HIGH-STRENGTH CONCRETE USING LOCAL MATERIALS

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Notations

A, Agg:	Aggregate
A/C:	Aggregate/Cement, ratio by weight
A/C+SF:	Aggregate/ Cement+Silica Fume, ratio by weight
B.R.U.W:	Bulk Rodded Unit Weight of Coarse Aggregate (oven dry basis)
C:	Cement
C.A:	Coarse Aggregate
F.A:	Fine Aggregate
.d:	Days
fc:	Average Compressive Strength
f'c:	Average Compressive strength at the Age of 28-days
f_r:	Modulus of Rupture
f_t:	Splitting Tensile Strength
HRWRA:	High-Range-Water-Reducer-Admixtures (Superplasticizer)
HSC:	High-Strength-Concrete (f'c > 42 MPa)
ITZ:	Interfacial Transition Zone (zone of transition from coarse aggregate to cement paste or the zone between coarse aggregate and cement paste).
MAS:	Maximum Aggregate Size of coarse aggregate used in concrete mix design
MCP:	ACI Manual of Concrete Practice
MPT:	Maximum Paste Thickness (mean distance between two adjacent coarse aggregates)
NSC:	Normal strength Concrete (f'c < 42 MPa)
OPC:	Ordinary Portland Cement (Type I Portland Cement)
SF:	Silica Fume
S.G:	Specific Gravity
SSD:	Saturated Surface Dry (one of aggregate state of moisture)
SQRT	Square Root
Vs:	Versus
W/C:	Water/Cement, ratio by weight
W/(C+SF):	Water/(Cement+silica Fume), ratio by weight

Abstract

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A Comprehensive experimental program has been completed, to investigate the possibility of producing HSC locally.

Significant high-strength values have been obtained, with high-range of workability using HRWRA, furthermore, a study was carried on the effects of aggregate/cementitious Materials ratio (A/C), Maximum Aggregate Size (MAS), & SF% level as a partial replacement for OPC, on the mechanical properties (compressive strength, splitting tensile strength, flexure tensile strength, & rate of gaining strength with time) of HSC.

Also the change of source type of aggregate has been applied using Basalt & Wadi aggregates one at time for each, which supports the research into studying the effect of shape and surface texture of aggregate on mechanical properties of HSC irrespective of the strength of aggregate itself, which shows a significant effect on tensile strength.

Basalt aggregate mixes show a significantly higher strength than those of Wadi aggregate mixes and for all mechanical properties studied.

The highest 28, 56-days respectively standard cylinder compressive strength of 77.4, 80.4 MPa have been obtained for Basalt using Silica-Fume at W/(C+SF) of 0.38.

28 and 56-days standard cylinder compressive strength of 94.2, 94.8 MPa have been obtained for Basalt using Silica-Fume and low MAS (9.5mm) at W/(C+SF) of 0.23.

1- Introduction

1.1 General Definitions:

High-Strength concrete (HSC) is defined as concrete that has a specified compressive strength f'c of 42 MPa or greater (ACI 211.4R-93, 1993). Two arguments are advanced to justify this definition of High-Strength concrete.

1. To produce concrete above 42 MPa (specified compressive strength), more stringent quality control and more care in the selection and proportioning of materials (Chemical admixtures, mineral admixtures e.g. Fly ash or silica fume, type and quality of aggregates, etc.), are needed. Thus, to distinguish this specially formulated concrete, which has a specified compressive strength above 42 MPa, it is called High-Strength (Kumar and Monterio, 1993).

2. Experimental studies shows that the micro- structure, and properties of concrete with specified compressive strength above 42 MPa are considerably different from those of conventional concrete.
   Since the latter is the basis of current concrete design practice (e.g., the empirical equation for estimating the elastic modulus from compressive strength), the designer should be alerted if concrete of higher than 42 MPa specified-compressive strength is treated as separate class (Kumar and Monterio, 1993).

1.2 Theory:

Strength of concrete is commonly considered its most valuable property, although in many practical cases other characteristics, such as durability and impermeability, may be more important which are usually associated with high strength.

However strength usually gives an overall picture of the quality of concrete, as strength is directly related to the structure of the hardened cement paste.

For a given cementitious materials and acceptable aggregates, the strength that may be developed by a workable, properly placed mixture of cement, aggregate, and water the same mixing, curing, and testing conditions is mainly influenced by the following factors (Neville and Brooks, 1990):

(1). Ratio by weight of mixing water to cementatious materials (W/C).
(2). Ratio by weight of aggregate to cementatious materials (A/C).
(3). Grading, surface texture, shape, strength of aggregate particles.
(4). Maximum size of aggregate.

For high-strength concrete we are considering, concrete of high strength which achieved by proper proportioning of MAS or use of mineral or chemical admixtures and not by steam curing or application of pressure or any special technique.
So we have to optimize the latter factors in determining the strength of concrete (to a highest possible value of strength).

1.3 Main guidelines for optimizing the factors which affect compressive strength of concrete

1.3.1 Water/Cementitious materials ratio (W/C):

W/C ratio affects the capillary porosity of the hardened cement paste at any stage of hydration. So for properly compacted concrete the W/C affects the volume of voids in concrete. Whenever W/C increases, total volume of voids increase also leading to a lower gel/space ratio, lower hydration products, and leads finally in results of lower strength concrete mix, that makes it clear why compacted concrete which have a very low porosity(low W/C ratio), have a higher strength.

Strictly speaking, the volume of voids in concrete, entrapped air influences strength of concrete, capillary pores, gel space, and entrained air if present.

So it is obvious that at proper workability lowering W/C ratio as much as possible is considered essential to produce a higher strength mix.

1.3.2 Aggregate/Cementitious materials ratio (A/C):

At fixed W/C a leaner mix (higher Aggregate/cementitious materials ratio), leads to a higher strength. This behavior is associated with that the total water content per cubic meter of concrete is already lower.in leaner mix so, the effect of W/C ratio being thus reduced (Neville and Brooks, 1990). Also the strength of interfacial transition zone (ITZ) between aggregate and cement paste depends on its maximum thickness(mean distance between two adjacent coarse aggregates), the lower this distance, the higher the transition zone strength (de Larrad and Belloc, 1997).

In other words it is important to remember that aggregates used for HSC mixes should be properly stronger than cementitious materials itself so there is an optimum ratio where cementitious materials plays a role of binding without being the limiting factor itself which is the best situation.

As a result in a leaner mix the voids form a smaller fraction of the total volume of concrete so higher strength in concrete mix.

1.3.3 Aggregates Selection and Properties:

For HSC, it is common that the strength of aggregates become a limiting factor that should be considered since they occupy the largest volume of any ingredient in the concrete.

1. Selection of Coarse Aggregates:

The coarse aggregate will influence significantly the strength and structural properties of the concrete. For this reason, coarse aggregates should be chosen that are sufficiently hard, free of fissures or weak planes, also aggregates particles should be clean and free of surface coatings all that could be achieved by:

a- crushing the aggregate particles
b- washing the aggregate particles.

Coarse aggregates with such properties exhibit better bonding to the cementatious matrix, thereby contribution to higher strength.

2. Selection of Fine Aggregate:

The grading and particle shapes of fine aggregates are significant factors in the production of HSC. Particle shape and surface texture can have as great an effect on the amount of mixing water and which in turn, reflects on compressive strength of concrete. Fine aggregate with same grading but with a difference of 1% in voids content may result in a 1 gal per yd^3 difference in water demand (ACI 211.4R-93, 1993).

The quantity of paste (cement+fines) required per unit volume of a concrete mixture decreases as the relative volume of coarse aggregate versus fine aggregate increases - lower surface area to be covered - (ACI 211.4R-93, 1993).

But because the amounts of cementitious materials in HSC mixes are large, the volume of fines tends to be high. Consequently the volume of sand can be kept to the minimum necessary to achieve workability and compatibility.

In this manner, it will be possible to produce higher strength concrete for a given cementitious materials content.

Fine aggregates with Fineness modulus (FM) in the range of 2.5 to 3.2 are preferable for HSC. Concrete mixtures made with a fine aggregate that has a FM of less than 2.5 may be sticky and result in poor workability and a higher water demand (ACI 211.4R-93, 1993).

3. Grading of aggregates:

The main factors governing the desired aggregate grading are:

a- The surface area of the aggregate which determines the amount of water necessary to wet all the solids.
b- The relative volume occupied by the aggregates.
c- The workability of the mix, and the tendency to segregation.

So at fixed W/C and segregation controlled, if the grading extends to a larger maximum aggregate size the overall specific surface is reduced and the water requirement decreases, also a lesser cement paste to cover the surface, and those leads to a higher strength.

4. Shape and surface texture of aggregate:

The effect of shape and surface texture are particularly significant, in the case of high-strength concrete. The flexural strength is more affected than compressive strength (Neville and Brooks, 1990).

Irregular and angular shapes provide a stronger bonding than rounded and flaky shapes (Cetin and Carrasquillo, 1998).

***A rougher texture results in a greater adhesive force between the particles and cement matrix. Likewise the large surface area of angular aggregate means that a larger adhesive force can be developed* (Neville and Brooks, 1990)**

5. Strength of aggregate:

Since at least three-quarters of the volume of concrete is occupied by aggregate it is clear that the compressive strength of concrete cannot exceed that of major part of the aggregate contained.

Also the strength potential of rock, irrespective of the grading and shape of particles significantly affects strength of concrete (de Larrad and Belloc, 1997). Aggregate stiffer than the matrix has been reported to increase the elastic modulus (Cetin and Carrasquillo, 1998).

The Los Angeles test combines the processes of attrition and abrasion, and gives results which shows a good correlation not only with the actual wear of the aggregate in concrete, but also with the compressive and flexure strengths of concrete when made with the same aggregate (Neville and Brooks, 1990)

Hence Los Angeles Abrasion test will be adopted to distinguish and express the strength potential of aggregate used in research.

In general, the strength and elasticity of aggregate depends on its composition, texture and structure and it is significantly affects the production of high-strength concrete mixes.

6. Maximum aggregate size (MAS):

It has been mentioned before that the larger the aggregate particle the smaller area to be wetted per unit weight. Thus extending the grading to a larger maximum size lowers the water requirement of the mix, so that for a specified workability and richness, the W/C can be lowered with a consequent increase in strength. This behavior has been verified by tests with aggregates up to 38mm maximum size and normal concrete mixes. But however, that above 38mm maximum size, and/or high cement contents the gain of strength due to reduced water requirement is offset by detrimental effects of lower bond area and of discontinuities introduced by the very large particles, particularly in rich mixes.

But again sizes below 38mm for normal concrete mixes (19mm in high strength concrete mixes), the decrease in water requirement is dominant (Neville and Brooks, 1990)

As a result:

1- In HSC mixes there are high amounts of cementitious materials so, lowering the maximum aggregate size (MAS) (limited to 19 mm maximum) leads to decrease maximum paste thickness (MPT) which proved to get higher strength (de Larrad and Belloc, 1997).

2- In general, the smallest size of aggregate produces the highest strength for a given W/C in the case of HSC (ACI 211.4R-93, 1993).

The use of the largest possible coarse aggregate is an important consideration if optimization of modulus of elasticity, creep, and drying shrinkage are important (ACI 211.4R-93, 1993).

1.4 Nature of Problem and General Considerations

Where workability is adequate, it appears that the W/C ratio holds the key to the porosity of both the hydrated cement paste and transition zone, (between cement paste and aggregate). Furthermore, with a low W/C ratio it is generally observed that considerably high strength is achieved for very small decrease in W/C ratio.

But however for the production of high-strength concrete, the opposing effect of W/C ratio on the consistency and strength of concrete cannot be harmonized without the use of water-reducing admixtures. This explains why the developments of plasticizing admixtures have helped to increase the production and use of high-strength concrete.

With normal-weight conventional concrete containing strong aggregate of 25 or 38mm maximum size and W/C ratios in the range of 0.4 to 0.7, it is generally the transition zone that is the weakest component of the system.

So at a given W/C ratio, the strength of concrete mixture can be increased significantly by simply reducing the maximum size of coarse aggregate particles in *rich mixes* because this has beneficial effect on the strength of transition zone.

Therefore, in proportioning high-strength concrete mixtures it is customary to limit the maximum size of aggregate to 19mm or lower.

The requirement of low W/C ratio and small aggregate size mean that the cement content of the concrete mixtures will be high, generally above (385 kg/m^3).

