore, we must arbitrarily decide what constitutes an acceptable percentage of specimens falling below the "minimum" design value. Using this percentage and knowing (or assuming) the standard deviation in strength that can be expected, we can determine the required mean strength for which our concrete is to be designed.
2. When carrying out tests on concrete, we are trying to evaluate the distribution in strength of all concrete in the structure, based upon a limited sample size. Therefore we can at best only *estimate* the strength of concrete in the structure.

Statistical approach to concrete-2

3. Because variations in concrete strengths are due not only to mix variations, but also to sampling variations, there are two risks that must be balanced: the "producer's risk" that satisfactory concrete will be rejected, and the "consumer's risk" that bad concrete will be accepted. The consumer's risk can be large if insufficient testing is carried out.
4. There must be some plan of action that can be followed if the concrete is considered not to have complied with the specifications.

ACI APPROACH TO VARIABILITY

ACI approach requires an average strength sufficiently in excess of the specified strength so that only an allowable proportion of low strengths will occur. The required average strength is computed from the following expression:

$$\sigma_{cr} = \frac{\sigma_c^{\prime}}{1 - tV}$$

or

$$\sigma_{cr} = \sigma_c^{\prime} + ts$$

where, σ_{cr} is the required average strength, $\sigma_c^{\prime}$ is the specified design strength, t is a constant depending on the proportion of tests allowed below $\sigma_c^{\prime}$. V is the predicted value of the coefficient of variation, and s is the predicted value of standard deviation. The constant t is obtained from Table 9.1.

Table 9.1. Values of t

% of tests falling within the limits $\mu \pm ts$	Chances of falling below lower limit	t
40	3 in 10	0.52
50	2.5 in 10	0.67
60	2 in 10	0.84
68.27	1 in 6.3	1.00
70	1.5 in 10	1.04
80	1 in 10	1.28
90	1 in 20	1.65
95	1 in 40	1.96
95.45	1 in 44	2.00
98	1 in 100	2.33
99	1 in 200	2.58
99.73	1 in 741	3.00

Reasonable control of structural concrete would be obtained if the probability of a test below the specified design strength is not greater than 1 in 10. However, ACI Building Code puts more restrictions:

1. The probable frequency of tests more than 3.5 MPa below $\sigma_c^{\prime}$ should not exceed 1 in 100. That is,

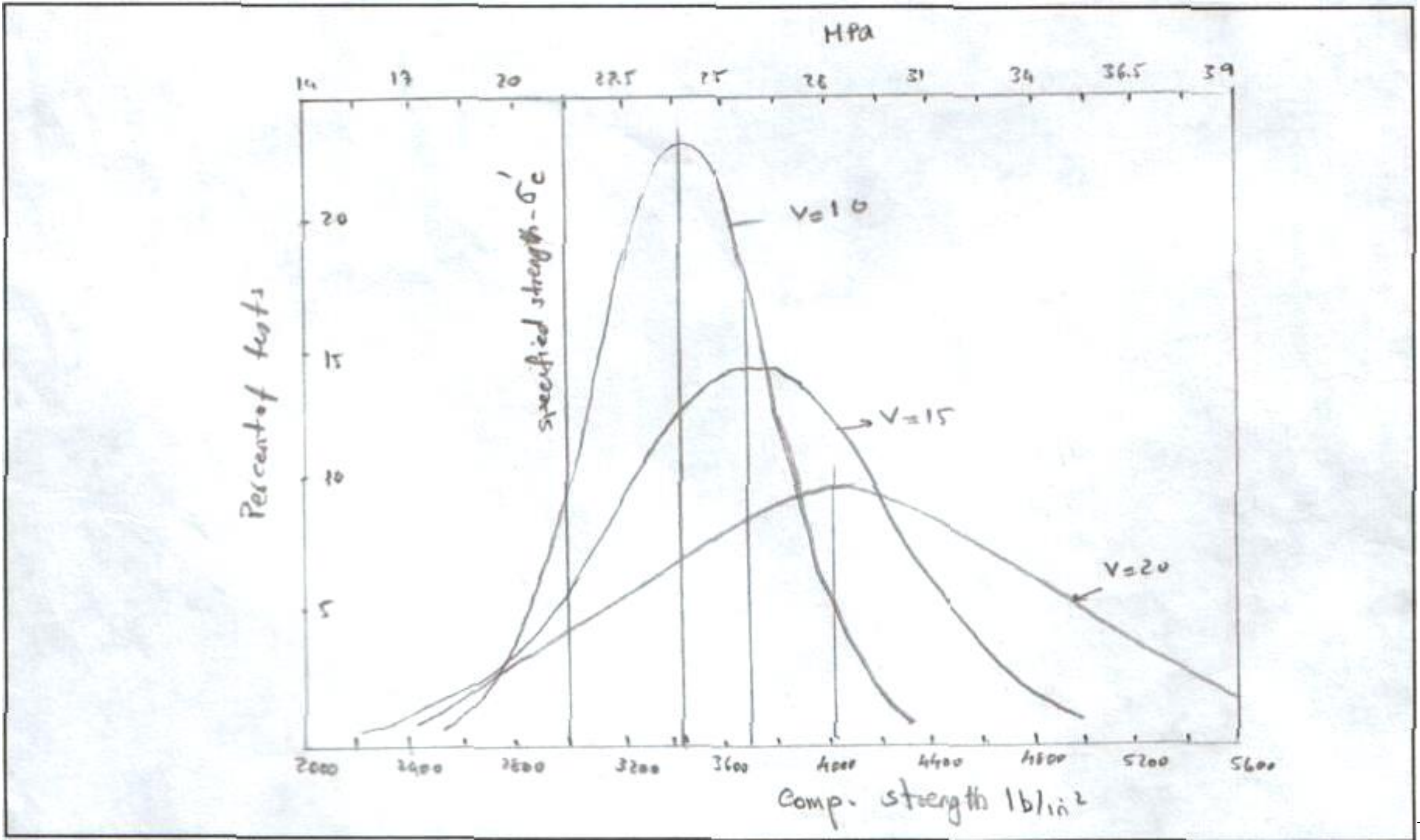
$$\sigma_{cr} = \sigma_c^{\prime} - 3.5 + 2.33s$$

2. The probable frequency of the average of three consecutive tests below $\sigma_c^{\prime}$ will not exceed 1 in 100.

$$\sigma_{cr} = \sigma_c^{\prime} + \frac{2.33s}{\sqrt{3}}$$

When the standard deviation is less than 3.5 MPa, the former equation governs; for larger standard deviations the latter governs.

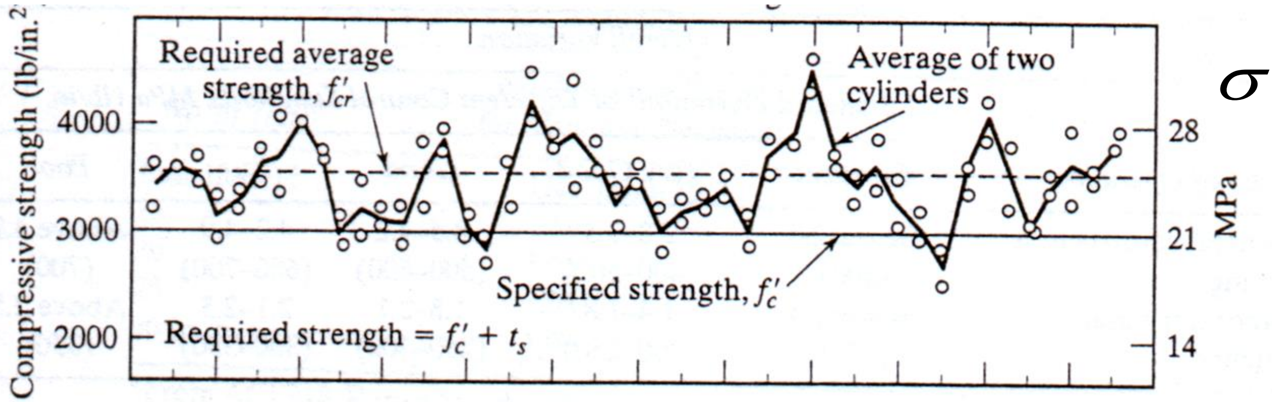
Figure shows the normal distribution curves for concrete with different coefficients of variation. It is obvious that as V increases, the average design strength must be increased. Therefore, **the better the quality control the more economical will be the concrete.**



QUALITY CONTROL CHARTS-1

Concrete production is a continuous process. The changes in either the mean strength or standard deviation may occur at some stage of the construction process. This might be due to the changes in the raw materials (e.g. moisture content of the aggregate), the batching equipment, testing procedures or variability of other reasons. In any event, the sooner such changes are detected, the sooner we can take the appropriate remedial measures. This implies that the concrete strengths must be assessed on a continuous basis. This can most effectively be done by using quality control charts. Three types of charts are employed; in each case the horizontal axis represents the consecutive sample numbers.

QUALITY CONTROL CHARTS-2

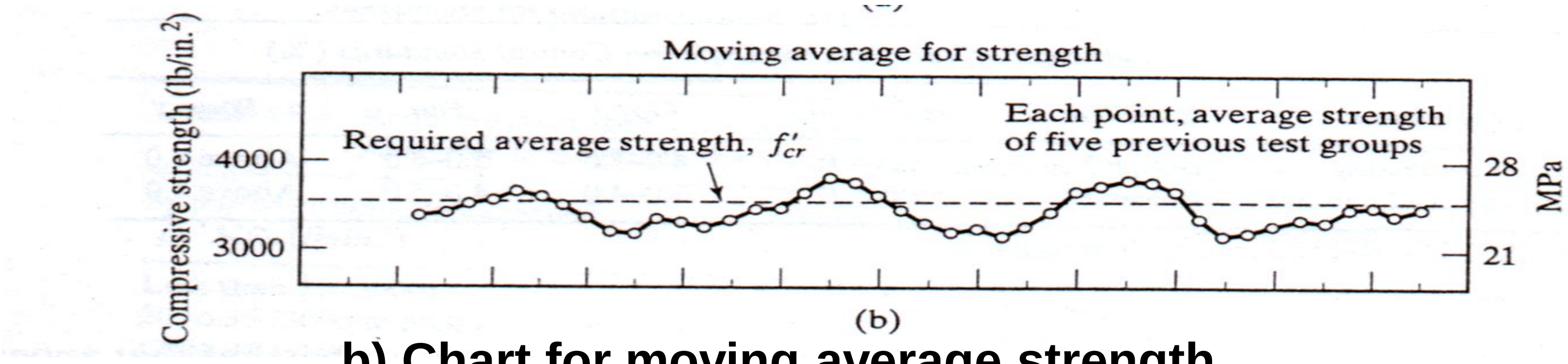


$$\sigma_{cr} = \sigma'_c + ts$$

a) Chart for individual strength values

This chart simply shows the number of “low” tests and gives some indication of the scatter. However, this chart is relatively insensitive to changes in the concrete quality.

QUALITY CONTROL CHARTS-3

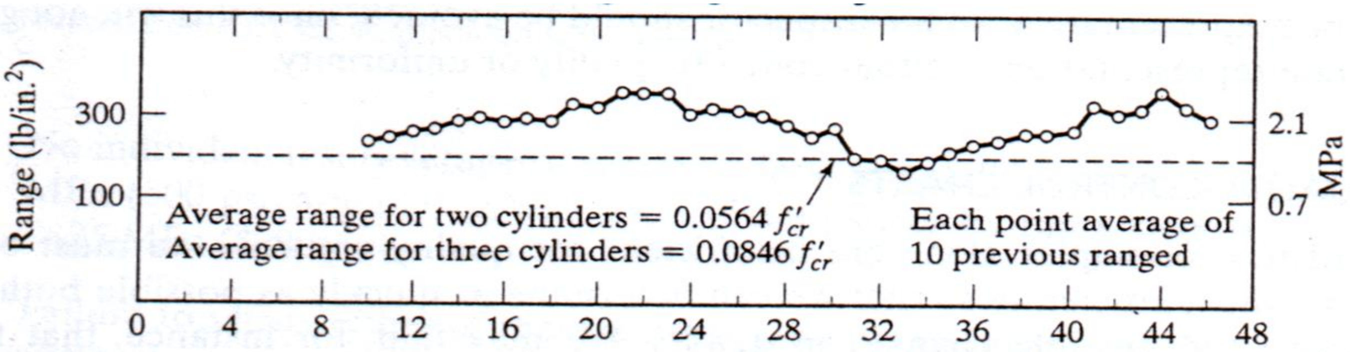


b) Chart for moving average strength

Each point on the graph represents the average of the previous five sets of strength tests. Each strength test will normally consist of breaking two or three specimens. This chart tends to smooth out chance variations and can be used to indicate trends that may be due to the influence of seasonal changes, changes in the procedures and so on. The number of tests to be averaged is determined by the particular job requirements.