With the increasing proportion of cement in concrete a strength plateau is reached, that is, there will be no more increase in strength with further increase in cement content. This is probably due to inhomogeneity of the hydrated Portland cement paste, in which the presence of large crystals of calcium hydroxide represents weak areas of cleavage under stress (Kumar and Monterio, 1993).
Such inhomogeneous and weak areas in the transition zone are vulnerable to microcracking even before the application of an external load. This happen as a result of thermal-shrinkage or drying-shrinkage stresses when considerable

differences may arise between the elastic response of the cement paste and that of the aggregate, and this explains also why leaner mixes is more strong than rich mixes up to optimum value. It should be noted that increase in cement content also means increase in cost, heat of hydration, and drying shrinkage of concrete (Kumar and Monterio, 1993).

When the inhomogeneity of the hydrated Portland cement paste becomes strength limiting in concrete, the obvious solution is to modify the microstructure so that the components of inhomogeneity are eliminated or reduced.

In the case of Portland cement products, an effective but expensive way to achieve this is by *incorporation of pozzolanic admixtures such as fly ash or silica fume*, which have three main effects that are equally important for the enhancement of strength of the system which are:

1. first effect appears from the basic A.S.T.M. specification(618-72) description for pozzolana as a siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but will in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties similar to those into principal hydration products (ASTM Standards, 1993).

2. Also the pozzolanic reaction is accompanied by a reduction in large pores, then more and more high strength could be gained -micro-filler effect- (Khayat *et al.*, 1997), (Kuennen, 1996), (Well, 1998).

3. An advantage of the use of pozzolana in high-strength concrete is that relatively less heat of hydration is evolved per unit strength, therefore the risk of thermal cracking is reduced (ACI 211.4R-93, 1993).

1.5 Silica Fume as Superior Pozzolana:

1.5.1 What is Silica Fume (SF)? :

Silica fume is a by-product of the manufacture of silicon metals and ferro-silicon metals. It is created by heating quartz, coal and iron and wood chips to approximately 1800 degrees C and collecting the tiny particulate present in the emissions from this combustion process (ACI 234R-96, 1996).

Silica fume is an extremely fine powder having particles with an average diameter of about 0.10 microns and a bulk density of 150 to 250 kg/m^3.

The particles are perfectly spherical in shape, a consequence of their origin (condensation-vapor). Silica fume varies in color from white to pale gray to black with a specific surface area in the order of 20,000 m^2/kg. In relative terms, silica fume is about 100 times finer than cement particles, or about the same fineness as cigarette smoke.

Other terms for silica fume include: condensed silica fume, microsilica, ferrosilicon dust, silica flue dust, and amorphous silica. Some of the end uses of silica fume include: cement and concrete applications, refractories, fertilizer, rubber and plastic filler, oilwell cementing, and cosmetics.

Silica fume creates two major effects, which improve the properties of cement paste, which leads to better concrete:

1.Microfiller effect
2.Pozzolanic effect

1.5.2 Microfiller effect of Silica Fume:

Silica fume particles are 100 times smaller than cement grains and are, therefore easily introduced into the spaces between the cement grains, thus reducing the space available for water.
This leads to dense packing of silica fume particles between the cement particles minimizing porosity.
This phenomena also leads to improve packing at the cement-aggregate interface, which increase bond between the mortar and aggregates.

1.5.3 Pozzolanic Effect of Silica Fume:

As the particles are amorphous silica (+85% $SiO2$) with an extremely high surface area, they react chemically with the calcium hydroxide from the cement to form calcium silicate hydrates or CSH. CSH is a hydration product found in hardened cement paste.
Increased CSH leads to higher strength and a pore-blocking effect reduces the permeability of concrete (ACI 234R-96, 1996).

1.5.4 Concrete Benefits From Silica Fume:

Increase Compressive Strength

Silica fume addition can be used to produce concrete with a significantly higher compressive strength than normal concrete.
Silica fume combined with superplasticizer and low W/C ratio produce concrete in the range of 60-150 MPa (Goldman and Benture, 1989), (Khayat *et al.*, 1997), (Kuennen, 1996), (Kumar and Monterio, 1993).

The contribution of SF as a pozzolanic and a micro-filler material resulted into improved aggregate matrix bond, which is associated with the formation of a less porous transition zone, which leads to a true composite material effect in this system, thus leading to a higher strength in concrete.

Reduce Concrete Permeability

With silica fume it is possible to reduce the permeability of the cement paste considerably as well as the porosity of the transition zone between the paste and the aggregates.
These improvements together with a reduction in internal bleeding are primarily due to the pozzolanic and filler effects of the silica fume (Khayat *et al.*, 1997), (Kuennen, 1996), (Well, 1998).

Improve Freeze/Thaw Durability

Experiments show that silica fume concrete has very good freeze/thaw durability when they have a satisfactory air-void system (Khayat *et al.*, 1997).

Improve Sulfate Resistance

The chemical reaction between the silica fume and calcium hydroxide from the cement produces secondary CSH, which densifies the paste. This chemical reaction is believed to be the principal reason for sulfate resistance (Well, 1996).

Reducing Chloride Permeability

Because it reduces chloride penetrability, silica fume is gaining popularity as a corrosion-protection system for parking garages, bridges, decks, and other structure.

That is pozzolanic feature in silica fume causes reaction with the excess calcium hydroxide [$Ca(OH)_2$]. This process results in a far less permeable micro-structure matrix that appears homogenous and that has no gaps and no large crystals of [$Ca(OH)_2$].

So in addition silica fume also increases abrasion and erosion resistance, increase resistance to aggressive chemicals acid attack (Khayat *et al.*, 1997), (Well, 1998).

Reduce Concrete Bleeding

Because the paste is considerably denser there are fewer capillary paths for the mix water to migrate to the surface in the form of bleed water (Goldman and Benture, 1989), (Kuennen, 1996).

- 1982 Demonstration slabs of 138 MPa concrete are placed at site of Dashields Lock & Dam on Ohio River in Pennsylvania (Kuennen, 1996).

- 1982 First use of shotcrete containing silica fume, attaining 90 MPa, at Lake Lynn Testing Faculty, southwestern Pennsylvania, Bureau of Mines (Kuennen, 1996).

- 1983 In two important applications, both involving abrasion resistance concrete, U.S. Army Corps of Engineers specifies silica fume for Kinsua Dam Stilling Basin Rehabilitation, Warren, Pa (70 MPa in seven days, and 86 MPa in 28-days), and Los Angeles River Low-Flow Channel liner (Kuennen, 1996).

- 1984 First U.S. bridge deck using silica fume is placed on Oct 18 in northeastern Ohio by Chapin & Chapin Norwalk, Ohio.
  First use of silica fume specified for commercial building in U.S. Nashville's 28-story Third National Bark and Finance Center. Concrete supplied by Metro Ready Mix (Kuennen, 1996).

- 1985 First use of silica fume in a parking garage, for repair work in Home's Garage, Pittsburgh, using 10 percent by weight of cementatious materials. Concrete was provided by Frank Bryan Inc. Southside. Pa (Kuennen, 1996).

- 1986 First use of silica fume in a new parking deck, the Manor Parking Structure in Pittsburgh, featuring cast in-place slabs, post tensioned two ways, and precast architectural parapets.
  Scotia Plaza is constructed in Toronto with a composite concrete /steel system, using 70 MPa silica fume mixes (Kuennen, 1996).

- 1988 Two early ultra-high-strength concrete Seattle towers are constructed, 97 MPa silica fume concrete is specified for 62-story Two Union Square, but at times reaches compressive strength of 131 MPa, nominal 97 MPa silica fume concrete specified for the 45-story Pacific First Center (Kuennen, 1996).

- 1989 High-strength 62 to 70 MPa prestressed girders using silica fume are incorporated into Perdio Pass marine structure, Gulf Shores (Kuennen, 1996).

- 1989-1992 Silica fume concrete is specified for reinforced concrete columns or composite concrete/steel structures in a wave of high-rise buildings that close 1980s. These includes Chicago's 311 south Wacker Drive (84 MPa), Atlanta's One Peachtree Center (84 MPa), Minneapolis' Dain Bosworth Tower (97 MPa), Apple Valley Minn Cleveland's Society Tower (84 MPa), and New Yourk City's Trump Palace [84 MPa] (Kuennen, 1996).

- 1996 All states have used silica fume at least once in bridge decks (Kuennen, 1996).

However, it is a must here to introduce some main alerts for engineers or contractors when dealing with HSC especially those which have silica fume or any type of pozzolana which are:

1. Structurally whenever compressive strength of concrete increases the ductility of hardened concrete decreases for both tension and compression, which significantly affects the behavior of reinforced concrete elements against sudden failure.
   So high-strength concrete should be tested for minimum permissible limits of ductility (Xie *et al.*, 1995).

2. Chemically it took time and moisture needed immediately to allow silica fume to have a very strong pozzolanic reaction (till the cement grains hydrate and generate [$Ca(OH)_2$] . So silica fume will react with [$Ca(OH)_2$] and create more calcium silicate hydrates (CSH) so that; care should be taken in dosing silica fume especially when high-early strength gain required. Retarding of gaining strength may happen, especially when silica fume partially replacing the binder and not added (ACI 211.4R-93, 1993).

3. At low doses of 5% or less, silica fume actually serve to liquefy the concrete, because silica fume particles are so fine, and fit in between the

binder grains, that displace water, which becomes free to help with the flowability of the concrete (it becomes its own water reducer).

But when more and more silica fume added, the surface area of the silica fume begins to outweigh its water displacement function, and surface forces begins to have a strong effect, and water reducer, or superpalsticizer must be added to overcome the need for more water (Kuennen, 1996).

4. As the dosage of silica fume increased, the bleed water that comes to the surface is reduced. If the surface is not protected, plastic shrinkage cracks may appear (Goldman and Benture, 1989).

5. It is important from health and safety point view to realize that care should be taken while work with micro-silicate materials, which are fine powder, it should be not inhaled or allowed to come into contact with eyes or skin. Appropriate measures should be taken to prevent the generation of airborne dust and its inhalation.

5. In economic point view the use of micro-silicate into cement blend gives a plus cost over cement itself alone.

1.6 Significance of Research

High strength-concrete (HSC) provides a better solution for reducing sizes and weights of concrete structural elements, particularly for long-span beams. However this reduction in cross sectional elements which reflects on the moment of inertia, I, of the member necessitates the investigation of many factors e.g. deflection under service load.

Use of high-strength concrete leads also to larger usable floor area (from reducing columns sizes), and leads to reducing in overall building height and dead loads, resulting from the use of thinner slabs and shallower beams. This helps in taller buildings and bridges with longer spans.

Generally this type of concrete has a high elastic modulus, so more flexure stiffness, also it has a low porosity, and fewer microcracks those which leads primarily to low permeability, so high-strength concrete exhibit excellent durability to various physical and chemical agents that are normally responsible for concrete deterioration (Kumar and Monterio, 1993).

Also for those latter extra behaviors of high-strength concrete in addition to many structural studies, which found that most of reinforced concrete elements behavior enhanced by use of high-strength concrete, it some times becomes called *High-Performance concrete* (Kumar and Monterio, 1993).

1.6.1 Enhancements in reinforced concrete elements behavior provided by using High-strength concrete

1. For any type/grade of concrete the material properties (E_c,C_t) where E_c is the modulus of elasticity of concrete under short-term loading, and C_t is the creep coefficient for plain concrete at time t, have appreciable effect on flexural rigidities of reinforced concrete members under sustained load. An improvement in the determination of these properties will, therefore, lead to a better estimation of long-term deformations (Ashour *et al.*, 1997), (Mahmood *et al.*, 1995).

2. With the advent of high-strength steel section (with yield strength up to 600 MPa), and high strength concrete (with compressive strength up to 100 MPa), the high-strength composite column offer significant economic benefit (Kilpatrik and Vijaya, 1999).

3. Very high-strength concrete produced with silica fume shows higher normalized ultimate strength values under triaxial loading conditions than very-high strength concrete produced without silica fume. This observation is explained as due to the increased shear strength of material obtained with the pozzolanic additives (Setunge *et al.*, 1993).

4. The torsional strength of deep beams increases with the increase of concrete compressive strength (Wafa *et al.*, 1995).

5. The influence of compression steel reinforcement on time-dependent deflections noted to be less significant for HSC, while addition of steel fibers improves the modulus of rupture and splitting strength of HSC (Ashour *et al.*, 1997)

1.7 Objective and Scope of Research

The main objective of research is to find out that HSC could be produced using local materials. Produced safely and into controllable manner by recommending sort of technique to be as cornerstone for mix design resulting from test mixes, recording and studying the effects of mix proportions on the properties of concrete.

The interference of workability will be eliminated (all mixtures will have acceptable or desirable workability by using high-range water reducer admixture HRWRA or well known as superplacticizer), other mix design factors will be kept constant including water/binder ratio for most of mixes, except the following factors varied one at time.