This chart would have σ_c' as the lower limit. A failure to meet the limitations of the chart would indicate that the required average strength σ_{cr} is not being reached, due to lower strengths, to more variability than anticipated, or poor testing techniques.

QUALITY CONTROL CHARTS-4



c) The moving average of the *range*

Each point represents the average of the ranges of the 10 previous sets of strength tests. This chart provides a check on the adequacy of the test procedures, as indicated by the range between companion cylinders comprising a strength test, can be expressed by

$$\overline{R}_M = \sigma_{cr} v_1 d_2$$

$\overline{R}_M$: the average range, v_1: within-test coefficient of variation,

d_2 : a constant depending on the number of cylinders tested.

Since v_1 should not exceed 5% for good control, the maximum average range should be,

$\overline{R}_M = (0.05x1.128)\sigma_{cr} = 0.0564\sigma_{cr}$ for **two** companion cylinders

$\overline{R}_M = (0.05x1.693)\sigma_{cr} = 0.08465\sigma_{cr}$ for **three** companion cylinders

CONCRETE TECHNOLOGY

DIMENSIONAL STABILITY OF CONCRETE

DIMENSIONAL STABILITY OF CONCRETE

Concrete will undergo volume changes while in service, due to:

- applied stress,
- change of moisture content and
- changes in temperature.

The response of the concrete to these conditions is complex, resulting in reversible, irreversible and time-dependent deformations.

PLASTIC SHRINKAGE

Loss of water from fresh concrete may cause cracking. Water loss may be due to evaporation of water from the surface or by suction by the underlying dry concrete or subgrade. If the amount of water lost from unit concrete area exceeds the amount of water brought to the surface by bleeding, ***plastic shrinkage cracking*** may occur. This is aggravated by a combination of **high wind velocity, low relative humidity, high air temperature and high concrete temperatures.**

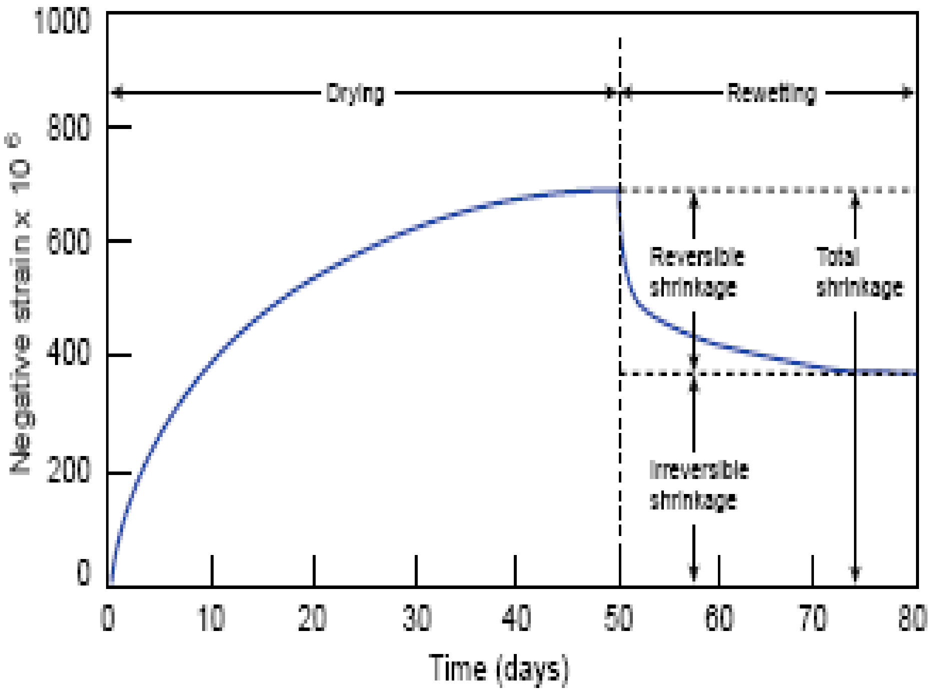
Plastic shrinkage increases by;

- increasing cement content of the mix and
- decreasing its W/C ratio.

Cracking due to plastic shrinkage

DRYING SHRINKAGE

Drying shrinkage is the strain caused by the loss of water from the hardened concrete. In concrete, aggregate has a restraining influence on the volume changes that takes place in the paste.



Typical behavior of concrete on drying and rewetting

- The part of the total shrinkage that occurs on the first drying is irreversible.
- The subsequent volume expansions that occur on rewetting and the volume contractions that occur on subsequent drying are smaller.

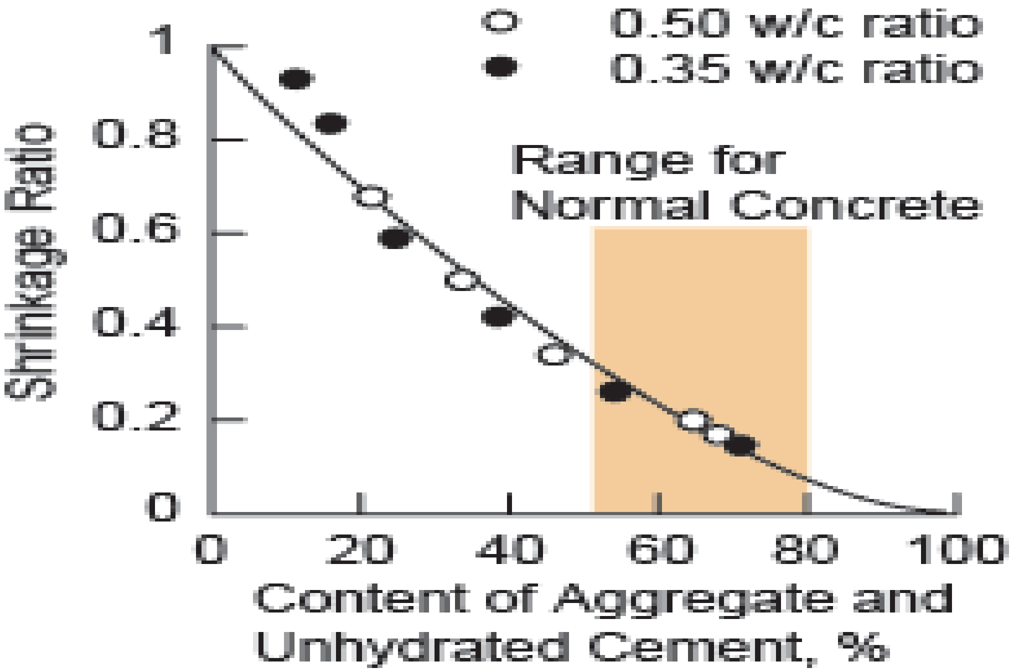
Cracking due to drying shrinkage

Effect of Aggregate on Shrinkage-1

The drying shrinkage of concrete will be less than that of pure paste because of the restraining effect of aggregate. With a few exceptions, aggregates are dimensionally stable under changing moisture conditions. The amount of restraint provided by aggregate depends on;

- the amount of aggregate in concrete,
- its stiffness and,
- the maximum size of coarse aggregate.

Effect of Aggregate on Shrinkage-2



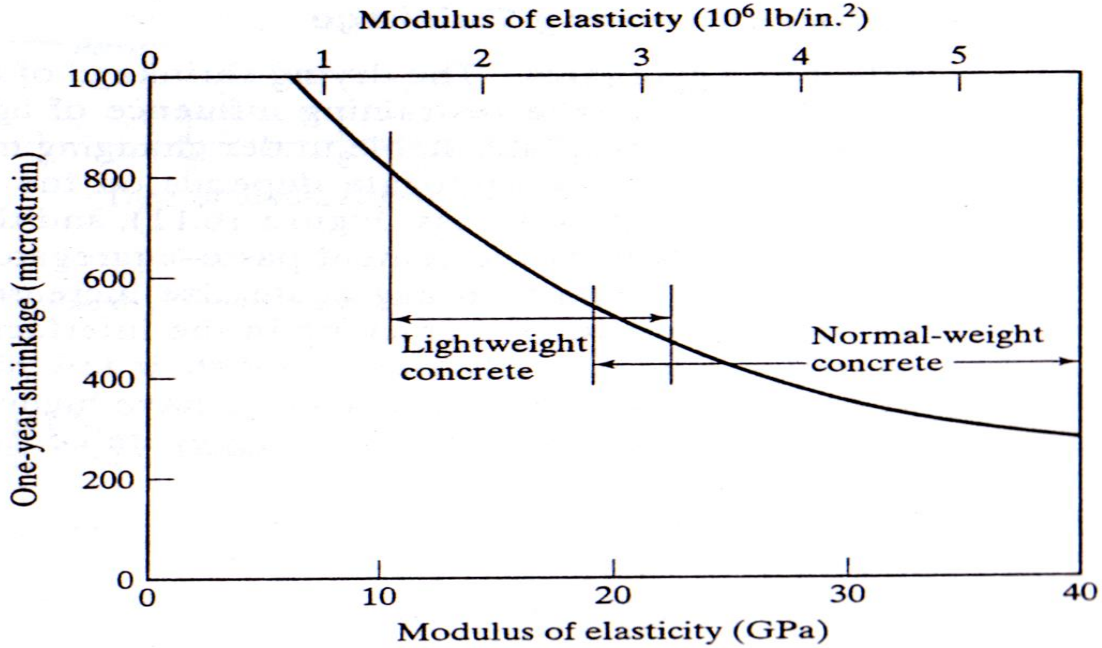
Influence of aggregate content on the drying shrinkage of concrete

Effect of Aggregate on Shrinkage-3

The stresses at the cement paste-aggregate interface due to drying shrinkage increase as the maximum aggregate size increases.

This leads to cracking in the interfacial region.

Lightweight aggregates are generally dimensionally stable, but their low modulus of elasticity means that lightweight concretes can be expected to have higher shrinkages than normal weight concretes.



Effect of stiffness of aggregate on the drying shrinkage of concrete

AUTOGENEOUS SHRINKAGE

If no additional water beyond that added during mixing is provided during curing, concrete will begin to dry internally, even if no moisture is lost to the surrounding as water is consumed by hydration. However, bulk shrinkage is only observed in concretes with a **low w/c ratio** (<0.3) and is increased by the addition of some types of **pozzolans** which makes concrete denser (e.g. silica fume).

The phenomenon is known as **self-desiccation** and is manifested as autogenous shrinkage (chemical shrinkage). Generally, the effects of self-desiccation are masked by expansion associated with the formation of ettringite or the hydration of free MgO.

CARBONATION SHRINKAGE-1

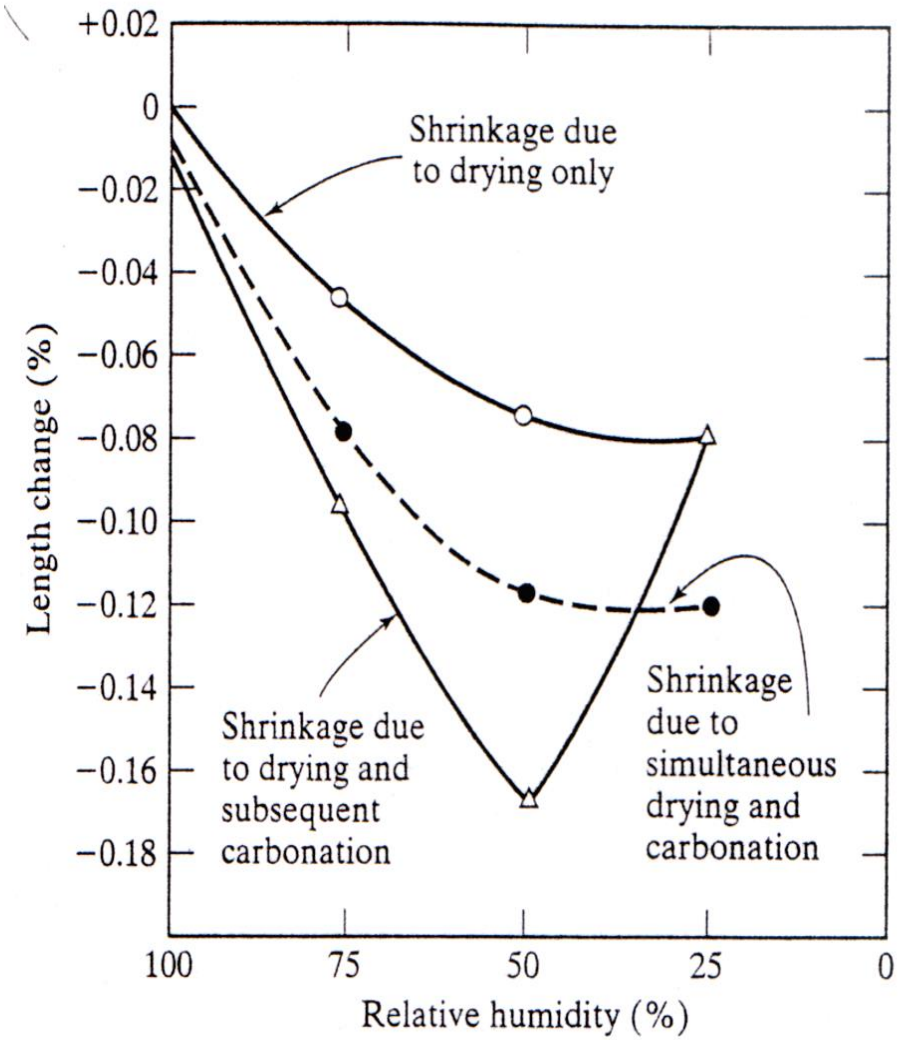
Carbonation shrinkage occurs when hydrated cement reacts with atmospheric carbon dioxide. The extent to which cement paste can react with carbon dioxide and hence undergo carbonation shrinkage is a function of **relative humidity** and is greatest around **50%** RH.

Carbonation shrinkage is fully *irreversible*.

Typical carbonation reactions are given below:

$$C-S-H+CH+\overline{C}\rightarrow C-S-H+C\overline{C}+H$$

CARBONATION SHRINKAGE-2



Shrinkage of mortar bars during carbonation

- Due to the fact that concrete pores are mostly filled with water at high humidities, CO_2 cannot penetrate into the paste very well.
- At very low humidities, an absence of water films is believed to lower the rate of carbonation.

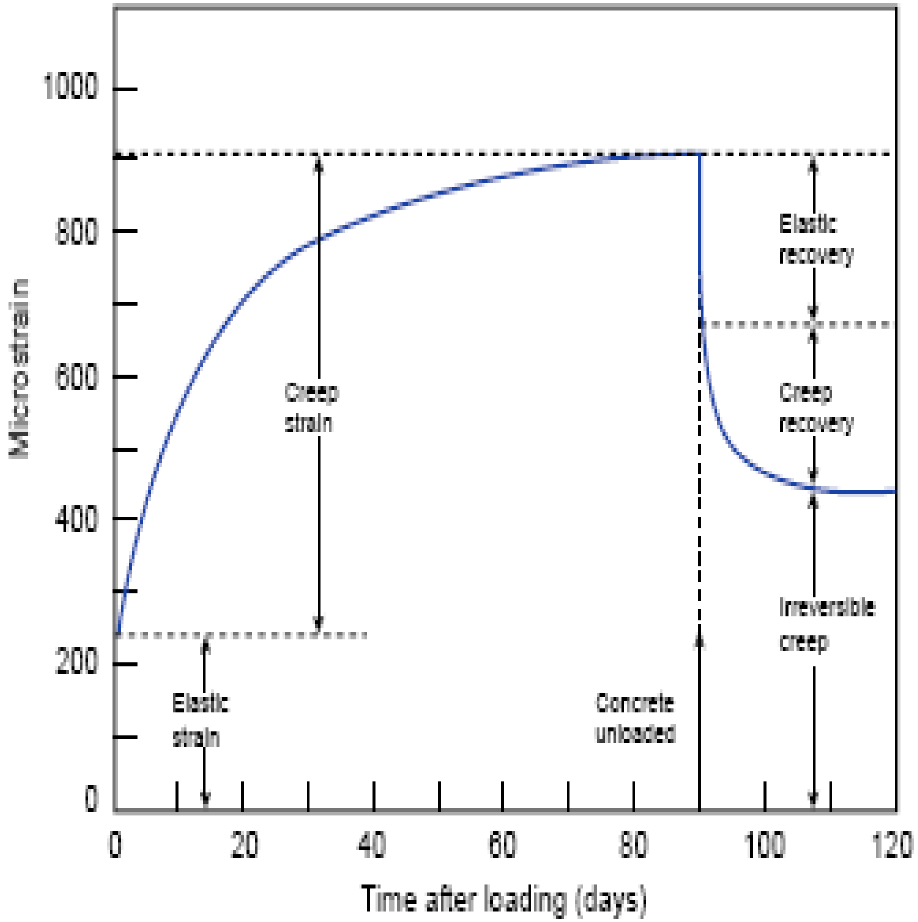
CREEP-1

Creep is the time dependent deformation under load. Unlike ceramics and metals that have negligible creep at room temperatures, concrete may show significant volume changes under load at room temperature and the presence of moisture also plays an important role.

Creep and drying shrinkage are interrelated phenomena;

- the strain-time curves are quite similar,
- the magnitude of strains is similar,
- they contain a considerable amount of irreversibility and
- like shrinkage, creep is a paste property and the aggregate in concrete behaves as a restraint.

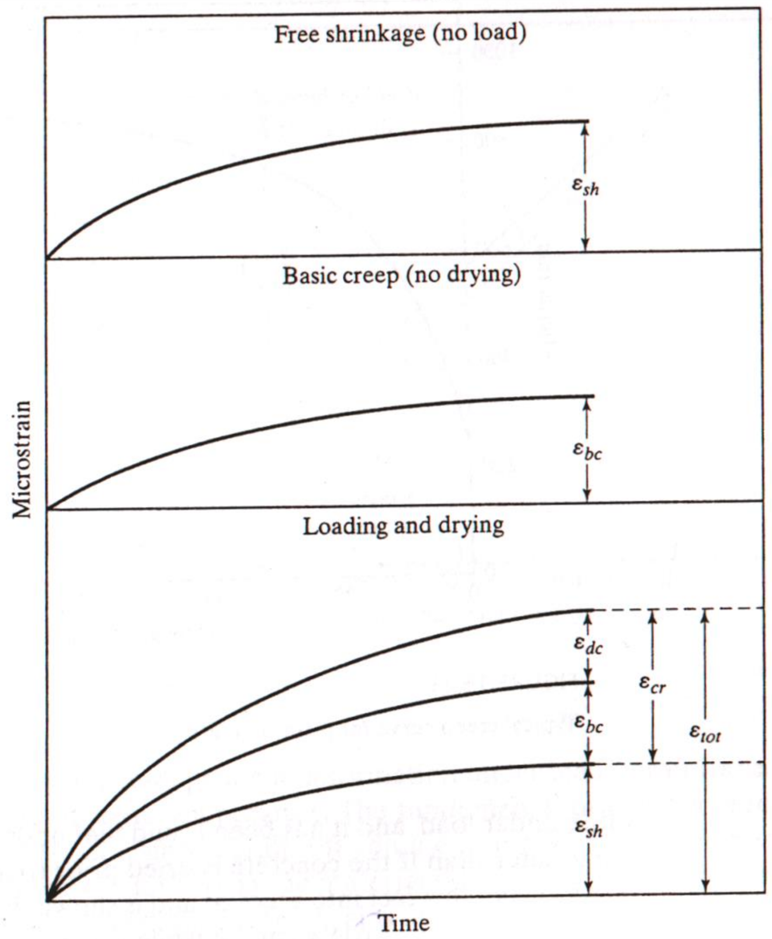
CREEP-2



Typical creep curve for plain concrete

- After unloading the specimen, the instantaneous recovery is approximately the same as the elastic strain which occurs at loading.
- Creep recovery is faster than creep strain.
- A considerable portion of total creep is irreversible.

CREEP-3



Creep of concrete under simultaneous loading and drying

- Under typical service conditions, concrete is most likely to be drying while under load, and it has been found that under such conditions creep deformations are greater than if the concrete is dried prior to loading. The excess deformation is called ***drying creep***.

Factors Affecting Creep

Water-cement ratio: specific creep (creep strain/ stress) increases with increasing water-cement ratio.

Curing conditions: increasing the period of moist curing decreases the creep.

Temperature: if concrete is maintained at elevated temperatures while under load, the amount of creep is increased over that of concrete held at room temperature.

Moisture: increasing the amount of free moisture in concrete increases creep.

Cement composition: increasing the C_3A content or decreasing the effective C_3S content seems to increase creep.

Chemical admixtures: they should be tested to evaluate their influence on creep.

Aggregates: their role on creep is similar to that in shrinkage.

CONCRETE TECHNOLOGY

LECTURE NOTES-CHAPTER 11
DURABILITY OF CONCRETE

DURABILITY OF CONCRETE

Durability of concrete is its ability to resist weathering action, chemical attack, abrasion, and other conditions throughout its service life in a structure.

The concrete in a structure must;

- ➢ have the required strength level to resist various types of loads,
- ➢ retain its original form and quality throughout its service life.

PERMEABILITY-1

Permeability is defined as the ease with which a fluid can flow through a solid. Obviously, the pore structure (**size** and **continuity** of the pores) determines the permeability of the material. For a steady-state flow, the coefficient of permeability (K) is determined from Darcy's equation:

$$\frac{dq}{dt} = K\frac{\Delta HA}{L\mu}$$

dq/dt : rate of liquid flow,
μ: viscosity of the fluid,
ΔH: pressure gradient,
A: surface area,
L: thickness of the solid.

PERMEABILITY-2

The permeability of concrete plays an important role in durability because it controls the rate of moisture ingress that contains aggressive chemicals or movement of water during heating-cooling or freezing-thawing.

Under these conditions, impermeability or water tightness becomes a primary factor in determining durability.

Permeability of hardened cement paste-1

In hardened cement paste, during the hydration process the size and continuity of the pores control the coefficient of permeability.

The amount of mixing water is responsible for the permeability of the hydrated cement paste because

i. its content determines first, the total space and

ii. subsequently the unfilled space after the water has been consumed either by hydration or evaporation.

The degree of hydration is the other important factor on the permeability of cement paste.

Permeability of hardened cement paste-2

Change in the permeability of cement paste(W/C=0.70) with increasing level of hydration

Age (days)	Coefficient of permeability (cm/sx10^{-11})
Fresh	20000000
5	4000
6	1000
8	400
13	50
24	10
ultimate	6

Permeability of aggregate-1

Compared to **30-40%** capillary porosity of hardened cement paste, the volume of the pores in most natural aggregates is usually less than **3**% and rarely exceeds **10**%. Therefore it is expected that the permeability of aggregate would be much lower than that of the typical cement paste.

The reason why some aggregates with only 10% porosity show much higher permeability than the cement paste is that the size of capillary pores in aggregate is usually much larger.

Permeability of aggregate-2

Permeability of some rocks and consequent w/c ratio of hardened cement paste having the same permeability

Type of rock	**Coefficient of permeability**	**w/c ratio of mature paste with same coeff. of permeability**
Dense trap	2.47×10^{-12}	0.38
Quartz diorite	8.24×10^{-12}	0.42
Marble	2.39×10^{-11}	0.48
Marble	5.77×10^{-10}	0.66
Granite	5.35×10^{-9}	0.70
Sandstone	1.23×10^{-8}	0.71
Granite	1.56×10^{-8}	0.72

Permeability of concrete-1

The permeability of mortar or concrete is higher than the permeability of corresponding cement paste even if low permeability aggregate particles are used in these mixtures.