1. A/C ratio (Aggregate/Cement ratio)
2. Aggregates source type (Basalt, Wadi).
3. Maximum aggregate size (MAS).
4. Level of Silica Fume (taken as partial replacement by weight of Cement).

Moreover, the following properties are going to be studied,

1. Properties of aggregate.
2. Compressive strength.
3. Splitting tensile Strength.
4. Flexure tensile Strength.

Finally a regression analysis of the results will be introduced to analyze the effects and Interaction of different properties, also mix design proportions will be presented for each test.

1.8 Methodology

An experimental program will be set up in order to study the following items:

(1). Properties of aggregates:

a. Specific gravity, Absorption, & Bulk rodded unit weight of aggregates (fine & coarse) according to ASTM C127-84, C29-78.
b. Abrasion resistance of coarse aggregate according to ASTM C131-69 & C535-69.
c. Trial mixes of aggregates (fine and coarse), will be tested to achieve a standard grading curve (extended for all sizes, and of 19,12.5, & 9.5mm maximum size) according to ASTM C33.

(2). Compressive strength of concrete:

Standard 150*300 mm test cylinders will be used for testing compression strength at 3,7,14,28, and 56 days according to ASTM C39-72, C192-69, C470-76 under various conditions as following:

The tests will be done using the two different source types of aggregate (Basalt, Wadi) one at time. But volume of coarse aggregate maintained the same to establish a comparison between the two types of materials in their effect on strength of given volume of concrete, and for each test there were a varied Aggregate/Binder ratios, and quantity of silica fume partially replaced the binder by weight.

(3).Splitting tensile strength of concrete:

Standard 150*300 test cylinders will be used to test splitting tensile strength at 28 days for different mixes types mentioned previously. According to ASTM C496-71.

(4). Flexure tensile strength of concrete:

Standard test prisms of 100*100*500 mm will be tested for each mix to determine the modulus of rupture at 28-days adopting two points loading approach according to ASTM C78.

(5). The mix of highest value of compressive strength will be retested for compressive, tensile, and flexure strength, but using lower than 19 mm maximum aggregate size (MAS) and for both aggregate source types.

(6). The mix of highest value of compressive strength will be retested, in compression, tensile, and flexure strength but lowering W/C, as much as possible, using 15% SF partial replacement, and low MAS to get a very high-strength concrete mix, and for both source type of aggregates.

Finally a regression analysis of results will be processed, conclusions and recommendations will be established.

2- Experimental Setup

2.1Experimental Preparations:

The experimental program begins with suiting and collection of proper required materials, which are aggregate, binder, water, chemical, and mineral admixtures.

2.1.1 Aggregate

A very important feature, which distinguishes HSC production from other types of concrete, is the careful choosing of aggregate (suiting of aggregate to fit the required properties along with low W/C ratio).

Two main source types of aggregate were chosen which are:

A- Crushed igneous rock aggregate and sand (Basalt) from Al-Azraq area.

B- Wadi deposit derived aggregate and sand from Jordan Valley area.

Figures 2.1, 2.2, 2.3, & 2.4, shows how the aggregate grading as delivered from source. Note that the grading of aggregate coarse and fine for both sources weren't complying with ASTM C-33 standards. Which makes it necessary to make all aggregates complying with the requirements of ASTM C-33 standards for grading of coarse and fine aggregate.
Therefore the following steps were done:

1. Basaltic aggregate were separated into four main batches that are:

   i. Batch #1B which has a 19mm maximum nominal size, and a minimum nominal size of 14mm (Figure 2.5).

   ii. Batch #2B which has a 14mm maximum nominal size, and a minimum nominal size of 9.5mm (Figure 2.6).

iii. Batch #3B which has a 9.5mm maximum nominal size, and minimum nominal size of 4.75mm (Figure 2.7).

vi. Batch #4B which passes sieve #4 according to ASTM sand standards (Figure 2.8).

2. Wadi aggregate prepared as three main batches as it delivered that are:

i. Batch #1W which has a 19mm maximum nominal size (Figure 2.9).

ii. Batch #2W which has a 12.5mm maximum nominal size (Figure 2.10).

iii. Batch #3W which is sand (Figure 2.4).

3. Then a several aggregate mix trials have been done to maintain a compatible grading with ASTM C-33, which presented in table 2.1 as final proportions:

Table 2.1 Final Coarse Aggregate Proportions

Batch No.	1B	2B	3B	1W	2W
Basalt C.A (Figure 2.11)	1	1	1		
Wadi C.A (Figure 2.12)				1	1

4. Fine aggregates were checked to have a compatible grading according to ASTM C-33, and added in accordance with method of ACI 211.4R-93 of mix design. The percentage of fine aggregate to whole aggregate ranged between 34% to 48% along research.

Therefore the aggregate used graded according to specifications to enhance workability and to avoid the problems of honeycomb or segregation as specifications cleared.

2.1.2 Testing of Aggregate

To insure a good quality of aggregate and control mix design proportions the aggregates have been tested for specific gravity and absorption for both coarse and fine aggregates according to ASTM C127 & C128, resistance to abrasion of coarse aggregates by use of Los Angeles Machine according to ASTM C131, bulk rodded unit weight of coarse aggregates according to ASTM C29 .

Test results are tabulated in Table 2.2 below:

Table 2.2 Aggregate Tests results

Type of Agg.	C.A Basalt	F.A Basalt	C.A Wadi	F.A Wadi
S.G Dry	2.769	2.857	2.577	2.563
S.G SSD	2.81	2.91	2.624	2.493
Absorption %	1.46	1.84	1.83	1.47
B.R.U.W	1485 kg/m³		1552 kg/m³	
Abrasion value loss/wt	20 %		26.8 %	
F.M		2.57		2.74

- S.G : Specific Gravity, B.R.U.W : Bulk Rodded Unit Weight, F.M : Fineness Modulus
  C.A : Coarse Aggregate, F.A : Fine Aggregate, wt : Weight.

- The aggregate batches were piled into dry state yard of lab under normal Jordan summer atmospheric conditions and covered with sheets, the aggregate stored into dry state until the date of mixing for all period of mixing of this research.

2.1.3 Binder

The choice of binder for HSC is a very important issue, unless high-early strength concrete is required, however using of super - plasticizers becomes one of HSC features because of low W/C ratio which help into accelerating strength gaining what ever is the type of binder (ACI 211.4R-93, 1993).

Type I Ordinary Portland Cement (OPC) is a general purpose and suitable for all uses where the special properties of other types are not required.

A freshly produced Ordinary Portland Cement type I according to (J.S.S 30/1981) were used and stored under lab conditions away from moisture or direct sunlight.

2.1.4 Chemical Admixtures

In the production of HSC, decreasing the W/C ratio by decreasing the water requirement rather than by increasing the total cementitious materials content, will usually produce higher compressive strength, but however it will be incorporate with workability and practical problems which may adversely affect the strength, for this reason, use of *high-range water reducers* (**HRWR/** or well known as **Superplasticizer**) as chemical admixture should be considered when producing HSC (ACI 212.3R and ASTM C494).

The use of chemical admixtures may improve and control the rate of hardening and slump loss, and result in accelerated strength gain, better durability, and improve workability. HRWR helps in dispersing cement particles, and they can reduce mixing water requirements by up to 30 %, thereby increasing concrete compressive strength.

The HRWR used in this research has the following properties after manufacture information:

1. Name & Trade Mark:

   Daracem 205 is a registered trademark of W.R. Grace & Co-Conn. Emirates Chemical LLC Dubai, U.A.E.

2. Advantages:

   - ❖ High compressive strength concrete especially at early ages.
   - ❖ Low W/C ratio, excellent durability.
   - ❖ Good surface finish, provide highly aesthetic concrete appearance.
   - ❖ Plastic concrete exhibits high cohesion, fluidity and flowability.

3. Typical Properties:

- Dark brown liquid appearance.
- 1.2 Specific gravity at 20C°.
- Air entertainment (Nil).
- Chloride content (Nil).

4. Compatibility with other cementatious materials:

- Compatible with all Portland, pozzolanic, and blast furnace cements.
- It also compatible with concrete containing fly ash and or silica fume.

5. Method of use:

To be added to concrete mixes either during the mixing cycle or at the same time as the water.

6. Addition rates:

Normal dosage 0.8 – 3 % by weight of cement. The optimum dosage should be assessed after preliminary trials depending upon the mix constituents.

7. Confirmation with specifications:

Daracem 205 conforms to requirements of ASTM C494 Type A and F, BS 5075, part 3, and ASTM C1017.

2.1.5 Mineral Admixtures (Silica Fume)

The superiority of silica fume as pozzolanic admixture have been thoroughly mentioned in sec 1.5 chapter 1 of this research. Here are the properties of silica fume used in this research after manufacture information:

1. Name & trade mark:

Cico silica fume is a trade registered for Al-Taher Group Company S.A.

2. Description:

Cico silica fume based on a chloride free pozzolanic material consisting of over 90 % silicon dioxide. Its addition to the cement mix will yield a concrete specially able to cope with middle eastern environment.

3. Advantages:

- Compatible with OPC.
- Increase compressive & flexure strength.
- Decrease bleeding and segregation.
- Increase abrasion resistance.
- Imparts resistance to passage of aggressive ions such as chlorides, sulfates, nitrates and other salts.
- High-early strength development.

4. General properties and compositions summarized in table 2.3 :

Table 2.3 General Properties of Silica Fume used in research

Appearance	Fine powder
Specific gravity	2.2
Bulk density, kg/m^3	300-600
Surface area, m^2/kg	18,000 – 22,000
Chloride content	Nil
BS 5075	
Ignition loss, %	< 3 %
Flammability	Non- flammable
Typical analysis	
SiO_2	> 90 %
Fe_2O_3	< 1.5 %
Al_2O_3	< 1 %

MgO	< 1 %
CaO	< 1 %
Na_2O	< 0.5 %
K_2O	< 0.5 %
SO_3	< 0.5 %
C	< 0.5 %

Table 2.3 Continue

5. Standard compliance:

Norwegian standard NS 3045, unidensified silica fume (ACI Report comm. 226, 1987).

6. Dosage and mix design:

The normal dosage is in the range of 8 – 10 % by weight of cement in addition to the cement, however the optimum dosage of Cico silica fume for specific requirements should be determined by trials using materials and conditions.

2.1.6 Mixing water

" If water is fit to drink it is all right for making concrete" (Neville and Brooks, 1990), the water used for mixing is a normal tap water in lab.

2.2 Mix-design Proportioning:

The approach of ACI Manual of Concrete Practice MCP. (ACI 211.4R-93, 1993), has been adopted in mix design proportioning, and it is held to correlate with the proposal and scope of this research, which investigate the effect of mix proportioning and material source type on HSC, and related structural properties, so that; isolated effect of each of the mix design variables was assessed by eliminating interference caused by workability (all mixtures had acceptable or desired workability) or other mix design factors were kept constant.

2.2.1 Basalt materials mixes

1. W/C ratio kept constant and equal to 0.38
2. Coarse aggregate content kept constant and equal to 0.72 fractional volume of oven dry rodded coarse aggregate for maximum nominal aggregate size of 19mm, and 0.65 fractional volume of oven dry rodded coarse aggregate for 9.5mm maximum nominal aggregate size, which represent the optimum coarse aggregate content for the previous maximum nominal aggregate sizes according to 211.4R-93 table 4.3.3 ACI MCP.
3. Slump of each mix was controlled to be within 190-240 mm, which gives an equal same consistency and uniformity for all different mixes using HRWR.
4. Quantity of HRWR was subtracted from required mixing water while absorbed water from aggregates it self was added.
5. The only variable was binder content, which reflects on fine aggregates content. The binder content was raised from 475 to 600 kg/m^3, fine aggregates which were proportioned with all aggregates content (F.A/A) from 0.34 to 0.48, this were applied to mixes numbered as 1B, 2B, 3B, & 4B, the binder is fully Type 1 OPC, maximum nominal aggregate size is 19mm.
6. In mix # 2B the maximum aggregate nominal size was reduced to 9.5mm, with fractional volume of rodded oven dry coarse aggregate of 0.65 as recommended in step No.2 in mix numbered as 5B.
7. In mix 5B W/C was reduced to 0.23 and a partial replacement of Type I OPC by weight with silica fume of 15 % to obtain a very-high- strength mix in mix numbered as 6B.