The reason for this behavior lies in the **microcracks** present in the **interfacial transition zone** between aggregate and cement paste.

Permeability of concrete-2

Factors that control the permeability of concrete are quite variable. The strength and permeability are related to each other through the ***capillary porosity***, the factors which influence the strength of concrete also influence the permeability. A reduction in the volume of large capillary pores in the cement paste matrix will reduce the permeability.

Proper attention to the

- ✓ aggregate size and grading,
- ✓ thermal and drying shrinkage strains and
- ✓ premature and excessive loading are necessary steps to reduce the microcracking in the interfacial transition zone.

LEACHING AND EFFLORESCENCE-1

Leaching is the dissolving out of the CH (and various salts) which takes place in hardened concrete under the effect of percolating water.

Cement hydration leads to the production of **C-S-H gel** and **CH**. While the C-S-H is essentially insoluble, CH is not resistant to the dissolving effect of water. Any water penetrating the concrete leads to dissolving of the CH.

Soft waters (such as rain water) are more effective in increasing the solubility of the CH.

LEACHING AND EFFLORESCENCE-2

CH and other salt solutions (such as the sulfates and carbonates of sodium, potassium or calcium) evaporate at the surface of the concrete. Thus, usually a white deposit, called **efflorescence,** is formed on the surface of the concrete. Efflorescence causes an aesthetic problem. It usually does not affect the durability greatly. However, when extensive leaching takes place, the porosity of the concrete is increased and thus its strength is decreased.

Increase in porosity makes the concrete vulnerable to aggressive chemicals.

SULFATE ATTACK-1

Sulfates are often present in **ground waters** (particularly in clayey soils), in **sea water**, in **rain water** (from air pollution) and **sewage waters** (because of biological growths).

When sulfate-containing waters seep into hardened concrete:

1) **Gypsum** is formed due to the reaction of the sulfates with the calcium hydroxide in the structure of the hcp:

$$CH + N\overline{S} + 2H \rightarrow C\overline{S}H_2 + NH$$

$$CH + M\overline{S} + 2H \rightarrow C\overline{S}H_2 + MH$$

2) **Ettringite** ($C_6A\overline{S}_3H_{32}$) is formed due to the reaction of gypsum with monosulfate ($C_4A\overline{S}H_{12}$) that is present in hcp:

$$C_4A\overline{S}H_{12} + 2C\overline{S}H_2 + 16H \rightarrow C_6A\overline{S}_3H_{32}$$

The formation of ettringite in hcp or concrete leads to very large volume expansion, and generates internal stresses leading to cracking.

SULFATE ATTACK-2

Comparison of the effects of Na and Mg sulfate

Both sodium sulfate and magnesium sulfate cause destructive expansion in hardened concrete. However, **magnesium sulfate can be more aggressive than sodium sulfate** because it may lead to the decomposition of C-S-H as follows:

- $C_3S_2H_3 + 3M\overline{S}(aq) \rightarrow 3C\overline{S}H_2 + 3MH + 2SH_X$

The silica gel (SH_X) may react slowly with MH to form crystalline magnesium silicate having no cementing property.

SULFATE ATTACK-3

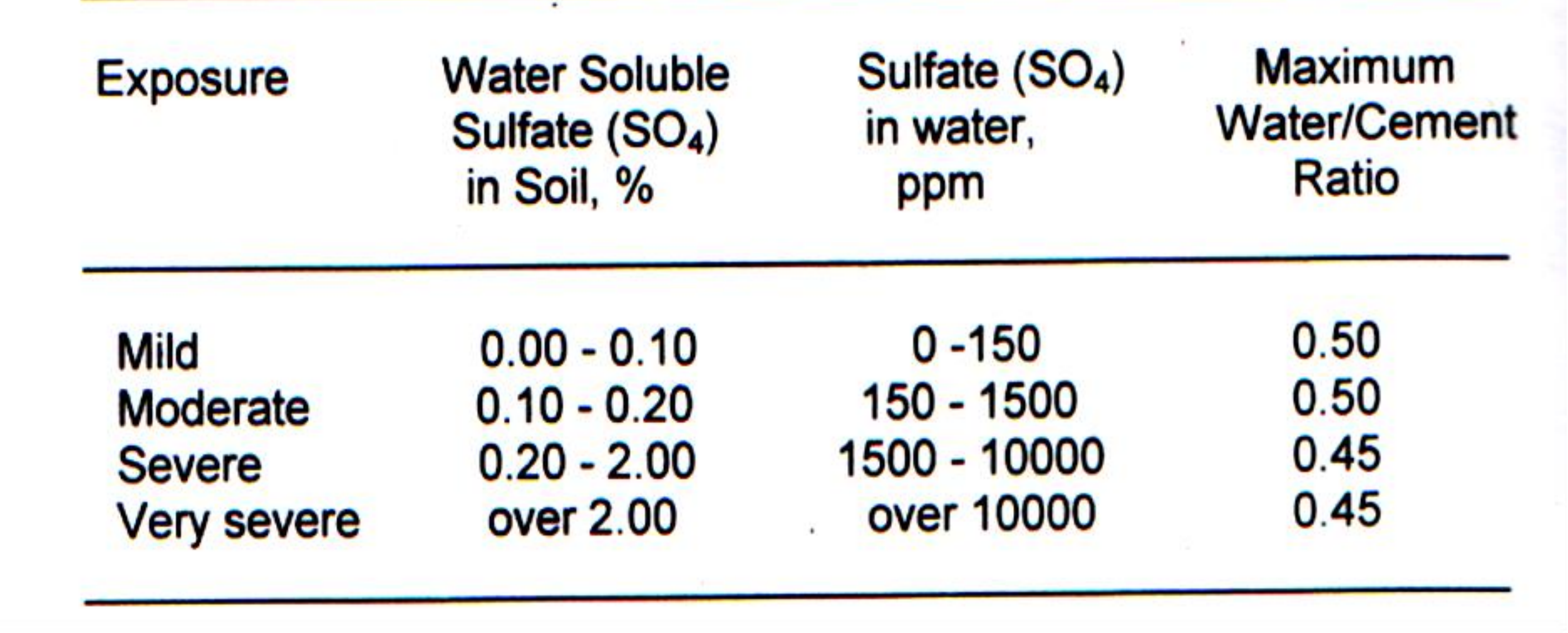
Seawater attack

- **The aggressive effect of sodium sulfate and magnesium sulfate of sea water is not so severe as the effect of those sulfates present in ground waters. Because gypsum and ettringite are more soluble in solutions containing the chloride ion, which is the major constituent of seawater. Thus, the deleterious expansion is relatively less. On the other hand, there may be some decrease in strength due to leaching.**
- **Great deterioration may take place in the intertidal zone because of frequent wetting and drying; crystallization of sea salts in the concrete on evaporation may contribute to expansive forces.**
- **The chloride ion in seawater promotes the corrosion of reinforcing steel in concrete.**

SULFATE ATTACK-4

Recommendation for concrete subject to sulfate attack

ACI recommendations for concrete subject to sulfate attack



Exposure	Water Soluble Sulfate (SO_4) in Soil, %	Sulfate (SO_4) in water, ppm	Maximum Water/Cement Ratio
Mild	0.00 - 0.10	0 -150	0.50
Moderate	0.10 - 0.20	150 - 1500	0.50
Severe	0.20 - 2.00	1500 - 10000	0.45
Very severe	over 2.00	over 10000	0.45

SULFATE ATTACK-5

Precautions against sulfate attack

1. Using a low water/cement ratio: the capillary porosity and permeability of the concrete and resultantly the penetration of sulfate-containing waters into the concrete can be lowered by decreasing W/C ratio.

2. Using the proper type of cement: ASTM Type II, Type IV, and especially Type V cements brings about the formation of relatively small amounts of gypsum due to sulfate attack, because these cements contain relatively smaller amounts of C_3S. Type V cement contains the least amount of C_3A. Therefore, it produces the least amount of ettringite.

3. The use of mineral admixtures: is beneficial because they decrease CH in hcp by pozzolanic reaction. Concretes made with such cements contain a lesser amount of CH and C_3A. Therefore less gypsum is produced when sulfate attack occurs.

ACID ATTACK

Hcp is an **alkaline** material. Thus, the presence of high concentrations of **acids** that is in contact with the hardened concrete leads to deterioration.

The acid attack takes place through the following reactions:

1.) The hydrogen ion accelerates the leaching of calcium hydroxide;

- $Ca(OH)_2 + 2H^+ \rightarrow Ca^{2+} + 2H_2O$

2.) If the concentration of the hydrogen ion is high, the C-S-H is attacked, and a silica gel having no appreciable cementing value is formed.

- $3CaO.2SiO_2.3H_2O + 6H^+ \rightarrow 3Ca^{+2} + 2(SiO_2.nH_2O) + 6H_2O$

CARBONATION-1

CO_2 is present in the atmosphere:

- about 0.03% by volume in rural air;
- 0.1% or even more in an unventilated laboratory,
- up to 0.3% in large cities.

Ground waters sometimes contain some amount of carbon dioxide in dissolved form.

When concrete is exposed to the effect of carbon dioxide, a reaction takes place as follows:

- $Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$

When carbonation occurs, concrete loses some of its **CH** and **water**. Therefore, carbonation is accompanied by shrinkage in concrete **(carbonation shrinkage).**

CARBONATION-2

Carbonation is a slow process starting on the surface of the concrete and proceeds towards the inner portions. The concrete that is within approximately 2.5-3.0 cm of the surface is under the effect of carbonation.

Since CH of the concrete present near the surface is reduced by carbonation, the alkalinity of the concrete in those carbonated sections is reduced. In this way, **carbonation makes the steel reinforcement more vulnerable to corrosion.**

REINFORCEMENT CORROSION-1

Corrosion of steel in concrete is an **electrochemical process** that requires a flow of electrons from the anodic area to the cathodic area on the steel bar.

The occurrence of the anodic and cathodic areas on the steel bar is **inevitable**. This is due to the differences in the electrochemical potentials on the surface of the bar.

The concrete in which steel bar is embedded acts as an electrolyte.

REINFORCEMENT CORROSION-2

The following reaction takes place on the anode:

- $Fe - 2e^- \rightarrow Fe^{2+}$

Ferrous ions (Fe^{2+}) at the anode pass into the solution while the negatively charged free electrons (e^-) pass along the steel into the cathode where they are absorbed and combine with water and oxygen.

- $2e^- + 1/2O_2 + H_2O \rightarrow 2(OH^-)$

Then, the (OH^-) combine with Fe^{2+} forming ferrous hydroxide, which is converted to ferric hydroxide and ferric oxide as follows:

- $Fe^{2+} + 2(OH^-) \rightarrow 2Fe(OH)_2$
- $2Fe(OH)_2 + H_2O + 1/2O_2 \rightarrow 2Fe(OH)_3$
- $2Fe(OH)_3 \rightarrow Fe_2O_3 + 3H_2O$

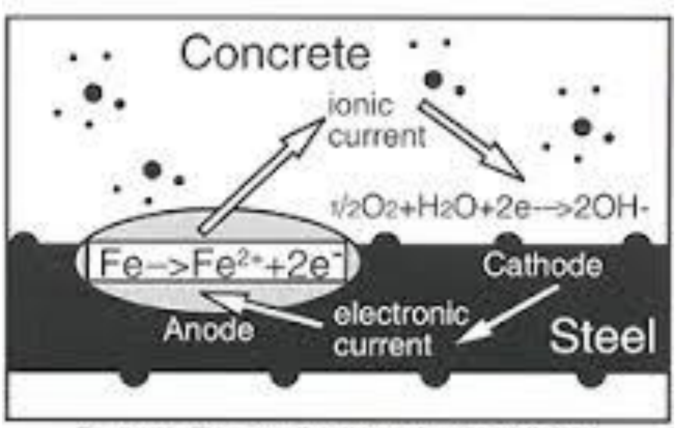
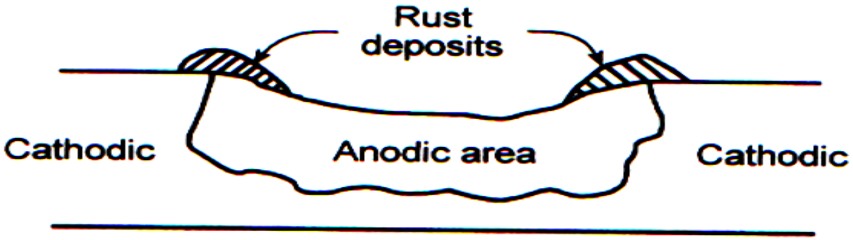


Figure 1: The anodic and cathodic reactions

REINFORCEMENT CORROSION-3

By producing Fe^{2+} and passing these into the solution, a decrease in iron content occurs at the anodic area. On the other hand, corrosion products settle on the cathodic area.



The volume of the corrosion products is **2-3** times more than the volume of metal. Thus, when corrosion products are formed on the steel bars a **great internal stress** is created within the concrete leading to **cracking** of concrete.

FREEZING-THAWING-1

The water in the capillary pores of the hardened concrete freezes below 0°C. A **9% volume increase** occurs as water turns to ice. When the capillary pores are more than 91% full of water and freezing occurs in such a condition, expansion takes place and the frozen water occupies a greater space than the volume of pores it is in.

Ice generation in the capillary pores causes a great hydraulic pressure, resulting in great internal stresses. However, when the temperature rises, the ice thaws, and the pressure is removed. The occurrence of many cycles of freezing and thawing leads to **cracking** and **deterioration** of concrete.

FREEZING-THAWING-2

Concrete that will be exposed to a moist environment and to freezing-thawing should be produced as **air-entraining concrete** by using an air-entraining admixture during the production of concrete.

By air entrainment, billions of **stable, discrete small air bubbles of spherical shape** are formed in the fresh cement paste.

The air content of the air-entrained concrete varies between 4 to 8%. Upon formation of hydraulic pressure in capillary pores during freezing, the water that is pushed out of the capillary pore gets into the air bubbles, and thus the pressure is relieved.

ABRASION

The abrasion resistance of concrete is the ability of its surface to resist being worn by rubbing and friction. Concrete floors or pavements may be abraded by **foot, vehicular traffic** and **wind** or **waterborne particles**.

- The aggregates used in concrete subjecting to abrasive forces should be hard and resistant to wear.
- An increase in water/cement ratio adversely affects the abrasion resistance of concrete. Generally, concrete that has high strength is more resistant to abrasion.
- A high amount of bleeding and improper surface finishing cause the concrete to be less resistant to abrasion.
- Sometimes an abrasion-resistance concrete surface is produced by using abrasion-resistant toppings. This is applied mostly in industrial floors.

DURABILITY OF AGGREGATES

Any lack of durability in the aggregates may lead to disastrous results in the concrete.

Physical durability is concerned with the susceptibility of aggregates to freezing-thawing, wetting-drying and wear.

Chemical durability is concerned with various forms of cement-aggregate reactions.

Physical Durability- *Soundness*

Aggregates are said to be unsound if the volume changes that accompany the environmental changes lead to disruption in concrete.

Volume changes can arise from alternate freezing and thawing or from repeated wetting and drying.

Fortunately, rocks that undergo appreciable volume change upon wetting-drying are very rare.

The effect of freezing-thawing becomes more severe when it occurs simultaneously with wetting-drying.

Physical Durability- *Soundness*

The freeze-thaw susceptibility of an aggregate depends on whether high internal stresses develop when the water inside the aggregate freezes and causes a volume increase. The stress developed is a function of ;

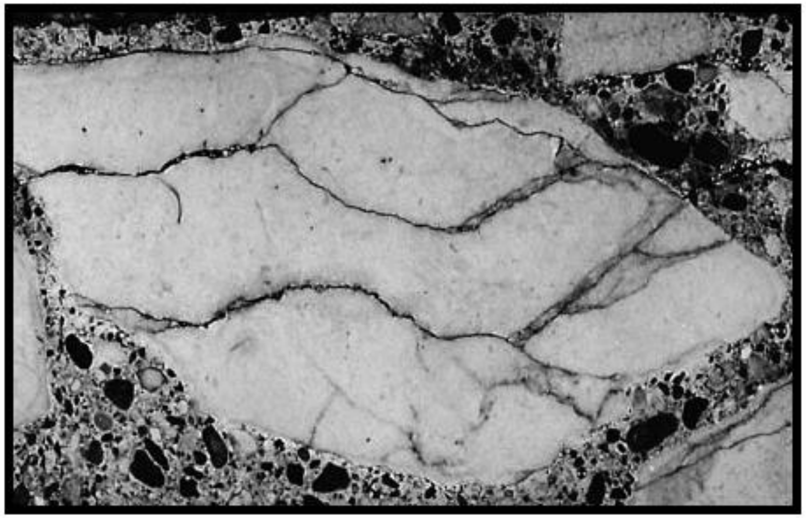
(i) porosity,
(ii) permeability,
(iii) size,
(iv) degree of saturation of the aggregate.

Friable sandstones, soft limestones, clay lumps, limestones that contain clay, and porous cherts are some examples of unsound aggregates.

Pop-outs

clarkson university course notes

D-cracking



clarkson university course notes

Soundness of aggregates is detected by treating the aggregates in sodium or magnesium sulfate solutions.

In this method, the aggregate sample is immersed in the sulfate solution for 18 hours. After being taken out of the solution, the aggregate is oven dried to a constant weight at 105±5°C. This operation is repeated for five cycles. At the end of five cycles, the loss in weight (due to breaking of aggregate pieces by the effect of sulfates) is determined.

The loss in weight should be less than

	TS 706		ASTM C33	
	Na_2SO_4	$MgSO_4$	Na_2SO_4	$MgSO_4$
CA	15%	22%	10%	15%
FA	18%	27%	12%	18%

Physical Durability- *Wear Resistance*

Abrasion is wearing by repeated rubbing or friction.

Some structures such as concrete pavements and concrete floors are subjected to abrasive forces. Therefore, the aggregates used in such structures should be resistant to abrasion.

A good aggregate plays an important role in the resistance of concrete to surface abrasion and wear.

Therefore, an aggregate must be hard, dense, strong, and free of soft, porous, and friable particles.

Abrasion resistance of aggregates is determined by "Los Angeles testing method”. Los Angeles testing machine consists of a hollow steel cylinder closed at both ends. The cylinder is mounted horizontally and can rotate around its axis. Loose steel balls are placed with oven dry aggregate in the cylinder and drum is rotated at 30-33 rpm.

The aggregate sample is considered to be resistant to abrasion when the weight loss (due to breaking) in the aggregate sample does not exceed 10% and 50% at the end of 100 and 500 revolutions of the cylinder, respectively.

Chemical Durability- *Alkali Aggregate Reaction*

- The pioneering research in 1940s correctly diagnosed the failures caused by the alkali-aggregate reaction.
- Alkali aggregate reaction (AAR) is a complex physico-chemical reaction between cement alkalies and certain reactive aggregates in concrete.
- The reaction forms a swelling gel upon intake of water, swelling of the gel results in excessive pressure and following cracking of concrete.

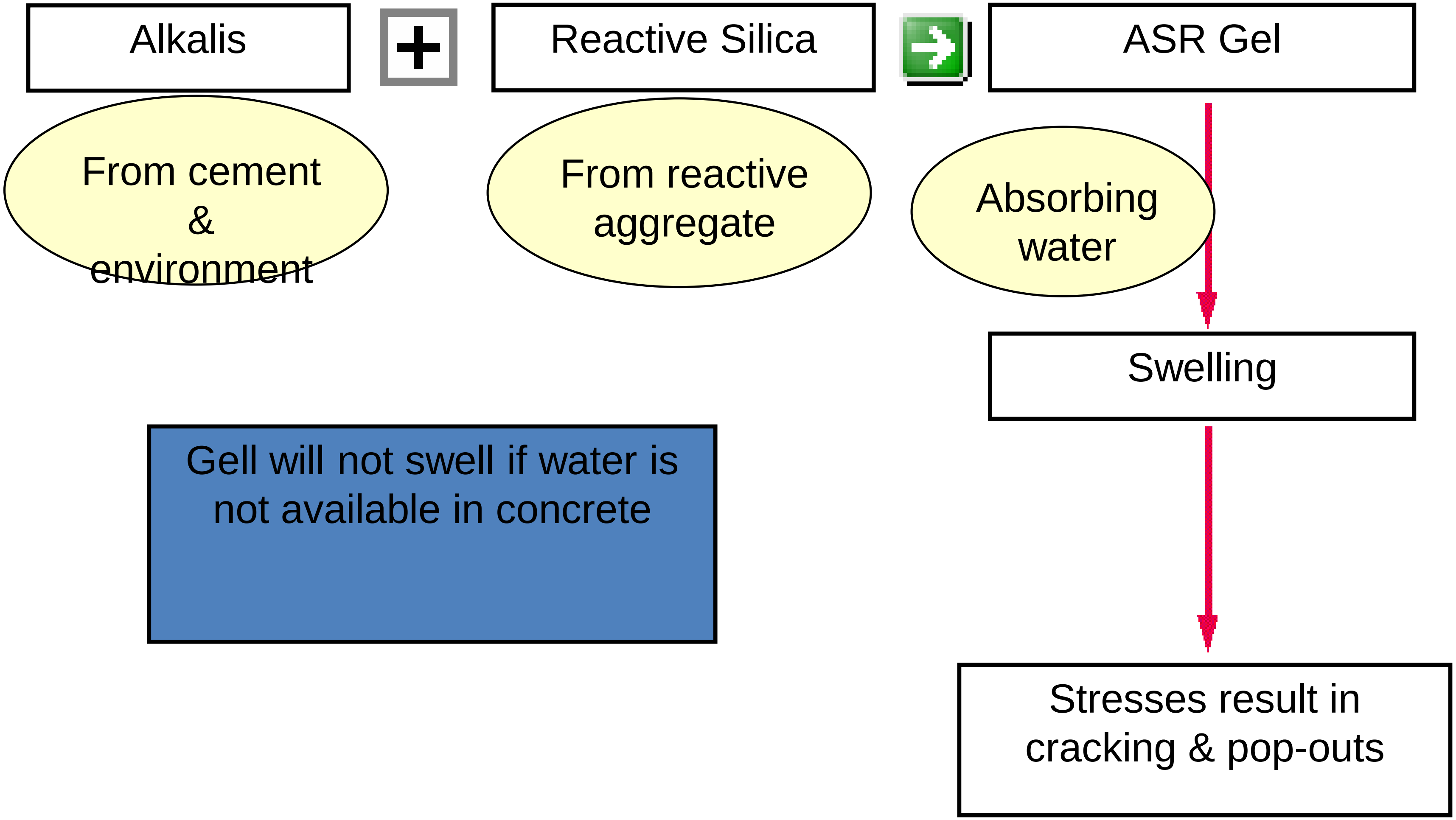
Chemical Durability- *Alkali Aggregate Reaction*

- AAR is grouped into three according to the reactive aggregate type:
- Alkali silica reaction (ASR); which was identified first, is the most rapid reaction occurring with various heterogeneously structured, porous, and sometimes hydrous silica minerals.
- Alkali-*silicate reaction or Slow/late-expanding alkali-silicate/silica reaction;* appears to occur with a delayed onset and at a slower rate and is associated with crystalline quartz-bearing rock types as the reactive constituent.
- *Alkali-carbonate reaction (ACR);* the reactive constituents are some dolomitic limestones.

Mechanism of ASR

- An alkali-silica reaction in the concrete starts with the attack of the alkalies of the cement on the reactive siliceous materials of the aggregate particles. As a result, alkali-silica gels are formed usually on the surfaces of the aggregate particles. This gel has the capacity of imbibing a great amount of water, swell and thus causing a very great expansion.
- Aggregates obtained from siliceous limestones, cherts, shales, flints, basalts, andesites, and opaline rocks may contain reactive minerals such as opal, quartz in microcrystalline form, chalcedony, tridymite or cristobalite may be reactive.