Table 2.4 Summary of Mix-Design Proportions in research

Mix #	Type 1 OPC % (Quantity kg/m^3)	Silica Fume % (Quantity kg/m^3)	Water kg/m^3 (W/C or W/C+SF)	Coarse Agg. Kg/m^3	Fine Agg. Kg/m^3	A/C or A/C+SF Ratio.	F.A/Agg F.A/A% Ratio.	HRWR %Binder (Slump mm)
1B	100 (475)	0 (0)	210.9 (0.38)	1069.2	807.2	3.95	43	3 (190)
2B	100 (500)	0 (0)	219.5 (0.38)	1069.2	757.5	3.65	41	3 (190)
3B	100 (550)	0 (0)	236.6 (0.38)	1069.2	657.9	3.14	38	3 (220)
4B	100 (600)	0 (0)	253.8 (0.38)	1069.2	558.2	2.71	34	1 (190)
5B	100 (500)	0 (0)	219.5 (0.38)	1036.8	791	3.65	43	3 (190)
6B	85 (425)	15 (75)	148 (0.23)	1036.8	975.9	4.0	48	10 (50)
1SB	95 (475)	5 (25)	219.5 (0.38)	1069.2	757.5	3.65	41	3 (220)
2SB	90 (450)	10 (50)	219.5 (0.38)	1069.2	757.5	3.65	41	3 (200)
3SB	85 (425)	15 (75)	219.5 (0.38)	1069.2	757.5	3.65	41	3 (190)
1W	100 (475)	0 (0)	202.3 (0.38)	1014	750	3.71	43	1.5 (190)
2W	100 (500)	0 (0)	210.7 (0.38)	988	687	3.35	41	1.5 (220)
3W	100 (550)	0 (0)	230.3 (0.38)	983	602	2.88	38	1 (200)
4W	100 (600)	0 (0)	243 (0.38)	984	506	2.48	34	1 (240)
5W	100 (500)	0 (0)	217.9 (0.38)	1084.6	591	3.35	35	1.5 (190)
6W	85 (425)	15 (75)	146 (0.23)	1084.6	757	3.68	41	4 (50)
1SW	95 (475)	5 (25)	210.7 (0.38)	988	687	3.35	41	1.5 (240)
2SW	90 (450)	10 (50)	210.7 (0.38)	988	687	3.35	41	1.5 (220)
3SW	85 (425)	15 (75)	210.7 (0.38)	988	687	3.35	41	1.5 (190)

B: Basalt, W: Wadi, Binder: OPC alone or OPC partially replaced by silica fume

- Mix #2B is a reference mix for mixes numbered 5B, 1SB, 2SB, & 3SB
- Mix #2W is a reference mix for mixes numbered 5W, 1SW, 2SW, & 3SW

2.3 Procedure of Testing:

2.3.1 Mixing

1. Place the mix ingredients coarser ingredients then finer (e.g. Coarse aggregate, then fine aggregate over, then binder).
2. Running the mixer (drum mixer revolute into just horizontal direction) for 1 min. to mix the ingredients.
3. Monitoring the well mixing of dry mixture and improving it manually if needed.
4. Add the required quantity of superplasticizer to mixing water.
5. Running the mixer and pour the mixing water in & wait for 1 min.
6. Monitoring the fluidity of mixture and well mixing of it, improve it manually if needed.
7. Running the mixer for extra 1 min. to maintain highest possible performance of superplasticizer after recommendation of manufacture, and also to avoid segregation as much as possible.
9. After that and for each mix, immediately a slump cone test has been held and recorded according to ASTM C143.

2.3.2 Molding test specimens

- A 12 150*300 mm standard cylinders were used to test compressive, & splitting tensile strengths.
- A 2 standard 100*100*500 mm prisms were used to test flexure strength.
- The procedure of casting the molds was according to ASTM-C31 standards and summarized as following:

1. Tamping rod diameter = 16 mm.
2. Method of consolidation was by rodding if measured slump > 75 mm or by internal vibration if measured slump < 75 mm.
3. Molding number of layers were three equals one third of cylinder height if the method of consolidation is by rodding, else two layers one half of cylinder height if the method of consolidation is by internal vibration.
4. Number of tamps per layer = 25

2.3.3 Curing of test specimens

After molding immediately the molds were covered with sheets and stored in lab environment for 24 +_ 4h, then the specimens have been took out from molds and placed into final moist curing (tank) in lab until the 3 hours before testing according to ASTM C192.

2.3.4 Testing of specimens

1. For each mix the compressive strength has been measured at 3, 7, 14, 28, & 56 day age, using standard 150*300 cylinders according to ASTM C39.

2. Splitting tensile strength has been measured at 28-days age, using 150*300 standard cylinders according to ASTM C496.

3. Flexure strength has been measured by using simple beam standard prisms 100*100*500 with third point loading to analyze modulus of rupture of concrete according to ASTM C78.

- Specimens for compressive strength test were capped prior of testing using hard sulfur.