Alkali-silica reaction (ASR)



Alkalis + Reactive Silica → ASR Gel

From cement & environment

From reactive aggregate

Absorbing water

Swelling

Stresses result in cracking & pop-outs

Gell will not swell if water is not available in concrete

Alkali Silica Reaction

Factors affecting expansion;

Nature of reactive silica minerals,

Opal,
Chalcedony,
Cristobalite,
Tridymite,
Cryptocrystalline quartz

Alkali Silica Reaction

Factors affecting expansion;

Amount and size of reactive silica,

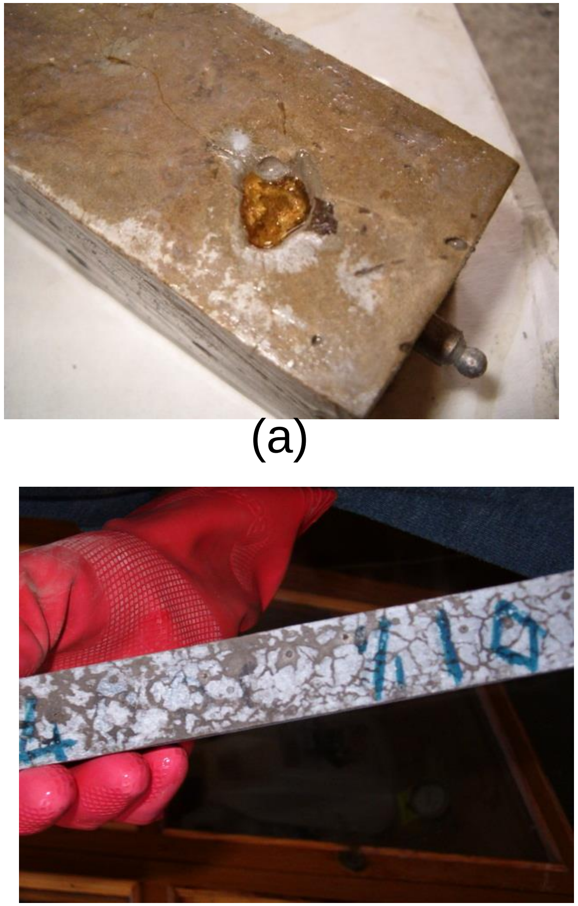
Amount of available alkali,

Experiments have shown that below 0.6% Na_2O equivalent* deleterious expansions do not occur.

**The Na_2O equivalent is given by $Na_2O+0.66K_2O$. The factor 0.66 accounts for the difference of molecular weights of Na_2O and K_2O.*

Available moisture

Airfield concrete subjected to AAR



(a)

(b)

Specimens subjected to AAR; a) pop-outs b) map-cracking

Control of Alkali Aggregate Reaction-1

(1) control of the alkalinity of concrete,

(2) control of the amount of reactive silica,

(3) control of moisture,

(4) use barium or lithium salts to inactivate the expansion of ASR gel

Control of Alkali Aggregate Reaction-2

Pozzolanic substances are commonly used to control the expansions caused by AAR. One reason suggested for the beneficial effects of pozzolans is that they reduce the alkalinity of the pore solution through the pozzolanic reactions with calcium hydroxide. Another theory proposed is that the highly reactive silica in the pozzolan rapidly reacts with the alkalies in the cement.

Alkali concentrations can be most readily controlled by using low-alkali cements with less than 0.6% Na_2O equivalent.

A low water-cement ratio may help to limit the amount of water needed for the alkali-silica gel to swell. No adverse expansion will result when external moisture is not available.

Deleterious Substances in Aggregate

Impurities: Impurities are either solid materials or soluble substances. Solid materials which are generally very fine (passing 75µm sieve) will appreciably increase the water requirement for a workable concrete if present in large amounts. They may adhere to the surfaces of larger aggregate pieces and interfere with the aggregate-matrix bond. Materials in this class are clay, silt, rock dust, and organic matter.

Organic matter may also react chemically with alkaline cement paste, forming soluble organic compounds that delay the setting time. If sea-dredged aggregates are used, they must be thoroughly washed to avoid salt contamination problems.

Unsound particles: Soft particles such as clay lumps, wood, and coal in aggregate will cause pitting and scaling at the surface of concrete. Coal may also swell in the presence of moisture or release undesirable organic compounds that interfere with setting and hardening. Weak, friable particles of low density, such as many shales and pumice should also be avoided if a good wearing surface is required. Reactive materials such as sulfide, gypsum, and cherts can also lead to serious problems.

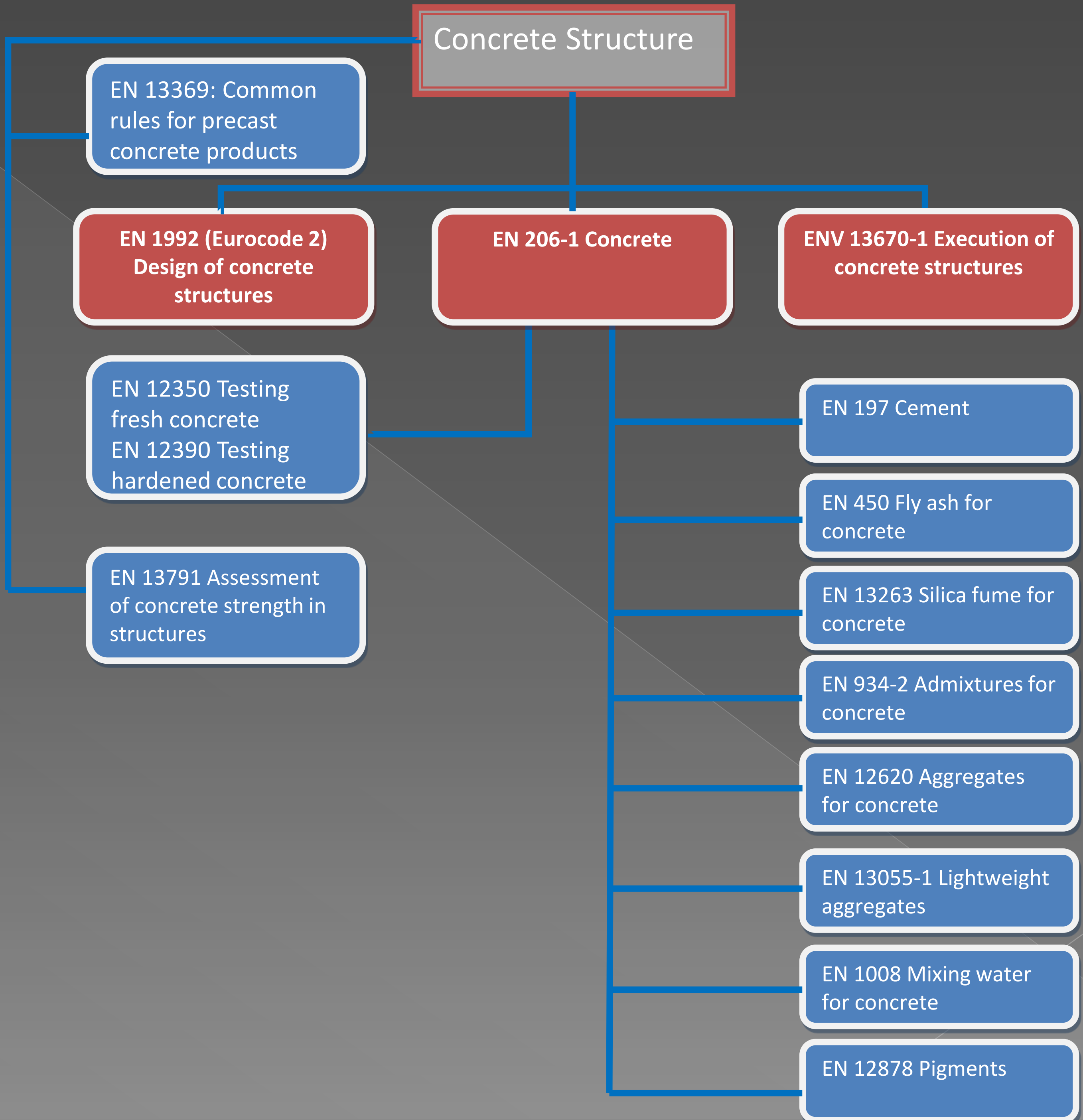
Concrete: Specification, Performance, Production and Conformity (EN 206-1)

Contents

- *Performance based approach*
- *Definitions and responsibilities*
- *Standard requirements*
- *Exposure classes*
- *Fresh concrete requirements*
- *Hardened concrete requirements*
- *Conformity criteria*

EN 206-1

- *EN 206-1 is a 'framework standard' with national provisions, detailed requirements, rules of application etc being provided by a complementary national standard (TS 13515)*
- *This standard has been accepted as a Turkish Standard (TS EN 206-1) since 08.12.2004.*



- Concrete Structure
  - EN 13369: Common rules for precast concrete products
  - EN 13791 Assessment of concrete strength in structures
  - **EN 1992 (Eurocode 2) Design of concrete structures**
  - **EN 206-1 Concrete**
    - EN 12350 Testing fresh concrete
EN 12390 Testing hardened concrete
    - EN 197 Cement
    - EN 450 Fly ash for concrete
    - EN 13263 Silica fume for concrete
    - EN 934-2 Admixtures for concrete
    - EN 12620 Aggregates for concrete
    - EN 13055-1 Lightweight aggregates
    - EN 1008 Mixing water for concrete
    - EN 12878 Pigments
  - **ENV 13670-1 Execution of concrete structures**

Performance-Based Approach

- *During the development of this standard, consideration was given to detailing a performance-based approach to the specification of durability is adopted.*
- *For this, a review of performance-related design and test methods has been undertaken.*
- *These methods are not sufficiently developed to be detailed in this standard.*
- *Therefore, performance-based local practices were allowed to be used as an alternative to prescriptive approach.*

Performance-Based Approach

- *CEN/TC 104 will continue to develop performance related methods for assessing durability at the European level.*
- *CEN/TC 154 Standardization Committee is aiming to implement new test methods in the next version of EN 206 revision in 2015.*
- *E.g., RILEM 219-ACS "Alkali Aggregate Reaction in concrete structures: performance testing and appraisal" Committee aims to develop reliable performance-based concrete tests for determination of AAR.*

Definitions and Responsibilities

- *Specifier: responsible for determination of concrete properties,*
- *Producer: responsible for conformity and production control,*
- *User (purchaser): responsible for placing the concrete in the structure.*

For ready-mixed concrete; purchaser is the specifier and has to give the specification to the producer!

This standard specifies requirements for:

- *The constituent materials of concrete,*
- *The properties of fresh and hardened concrete and their verification,*
- *The limitations for concrete composition,*
- *The specification of concrete,*
- *The delivery of fresh concrete,*
- *The production control procedures,*
- *The conformity criteria and their evaluation*

The standard applies to normal-weight, heavy-weight and light-weight concrete.

Classification

- Exposure Classes Related to Environmental Actions

- Fresh Concrete

* *Consistence classes*

* *Classes related to maximum aggregate size*

- Hardened Concrete

* *Compressive strength classes*

* *Density classes for lightweight concrete*

Exposure Classes Related to Environmental Actions

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 – No risk of corrosion or attack		
X0	For plain concrete: All exposures except where there is freeze/thaw, abrasion or chemical attack For reinforced concrete: very dry	Concrete inside buildings with very low air humidity

Class Designation	**Description of the environment**	**Informative examples where exposure classes may occur**
2 – **Corrosion induced by carbonation**		
Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified as follows		
XC 1	Dry or permenantly wet	Concrete inside buildings with low air humidity Concrete permenantly submerged in water
XC 2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC 3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC 4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2.

Class designation	**Description of the environment**	**Informative examples where exposure classes may occur**
3 – Corrosion induced by clorides other than from sea water		
Where concrete containing reinfrocement or other embedded metal is subject to contact with water containing clorides, inducing de-icing salts, from sources other than from sea water, the exposure shall be classified as follows:		
XD 1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD 2	Wet, rarely dry	Swimming pools Concrete exposed to industrial waters containing chlorides
XD 3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs

Class Designation	**Description of the environment**	**Informative examples where exposure classes may occur**
4 – Corrosion induced by chlorides from sea water		
Where concrete containing reinforcement or other embedded metal is exposed to clorides from sea water or air carrying salt originating from sea water, the exposure shall be classified as follows:		
XS 1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS 2	Permenantly submerged	Parts of marine structures
XS 3	Tidal, splash and spray zones	Parts of marine structures

Class designation	Description of the environment	Informative examples where exposure classes may occur
5 – Freeze/thaw attack with or without de-icing agents		
Where concrete is exposed to significant attack by freeze/thaw cycles whilst wet, the exposure shall be classified as follows:		
XF 1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF 2	Moderate water saturation, with de-icing salt	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF 3	High water saturation, without de-icing salt	Horizontal concrete surfaces exposed to rain and freezing
XF 4	High water saturation, with de-icing agent or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zones of marine structures exposed to freezing

Class designation	**Description of the environment**	**Informative examples where exposure classes may occur**
6 – Chemical Attack		
Where concrete is exposed to chemical attack from natural soils and ground water, the exposure shall be classified as given below.		
XA 1	Slightly aggresive chemical environment	
XA 2	Moderately aggresive chemical environment	
XA 3	Highly aggresive chemical environment.	

Others..

- *Limiting values for exposure classes for chemical attack from natural soil and ground water are given in the standard.*

Fresh Concrete

- Consistence Classes

Consistence classes are designated according to four different test methods. There is no direct relationship between these consistence classes.

1. *Slump classes*

2. *Vebe classes*

3. *Compaction classes*

4. *Flow classes*

E.g. Slump Classes

Class	Slump , mm
S 1	10 – 40
S 2	50 – 90
S 3	100 – 150
S 4	160 - 210
S 5	≥ 220

REQUIREMENTS FOR HARDENED CONCRETE

- STRENGTH
** Compressive strength : TS EN 12390 - 3*
** Tensile splitting strength : TS EN 12390 - 6*

- DENSITY : TS EN 12390-7

- Resistance to water penetration : Its method and conformity criteria shall be agreed between the specifier and producer.

- Resistance to fire : (Euro class A)

Compressive strength classes for normal-weight and heavy-weight concrete

Compressive strength class	**Minimum characteristic cylinder strength $F_{ck,cyl}$ N/mm^2**	**Minimum chracteristic cube strength $F_{ck,cube}$ N/mm^2**
C 8/10	8	10
C 12/15	12	15
C 16/20	16	20
C 20/25	20	25
C 25/30	25	30
C 30/37	30	37
C 35/45	35	45
C 40/50	40	50
C 45/55	45	55
C 50/60	50	60
C 55/67	55	67
C 60/75	60	75
C 70/85	70	85
C 80/95	80	95
C 90/105	90	105
C 100/115	100	115

Compressive strength classes for light-weight concrete

* Density classes and definition of lightweight concrete is given in the standard

Compressive strength class	Minimum characteristic cylinder strength $F_{ck,cyl}$ N/mm^2	Minimum characteristic cube strength [a] $F_{ck,cube}$ N/mm^2
LC 8/9	8	9
LC 12/13	12	13
LC 16/18	16	18
LC 20/22	20	22
LC 25/28	25	28
LC 30/33	30	33
LC 35/38	35	38
LC 40/44	40	44
LC 45/50	45	50
LC 50/55	50	55
LC 55/60	55	60
LC 60/66	60	66
LC 70/77	70	77
LC 80/88	80	88

a) Other values may be used if the relationship between these and the reference cylinder strength is established with sufficient accuracy and is documented.

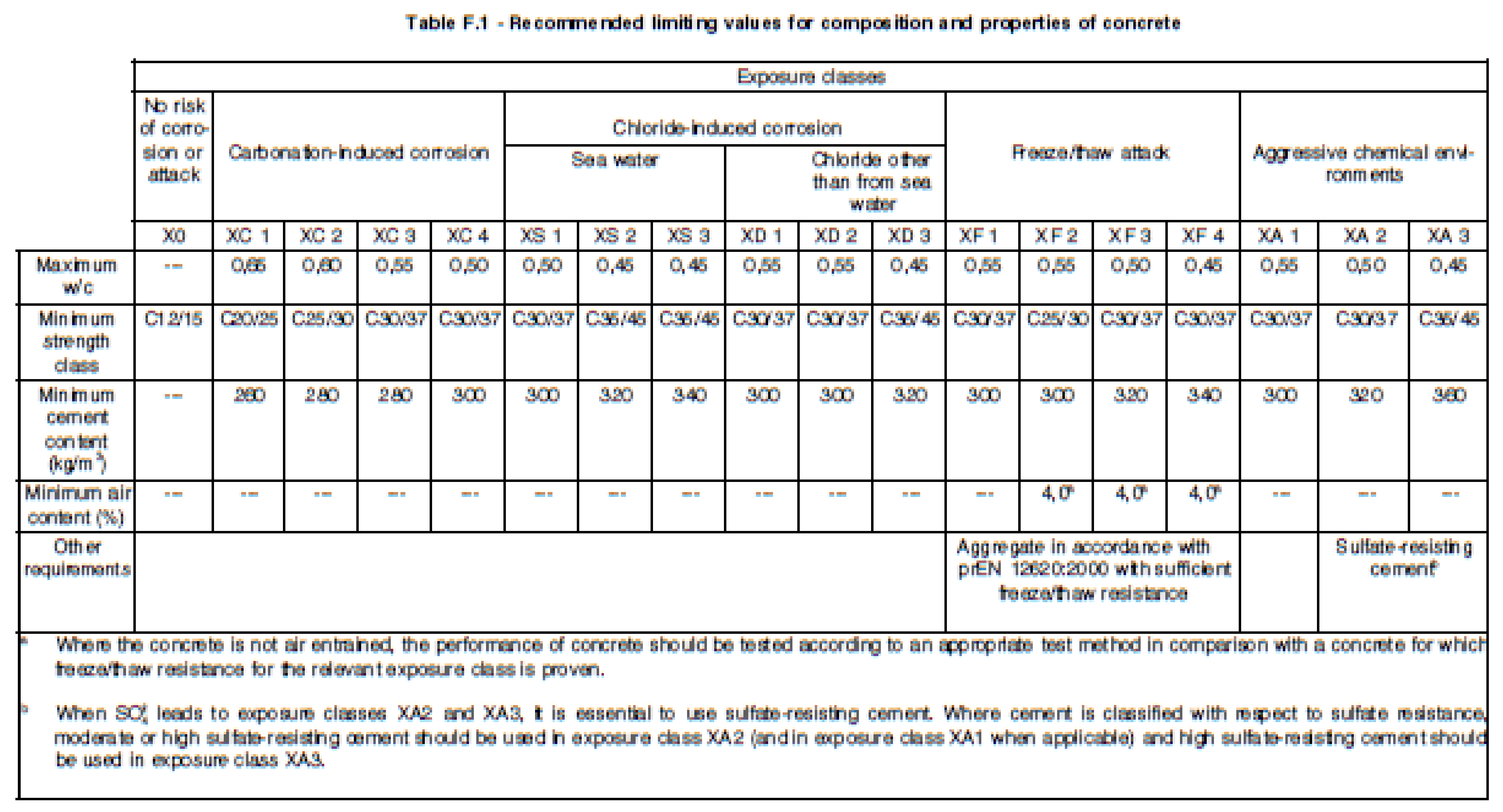


Table F.1 - Recommended limiting values for composition and properties of concrete

	Exposure classes																	
	No risk of corrosion or attack	Carbonation-induced corrosion				Chloride-induced corrosion						Freeze/thaw attack				Aggressive chemical environments		
						Sea water			Chloride other than from sea water									
	X0	XC 1	XC 2	XC 3	XC 4	XS 1	XS 2	XS 3	XD 1	XD 2	XD 3	XF 1	XF 2	XF 3	XF 4	XA 1	XA 2	XA 3
Maximum w/c	--	0,65	0,60	0,55	0,50	0,50	0,45	0,45	0,55	0,55	0,45	0,55	0,55	0,50	0,45	0,55	0,50	0,45
Minimum strength class	C12/15	C20/25	C25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
Minimum cement content (kg/m^3)	--	260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
Minimum air content (%)	--	--	--	--	--	--	--	--	--	--	--	--	4,0[a]	4,0[a]	4,0[a]	--	--	--
Other requirements												Aggregate in accordance with prEN 12620:2000 with sufficient freeze/thaw resistance					Sulfate-resisting cement[b]	

[a] Where the concrete is not air entrained, the performance of concrete should be tested according to an appropriate test method in comparison with a concrete for which freeze/thaw resistance for the relevant exposure class is proven.

[b] When SO$_4^{2-}$ leads to exposure classes XA2 and XA3, it is essential to use sulfate-resisting cement. Where cement is classified with respect to sulfate resistance, moderate or high sulfate-resisting cement should be used in exposure class XA2 (and in exposure class XA1 when applicable) and high sulfate-resisting cement should be used in exposure class XA3.

Basic requirements for constituent materials are given in the appropriate standards.

- **EN 206-1 Concrete**
  - EN 197-1 Cement
  - EN 450 Fly ash for concrete
  - EN 13263 Silica fume for concrete
  - EN 934-2 Admixtures for concrete
  - EN 12620 Aggregates for concrete
  - EN 13055-1 Lightweight aggregates
  - EN 1008 Mixing water for concrete
  - EN 12878 Pigments

Specification of Concrete

- *The specifier shall ensure that all the requirements for concrete properties are included in the specification given to the producer.*
- *The specifier shall also specify any requirements for concrete that are needed for transportation after delivery, placing, compaction, curing or further treatment.*

The specifier shall take account of;

- The application of the fresh and hardened concrete,
- The curing conditions,
- The dimensions of the structure (heat development),
- The environmental actions to which the structure is to be exposed,
- Any requirements for exposed aggregate or tooled concrete finishes,
- Any requirements related to the cover of reinforcement or minimum section width (e.g. max. nominal upper aggregate size),
- Any restrictions on the use of constituent materials with established suitability, e.g. resulting from exposure classes.

Delivery of Fresh Concrete

Information from the user to the producer:

The user shall agree with the producer the

- *Delivery date, time and rate*

And where appropriate inform the producer of:

- *Special transport on site,*
- *Special methods of placing,*
- *Limitation of delivery vehicle*

Delivery of Fresh Concrete

Information from the producer to the user (on request):

- *Type and strength class of cement and type of aggregates,*
- *Type of admixtures, type and approximate content of additions, if any,*
- *Target water/cement ratio,*
- *Results of relevant previous tests for the concrete, e.g. from production control or from initial tests,*
- *Strength development,*
- *Sources of the constituent materials.*

For the determination of curing time, information on the strength development of the concrete may be given by the producer to the user with reference to the below table:

Strength Development of Concrete at 20°C

Strength Development	**Estimate of Strength Ratio ($f_{cm,2}$ / $f_{cm,28}$)**
Rapid	$\geq$ 0.5
Medium	$\geq$ 0.3 to < 0.5
Slow	$\geq$ 0.15 to < 0.3
Very slow	< 0.15

Conformity Control

- *Conformity control criteria which interest the user will be mentioned here(indentity testing).*
- *Where identity testing is to be performed, the particular volume of concrete shall be defined, e.g.:*
  - *Single batch or load where there is doubt as to the quality,*
  - *The concrete supplied for each storey of a building or groups of beams/slabs or columns/walls of a storey of a building or comparable parts of other structures,*
  - *The concrete delivered to a site within one week, but not more than 400m3.*

Identity Criteria for Compressive Strength

Concrete under Production Control Certification

Identity of concrete is assessed for each individual strength test result and the average of “n” overlapping discrete results as identified in below table.

Concrete is deemed to come from a conforming population if both criteria in below table are satistifed for “n” results derived from strength tests on samples taken from the defined volume of concrete.

Identity Criteria for Compressive Strength

Number “n” of test results for compressive strength from the defined volume of concrete	Criterion 1	Criterion 2
	Mean of “n” results (f_{cm}) N/mm^2	Any individual test result (f_{ci}) N/mm^2
1	Not applicable	$\geq f_{ck}$ -4
2 – 4	$\geq f_{ck}+1$	$\geq f_{ck}$ –4
5 - 6	$\geq f_{ck}+2$	$\geq f_{ck}$ –4

Concrete not under production control certification

From the defined volume of concrete, at least 3 samples shall be taken for the testing. The concrete deemed to come from a conforming population if the conformity criteria below are established.