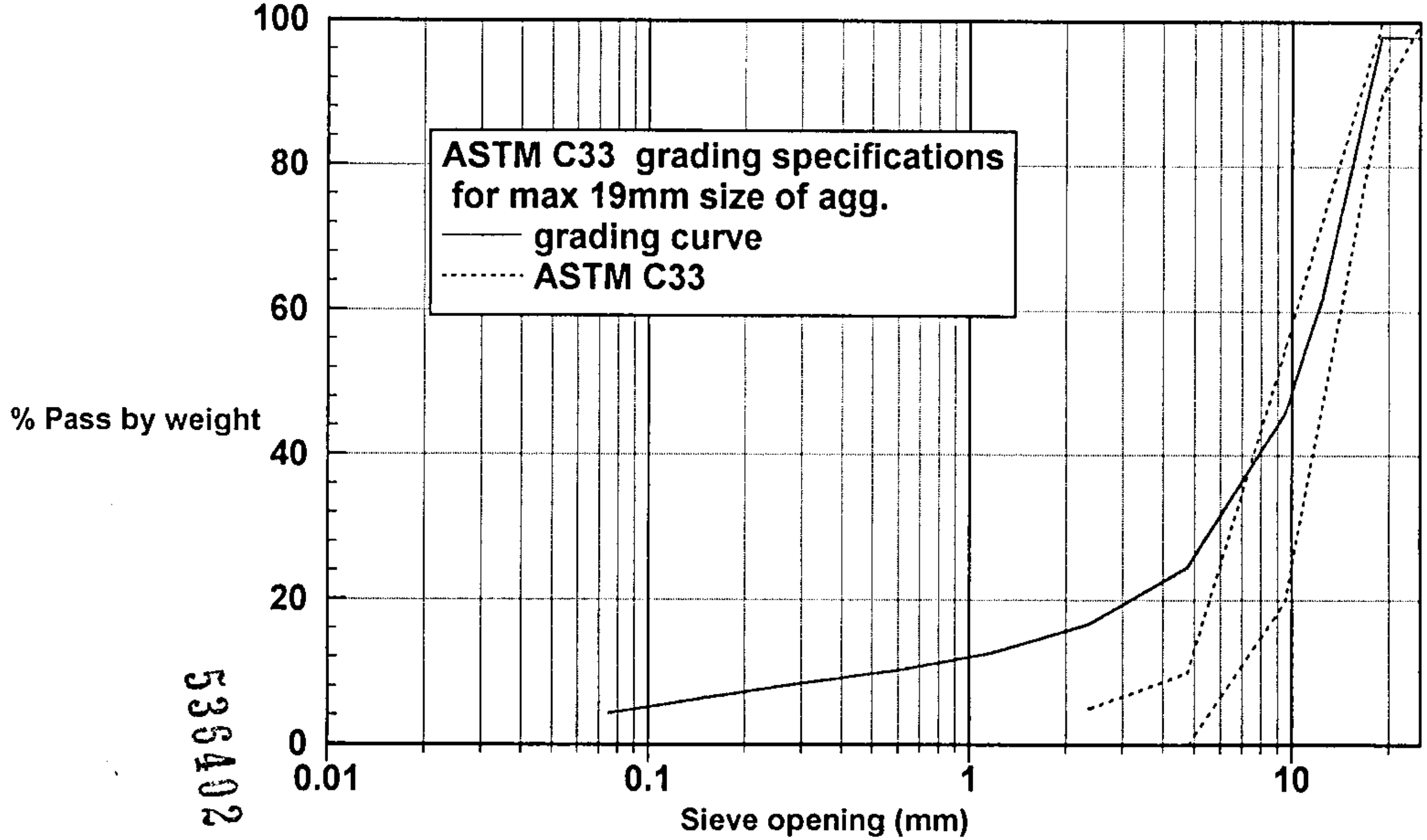

Figure 2.1 Basalt grading as delivered from source

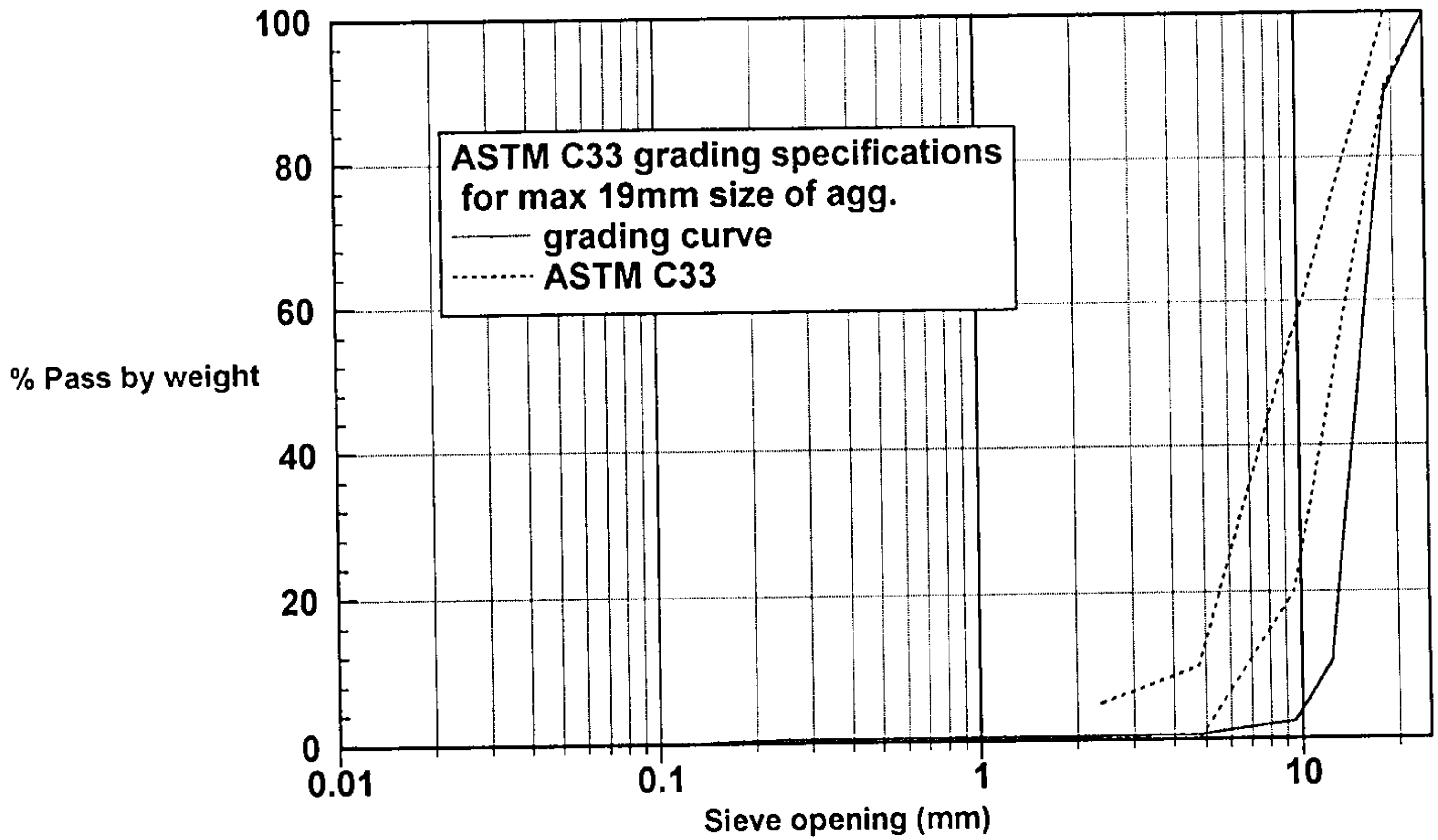

Figure 2.2 Wadi batch# "1W" grading as delivered from source

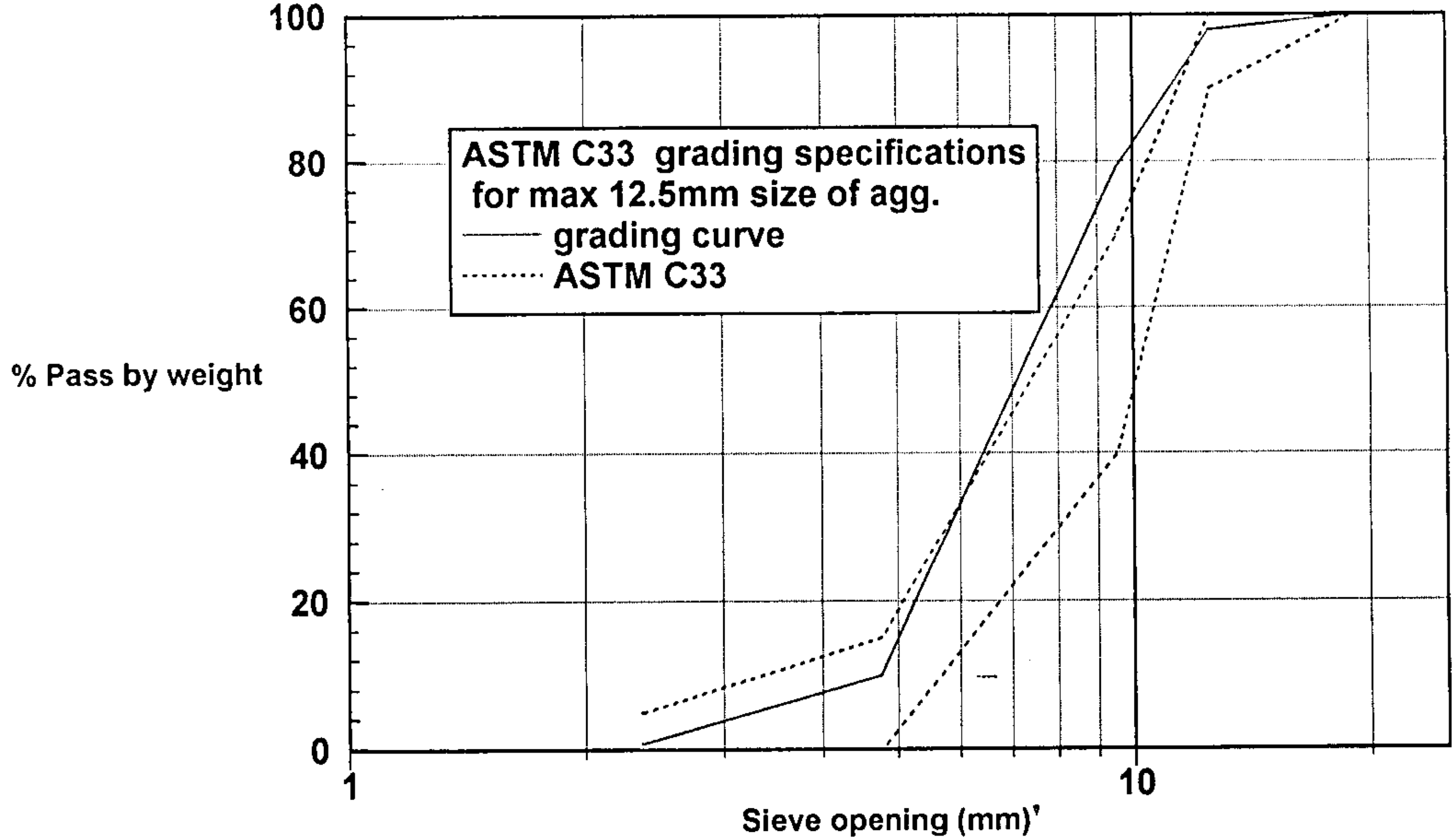

Figure 2.3 Wadi batch# "2W" grading as delivered from source

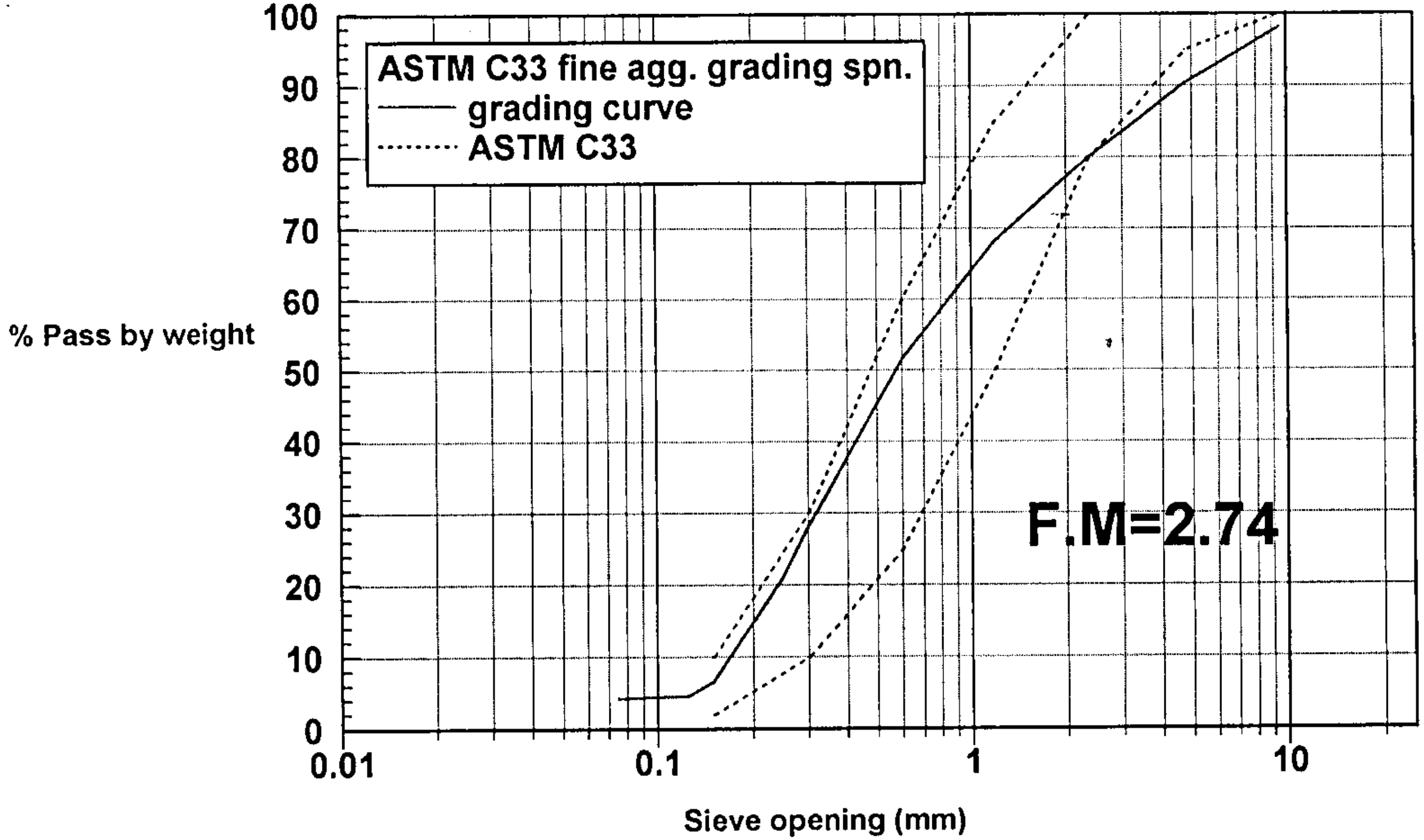

Figure 2.4 Wadi fine agg. grading as delivered from source

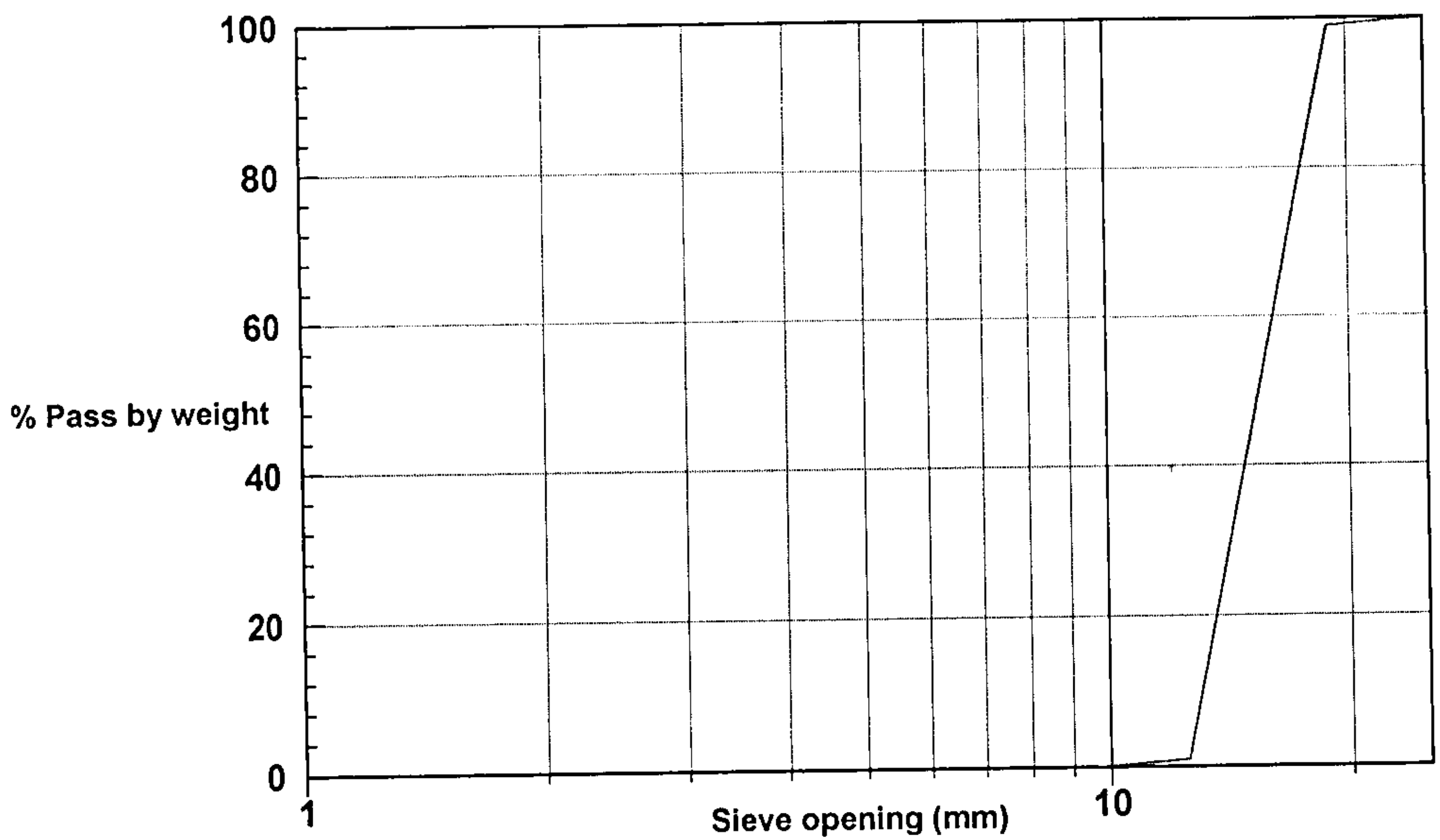

Figure 2.5 Grading chart of max 19 mm min 14 mm Basalt batch #1B

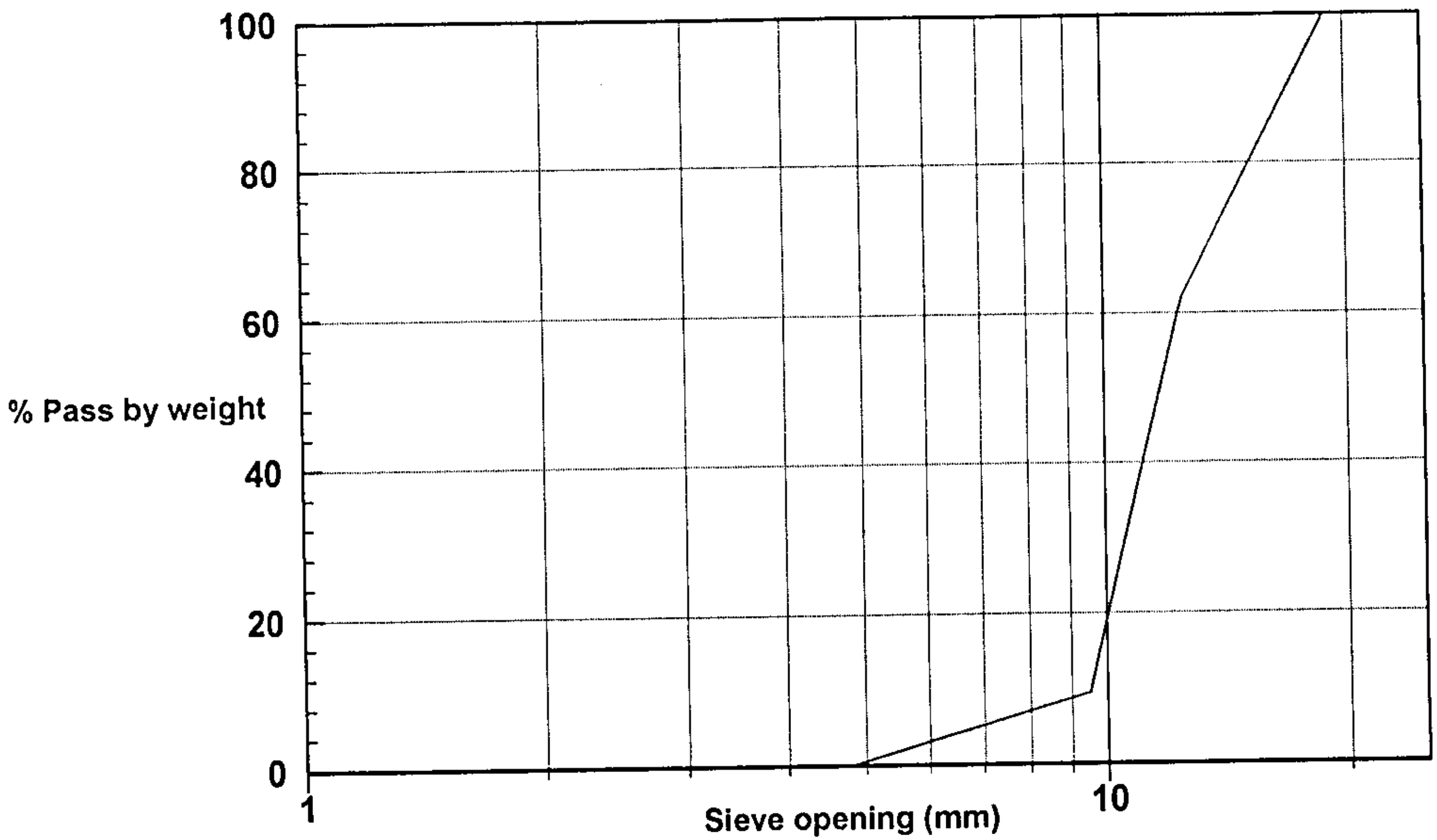

Figure 2.6 Grading chart of max 14 mm min 9.5 mm Basalt batch #2B

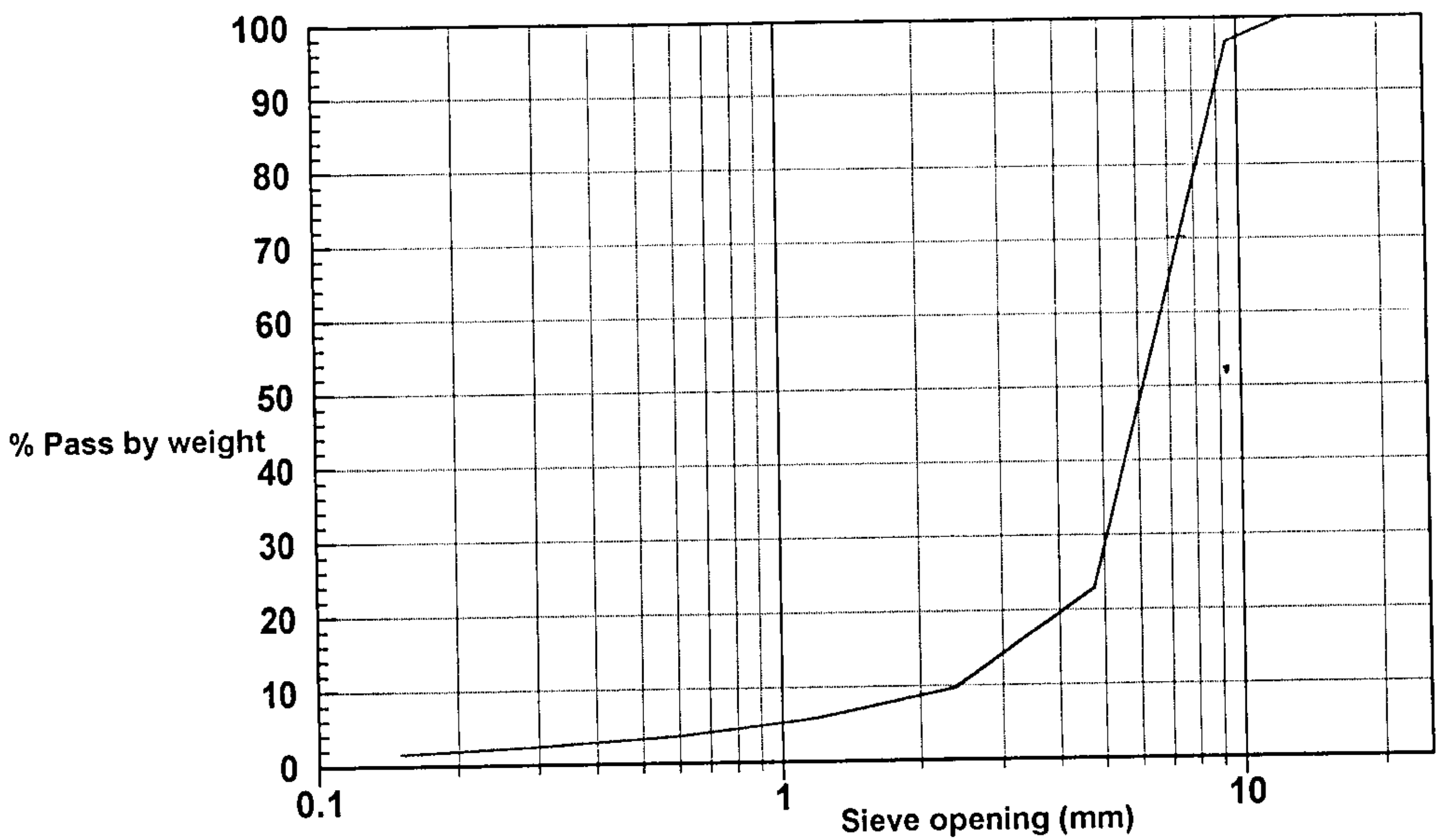

Figure 2.7 Grading chart of max 9.5 mm Basalt batch #3B

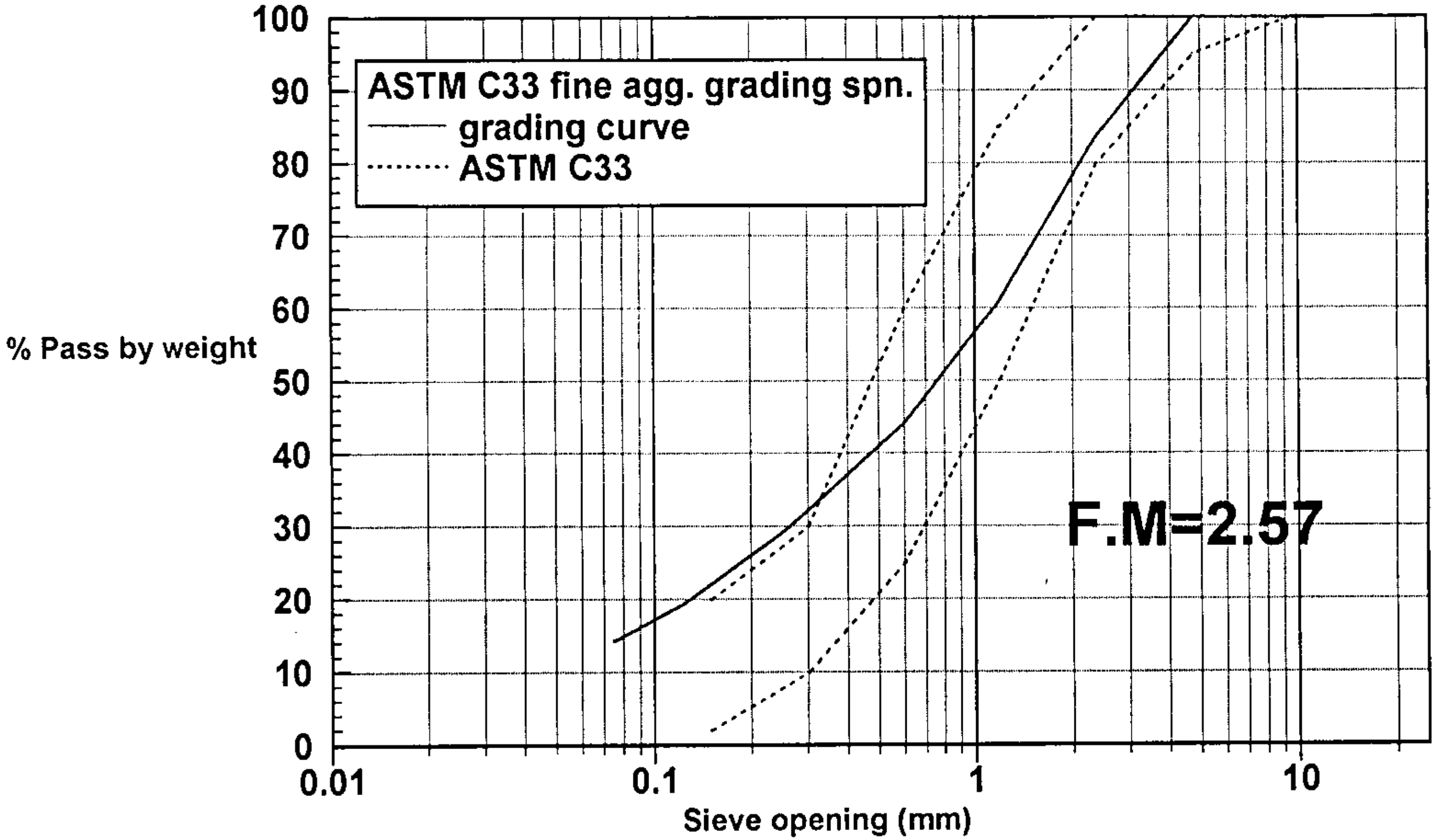

Figure 2.8 Basalt fine agg. grading as delivered from source

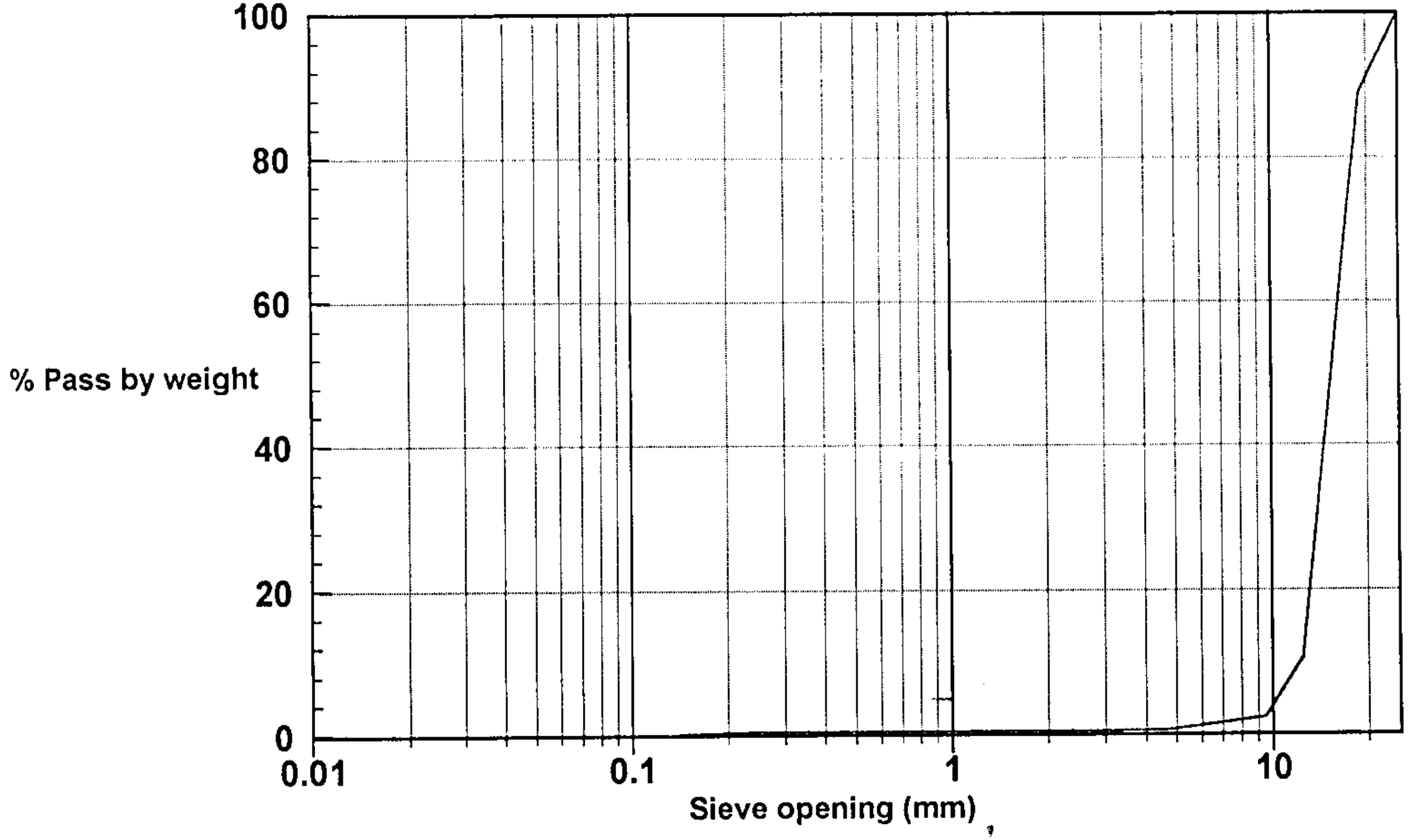

Figure 2.9 Grading Chart of Wadi batch#1W

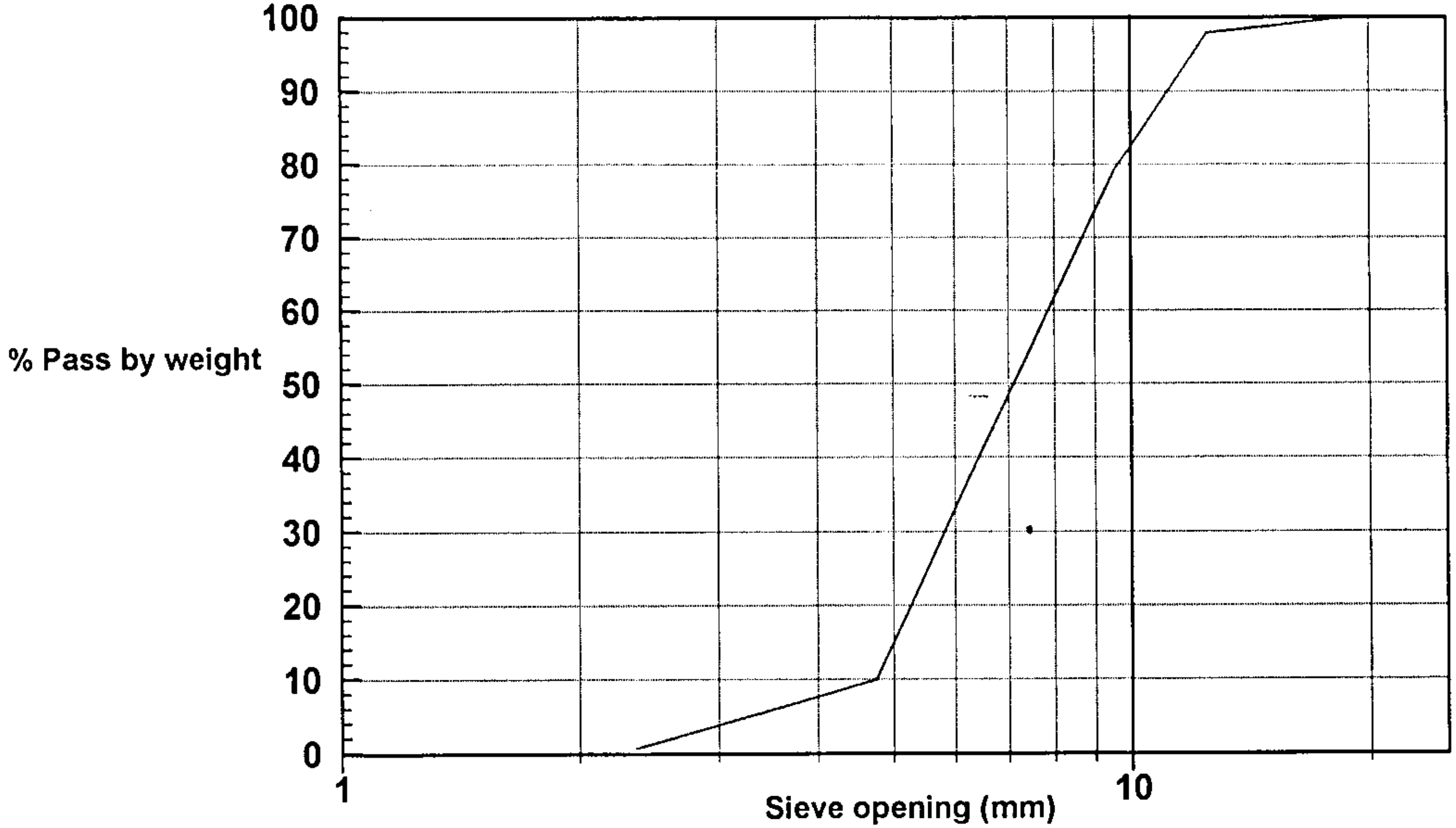

Figure 2.10 Grading Chart of Wadi batch#2W

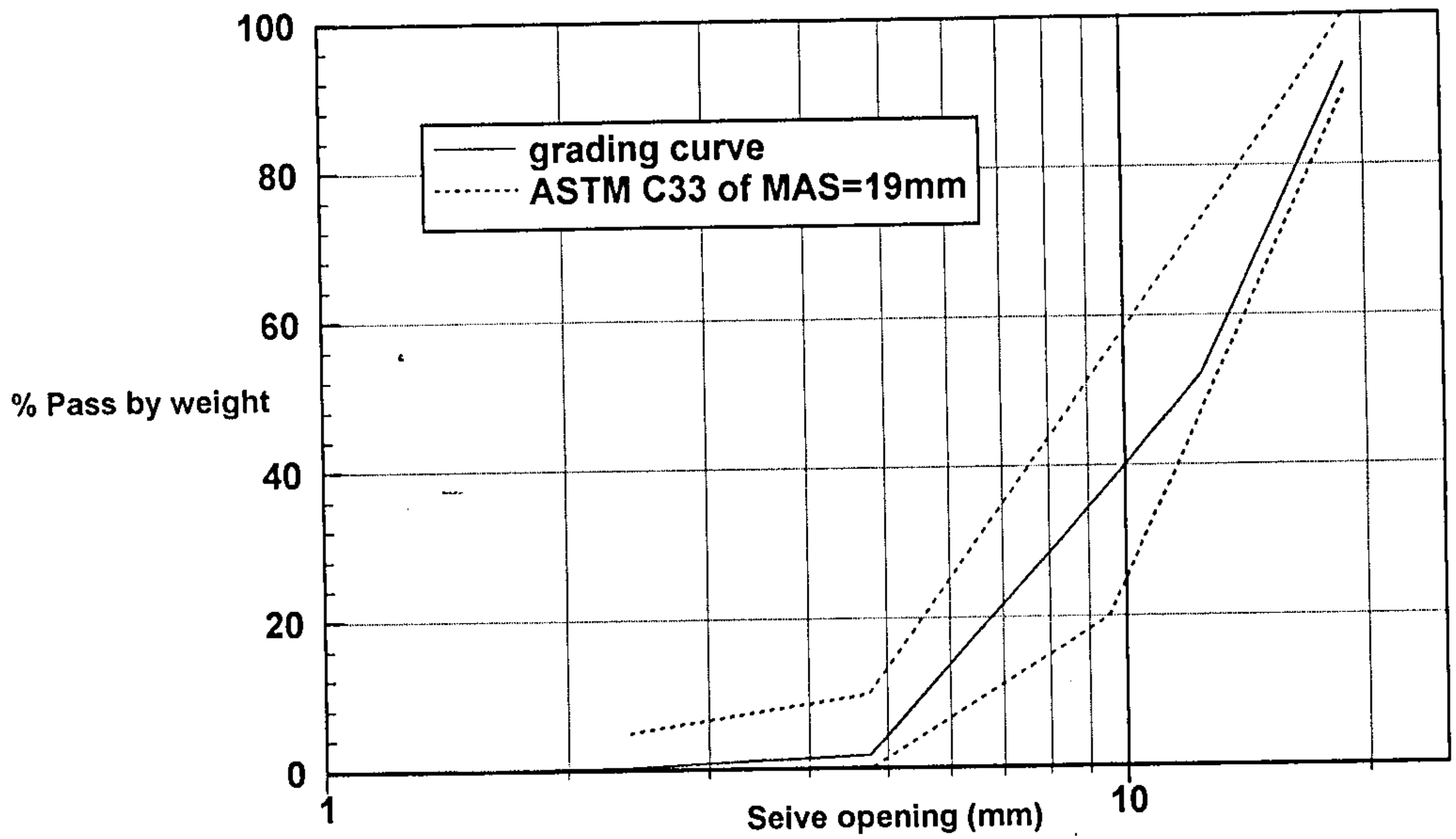

Figure 2.12 Final grading of Wadi coarse agg. based on 1:1 (1W:2W) of two main batches of max size 19&12.5 mm

3- Results & Discussion

3.1 Compressive Strength of HSC:

180 Standard 150*300 mm cylinders (10 for each mix) have been tested for uniaxial compressive strength at the age of 3, 7, 14, 28, & 56 days according to ASTM C39.

The mix-design proportions and scope of research are presented in sec 2.2, Table 2.4.

3.1.1 Effect of Aggregate/Cement Ratio (A/C) on compressive strength of HSC:

A/C ratio has been varied to investigate its effect on compressive strength of HSC, and for both Basalt and Wadi materials.
The test results are presented in Tables 3.1, & 3.2, Figures 3.1, & 3.2.

Table 3.1 Compressive strength development with time the effect of varied A/C ratio for Basalt Agg. Mixes.

		Compressive strength (MPa)				
Mix#	A/C	3-d	7-d	14-d	28-d	56-d
1B	3.95	43.0	44.7	56.8	60.0	69.0
2B	3.65	44.1	46.1	57.2	62.3	72.2
3B	3.14	38.2	39.1	48.7	51.6	68.3
4B	2.71	32.0	33.8	39.9	42.8	57.0

Table 3.2 Compressive strength development with time the effect of varied A/C ratio for Wadi Agg. Mixes.

		Compressive strength (MPa)				
Mix#	A/C	3-d	7-d	14-d	28-d	56-d
1W	3.71	34.8	36.0	43.4	48.4	51.8
2W	3.35	33.7	34.8	40.5	45.1	50.2
3W	2.88	20.8	31.2	38.2	43.4	46.4
4W	2.48	20.0	26.3	36.4	39.6	42.4

Discussion of Compressive Strength Results:

1. Test results show that for the same maximum agg. Size (MAS), gradation, & W/C, for HSC a leaner (high-A/C) mixes show a higher compressive strength up to optimum value of A/C (Figures 3.3, & 3.4). That because in HSC mixes there are already a high cement content, So the W/C is not the only factor controlling most mechanical properties, including the compressive strength as the case in conventional normal strength concrete NSC.

   The situation appears to be different in HSC whose mechanical properties are affected by improving cement paste and interfacial transition zone ITZ properties, the stress being shared by the two components: the aggregate and cement paste - more like composite behavior than NSC- (Cetin and Carrasquillo, 1998).

   Note that the results conform with the theory presented in sec 1.3.2 for A/C ratio effect, that whenever increasing cement content (decreasing A/C ratio //rich mix) in HSC, the maximum paste thickness MPT increases and becomes strength limiting factor which is more porous and weaker than aggregate used in HSC production, which leads to lower strength (de Larrad and Belloc, 1997).

2. The optimum ratio shows that there is a required amount of cement paste to cover all surface area of aggregates properly to form a concrete matrix and this content affect A/C for sure which in result affected by:

   a- Volume of Aggregates.
   b- MAS.
   c- Gradation of Aggregates.
   d- Shape & Surface Texture of Aggregates.