Criterion 1 : $f_{cm} \geq f_{ck}+4\ (N/mm^2)$

Criterion 2 : $f_{ci} \geq f_{ck}-4\ (N/mm^2)$

f_{cm} : Mean of test results

f_{ci} : Any individual test result

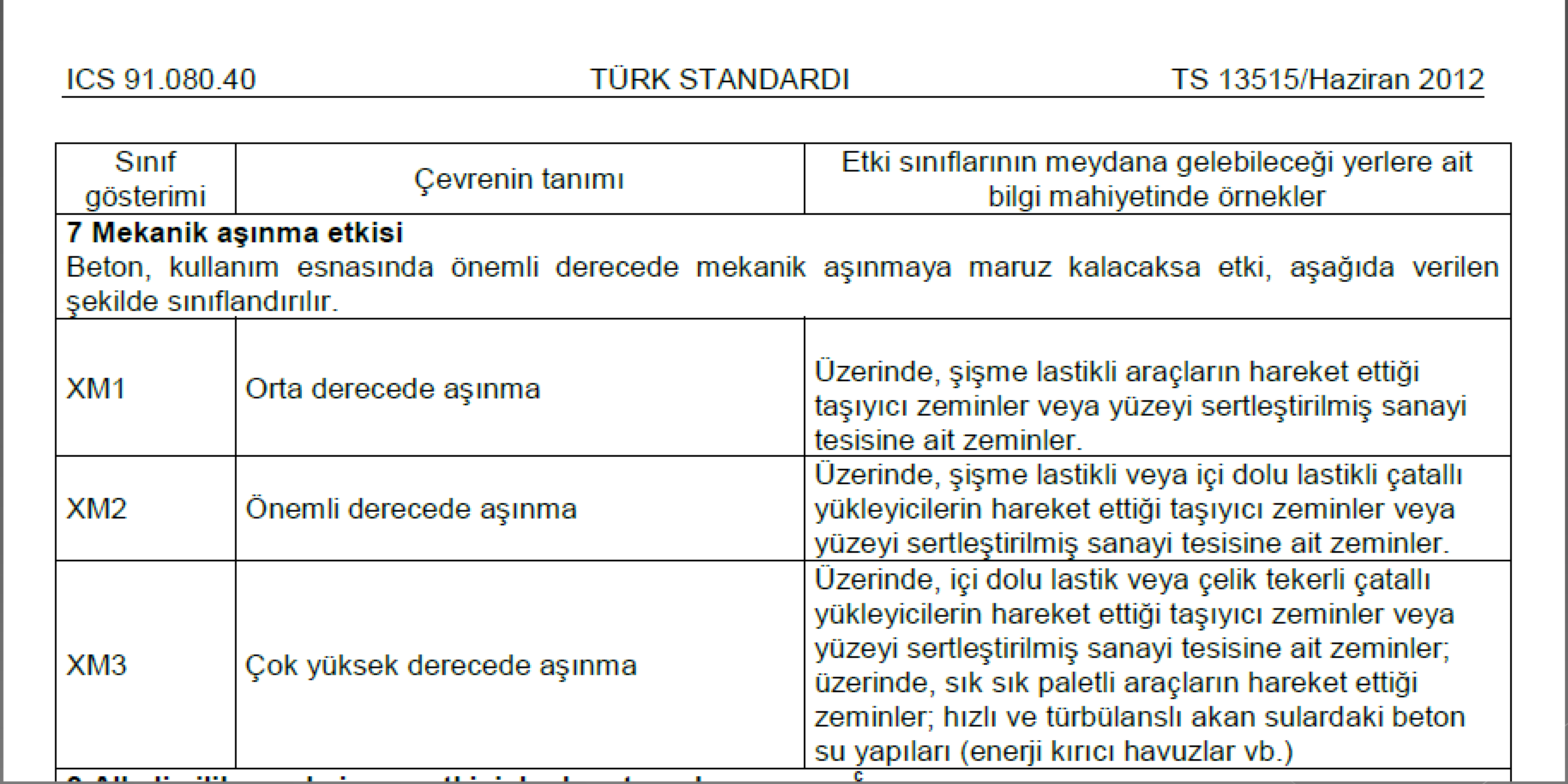
Conformity Criteria other than Strength

Conformity Criteria are defined for the following properties of concrete other than compressive and tensile strength:

-Density,
-Water/cement ratio,
-Cement content,
-Air content of air-entrained fresh concrete,
-Chloride content of concrete,
-Consistence

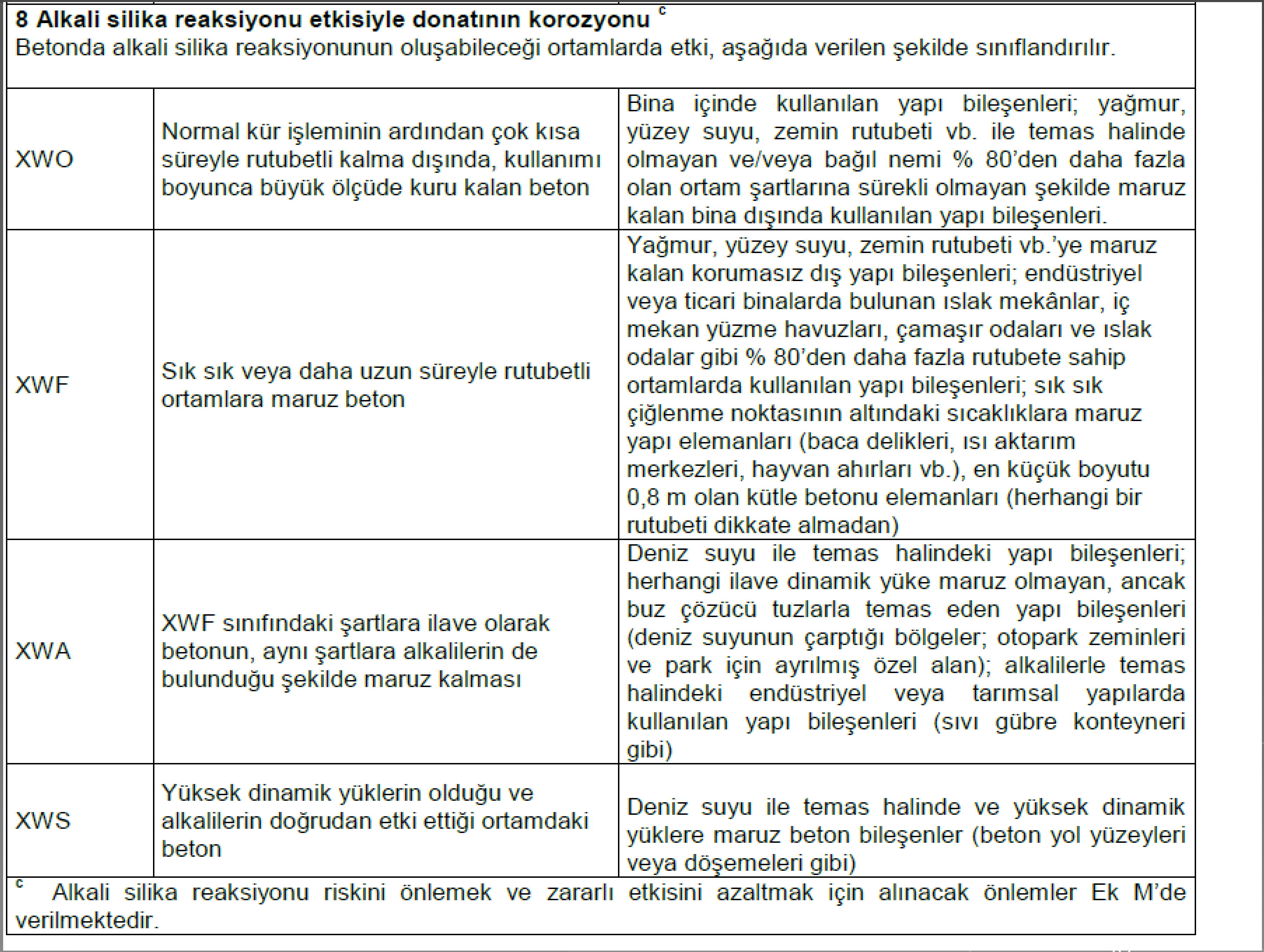
TS 13515

Complementary Turkish standard to TS EN 206-1



ICS 91.080.40 TÜRK STANDARDI TS 13515/Haziran 2012

Sınıf gösterimi	Çevrenin tanımı	Etki sınıflarının meydana gelebileceği yerlere ait bilgi mahiyetinde örnekler
7 Mekanik aşınma etkisi Beton, kullanım esnasında önemli derecede mekanik aşınmaya maruz kalacaksa etki, aşağıda verilen şekilde sınıflandırılır.		
XM1	Orta derecede aşınma	Üzerinde, şişme lastikli araçların hareket ettiği taşıyıcı zeminler veya yüzeyi sertleştirilmiş sanayi tesisine ait zeminler.
XM2	Önemli derecede aşınma	Üzerinde, şişme lastikli veya içi dolu lastikli çatallı yükleyicilerin hareket ettiği taşıyıcı zeminler veya yüzeyi sertleştirilmiş sanayi tesisine ait zeminler.
XM3	Çok yüksek derecede aşınma	Üzerinde, içi dolu lastik veya çelik tekerli çatallı yükleyicilerin hareket ettiği taşıyıcı zeminler veya yüzeyi sertleştirilmiş sanayi tesisine ait zeminler; üzerinde, sık sık paletli araçların hareket ettiği zeminler; hızlı ve türbülanslı akan sulardaki beton su yapıları (enerji kırıcı havuzlar vb.)



8 Alkali silika reaksiyonu etkisiyle donatının korozyonu [c]
Betonda alkali silika reaksiyonunun oluşabileceği ortamlarda etki, aşağıda verilen şekilde sınıflandırılır.

XWO	Normal kür işleminin ardından çok kısa süreyle rutubetli kalma dışında, kullanımı boyunca büyük ölçüde kuru kalan beton	Bina içinde kullanılan yapı bileşenleri; yağmur, yüzey suyu, zemin rutubeti vb. ile temas halinde olmayan ve/veya bağıl nemi % 80’den daha fazla olan ortam şartlarına sürekli olmayan şekilde maruz kalan bina dışında kullanılan yapı bileşenleri.
XWF	Sık sık veya daha uzun süreyle rutubetli ortamlara maruz beton	Yağmur, yüzey suyu, zemin rutubeti vb.’ye maruz kalan korumasız dış yapı bileşenleri; endüstriyel veya ticari binalarda bulunan ıslak mekânlar, iç mekan yüzme havuzları, çamaşır odaları ve ıslak odalar gibi % 80’den daha fazla rutubete sahip ortamlarda kullanılan yapı bileşenleri; sık sık çiğlenme noktasının altındaki sıcaklıklara maruz yapı elemanları (baca delikleri, ısı aktarım merkezleri, hayvan ahırları vb.), en küçük boyutu 0,8 m olan kütle betonu elemanları (herhangi bir rutubeti dikkate almadan)
XWA	XWF sınıfındaki şartlara ilave olarak betonun, aynı şartlara alkalilerin de bulunduğu şekilde maruz kalması	Deniz suyu ile temas halindeki yapı bileşenleri; herhangi ilave dinamik yüke maruz olmayan, ancak buz çözücü tuzlarla temas eden yapı bileşenleri (deniz suyunun çarptığı bölgeler; otopark zeminleri ve park için ayrılmış özel alan); alkalilerle temas halindeki endüstriyel veya tarımsal yapılarda kullanılan yapı bileşenleri (sıvı gübre konteyneri gibi)
XWS	Yüksek dinamik yüklerin olduğu ve alkalilerin doğrudan etki ettiği ortamdaki beton	Deniz suyu ile temas halinde ve yüksek dinamik yüklere maruz beton bileşenler (beton yol yüzeyleri veya döşemeleri gibi)

[c] Alkali silika reaksiyonu riskini önlemek ve zararlı etkisini azaltmak için alınacak önlemler Ek M’de verilmektedir.