3. The previously mentioned effects clarify that why Wadi material mixes have not reach optimum value of A/C ratio (Fig 3.3) like what happened to basalt ones, for the mix design proportioning adopted, see page 29, sec 2.2.2. (Wadi- mixes still having a potential to have a higher strength for more leaner mix), and that because of its rounded shape which offer less surface area than angular Basalt Aggregates hence, requires lower cement paste to cover all surfaces without being thick and limiting factor itself.

4. Although the same aggregate volume, W/C, MAS, were used in both Basalt and Wadi mixes, Basalt mixes show a significantly higher compressive strength than Wadi mixes do, and at all ages, this effect reduced with decreasing A/C but however remains significant (at 28-d from 8 to 37%), see Figures 3.5, 6, 7, & 8.

5. The increase of strength to the right of Basalt Agg. mixes resulted in fact from different type of aggregate used which is very important issue in production of HSC to choose properly suited materials to improve the properties of ITZ, the following properties for aggregate affects strength and clarify why Basalt agg. mixes results are stronger than Wadi ones :

a- Strength of aggregate:

Undoubtedly and irrespective of the grading and shape of particles the strength potential of the rock has a very important role in HSC.

Basalt coarse aggregates have a lower abrasion 20% loss by weight versus 26.8% for Wadi Coarse aggregate (see Table 2.2), which indicates a higher strength potential for Basalt coarse aggregates over those of Wadi coarse aggregates.

Basalt aggregates have a higher specific gravity (S.G see Table 2.2), than Wadi aggregates, which indicates stiffer aggregates, which reduces the difference in stiffness between cement paste and aggregates (Cetin and Carrasquillo, 1998).
Difference in stiffness is the responsible of stress concentrations at high-stress levels, so a stiffer aggregates, the lesser stiffness difference, the lesser the stress concentration, the more

homogeneous (composite action), finally leads to higher strength (Cetin and Carrasquillo, 1998).

b- Shape and Texture of Aggregates:

Due to mechanical interlocking, and surface texture, a rough angular Basalt aggregate mixes possess a better bond between the cement paste and aggregates, more than a smooth rounded Wadi aggregates (Neville and Brooks, 1990).

Increasing aggregates roughness leads to higher interface (ITZ) fracture energy (Trende and Buyukozturk, 1998).

3.1.2 Effect of Maximum Aggregate Size (MAS) on Compressive Strength of HSC:

The MAS in mix 2B & 2W which are stated as reference mixes was reduced to 9.5mm, and 12.5mm respectively to study the MAS effect on compressive strength of HSC.

The results are presented in Tables 3.3, & 3.4.

Table 3.3 Compressive strength development with time the effect of MAS for Basalt Agg. mixes

Mix#	MAS (mm)	Compressive strength (MPa)				
		3-d	7-d	14-d	28-d	56-d
2B	19	44.1	46.1	57.2	62.3	72.2
5B	9.5	52.0	53.8	61.0	66.5	75.0

Table 3.4 Compressive strength development with time the effect of MAS for Wadi Agg. Mixes

Mix#	MAS (mm)	Compressive strength (MPa)				
		3-d	7-d	14-d	28-d	56-d
2W	19	33.7	34.8	40.5	45.2	50.2
5W	12.5	36.2	41.0	45.4	47.0	50.7

1. Test results show a slightly higher compressive strength when MAS reduced for both, Basalt and Wadi mixes (Figures 3.9, & 3.10).

   This related to that a lower MAS leads to a higher surface area of aggregates which utilize higher cement paste without increasing maximum paste thickness MPT, hence higher compressive strength (de Larrad and Belloc, 1997).

2. That is correct for HSC mixes because they are already rich mixes (high cementitious materials content), and it is not necessarily correct for conventional Normal Strength Concrete NSC mixes with normal cement content.

3. The balancing of two effects (cementitious materials content & MAS) leads, that there is an optimal MAS for each cement content level -richness of mix- (Cordon and Gillespie, 1963).

3.1.3 Effect of Silica Fume (SF) on Compressive Strength of HSC:

Again mixes 2B & 2W are stated as references and they are retested for compressive strength at 3, 7, 14, 28, & 56 days but using a partial replacement of SF of 5, 10, & 15% by weight instead of full OPC as a cementitious material.

The results are presented in Tables 3.5, & 3.6.

Table 3.5 Compressive strength development with time the effect of SF as a partial replacement of OPC for Basalt Agg. mixes

Mix#	SF%	Compressive strength (MPa)				
		3-d	7-d	14-d	28-d	56-d
2B	0	44.1	46.1	57.2	62.3	72.2
1SB	5	49.4	50.4	58.0	65.6	70.3
2SB	10	39.6	51.8	58.9	77.4	80.4
3SB	15	37.4	49.7	58.0	75.6	79.5

Table 3.6 Compressive strength development with time the effect of SF as a partial replacement of OPC for Wadi Agg. mixes.

Mix#	SF%	Compressive strength (MPa)				
		3-d	7-d	14-d	28-d	56-d
2W	0	33.7	34.8	40.5	45.1	50.2
1SW	5	34.8	35.9	41.5	47.5	48.7
2SW	10	24.7	43.0	53.2	61.1	62.3
3SW	15	24.6	43.2	53.2	60.8	61.1

Note that test results shows a different behavior when using a partial replacement level of 5% other than when using levels of 10-15%, so each item will be discussed separately.

A. 5% Level of replacement:

For 5% level of replacement the high-early strength at 3-days age is more than reference mixes for both Basalt and wadi.

The compressive strength continue higher than reference mixes effectively from 3 to 28-days age , but have approach the same strength for reference mixes at 56-days age (Figures 3.11, & 3.12).

The effect of 5% level is more significant into fresh state of concrete, that relating to table 2.4, note that slump increases more than reference mixes when using the level of 5% SF replacement (increases workability) This complying with that at low SF doses < 5% , SF very fine particles displace water to become free to help with the flowability of concrete (water reducing effect), more details cleared in sec 1.1.5 (Kuennen, 1996).

B. 10-15% Level of replacement:

1. For 10-15% SF partial replacement test results shows a lower early- compressive-strength at the age of 3-days less than reference mixes.
   But after the age of 7-days the compressive strength of SF mixes becomes significantly higher than reference mixes

2. Compressive strength wasn't significantly enhanced by increasing SF replacement level from 10 to 15% (they are slightly the same Compressive strength) (Figures 3.11, & 3.12).

3. A major contribution of the SF in strengthening of the concrete matrix, is its influence on the micro-structure of the interfacial transition zone (ITZ) and subsequently the aggregate-paste bond which summarized as following:

   a. Mercury intrusion porosimetry has shown that SF makes the pore structure of paste and mortar more homogenous by decreasing the number of large pores - micro-filler effect – (ACI 234R-96, 1996).

   b. The presence of SF resulted in reduction in $Ca(OH)_2$-free lime- which is one of hydration products, this reduction becomes larger with increase in SF% level and age.

      The responsible of this reduction is the pozzolanic reaction between SF & Free lime ($Ca(OH)_2$) to produce Calcium-Silicate-Hydrate (CSH) crystals which is one of hardened hydration products which contributes in minimizing pores - Pozzolanic effect - (Goldman and Benture, 1989).

   c. Hence the micro-structure of (ITZ), becomes less porous which resulted in more homogenous transition zone without the presence of porous pockets or $Ca(OH)_2$ rim (Goldman and Benture, 1989).

4. As indicated by test results the presence of SF accelerates the early gain of strength generally at the age of 7-days and after for all replacement levels.

   This acceleration is due to that SF accelerates the hydration of cement resulted from mere presence of numerous fine particles has a catalytic effect on cement hydration (ACI 234R-96, 1996).

5. It is interesting to observe that at the level of 5% SF replacement there were a higher early-strength at the age of 3-days than reference mix and more levels of 10-15% which are at the contrary resulted into lower early-strength at the age of 3-days than those of reference mixes until the age of 7 days, (Figures 3.11, & 3.12).

   This may be explained on the basis of pozzolanic behavior of SF, so that at the level of 5% even at very-early age there were a sufficient content of $Ca(OH)_2$ produced to make the pozzolanic reaction of low-level SF replacement counteract the effect of lowering cement content.
   On the other hand at higher replacement levels it took time for cement to hydrate and produce a sufficient quantity of $Ca(OH)_2$ to utilize pozzolanic effect of SF. And the test results shows that time period to do that was not more than 7-days what-ever the level of replacement is (Figures 3.11, & 3.12).

6. Test results show also that there are no further increase in compressive strength with further increasing of SF level of replacement after 10%. This because of a reduction of relative effectiveness of SF to improve mechanical properties of mixtures with low W/C or W/C+SF ratios, due to fact that the importance of densification of capillary pores contributed to the use of SF can be decreased with the reduction of capillary porosity already with low W/C or W/C+SF ratios.

   Furthermore the beneficial effect of SF in improving cohesiveness and reducing imperfections at the interface between paste and aggr--egates (ITZ), decreases with the increase in cement content which is the case in HSC (Khayat *et al.*, 1997).

   Hence, for a specific W/C+SF and cement content, all other strength determining factors fixed, there is an optimal SF per cent of replacement, so that no more extra strength gained, or it may be reduce the strength at very-high levels over that optimal level of replacement (becomes a matter of lower cement content).

7. Finally its clear that Wadi mixes have been relatively more affected when using SF comparing with reference mix than Basalt, because Basalt aggregates shows already a stronger bond properties more than Wadi aggregates, which reduces relatively the effect of SF over Basalt aggregate mixtures.

3.1.4 Toward Very-High-Compressive Strength Value:

A mix of 0.23 W/C+SF ratio, with favorable conditions resulted from previous HSC mixes of MAS 9.5 & 12.5 mm for Basalt and Wadi respectively, & 15% SF as a partial replacement, the aggregate volume used according to optimum specified by (ACI 211.4R-93, 1993).

This helps into extending trends to a very-high value of compressive strength the results are presented in Table 3.7, (Figures 3.13, & 3.14).

Table 3.7 Compressive strength development (MPa) with time the effect of W/C+SF =0.23, MAS=9.5mm for 6B, 12.5mm for 6W, 15% SF partial replacement from OPC (see table 2.4 for details of mix proportions)

Mix#	Mix type	7-d	28-d	56-d
6B	Basalt	79.2	94.2	94.8
6W	Wadi	80.0	88.0	88.7

3.1.5 Rate of Gain Compressive Strength with Time for HSC:

Test results indicates that the rate of gaining compressive strength with time affected by the following factors assuming a fixed W/C:

1. Type of Aggregates Basalt or Wadi.
2. A/C ratio.
3. MAS.
4. Presence of SF as a partial replacement & the level of replacement.

The rate of compressive strength gain as a per cent of 56-days compressive strength are presented in Tables 3.8-19.

Table 3.8 Compressive strength Rate of Gain with time as a Percent of 56-days Compressive Strength A/C effect (Basalt mixes).

Mix Details	A/C	3-d	7-d	14-d	28-d
1B W/C =0.38 MAS = 19mm	3.95	62%	65%	82%	87%
2B W/C =0.38 MAS = 19mm	3.65	61%	64%	80%	86%

Table 3.9 Compressive strength Rate of Gain with time as a Percent of 56-days Compressive Strength A/C effect (Basalt mixes).

Mix Details	A/C	3-d	7-d	14-d	28-d
3B W/C =0.38 MAS = 19mm	3.14	56%	57%	71%	76%
4B W/C =0.38 MAS = 19mm	2.71	56%	59%	70%	75%

Table 3.10 Compressive strength Rate of Gain with time as a Percent of 56-days Compressive Strength MAS effect (Basalt mixes).

Mix Details	MAS	3-d	7-d	14-d	28-d
2B W/C =0.38 A/C =3.65	19 mm	61%	64%	80%	86%
5B W/C =0.38 A/C =3.65	9.5 mm	69%	72%	81%	89%

Table 3.11 Compressive Strength Rate of Gain with time as a Percent of 56-days Compressive Strength SF effect (Basalt mixes).

Mix Details	SF%	3-d	7-d	14-d	28-d
2B W/C =0.38 MAS = 19mm A/C = 3.65	0.0	61%	64%	80%	86%
1SB W/C+SF =0.38 MAS = 19mm A/C+SF = 3.65	5	70%	72%	82%	93%

Table 3.12 Compressive Strength Rate of Gain with time as a Percent of 56-days Compressive Strength SF effect (Basalt mixes).

Mix Details	SF%	3-d	7-d	14-d	28-d
2B W/C =0.38 MAS = 19mm A/C =3.65	0.0	61%	64%	80%	86%
2SB W/C+SF = 0.38 MAS = 19mm A/C+SF =3.65	10	49%	64%	73%	96%
3SB W/C+SF =0.38 MAS = 19mm A/C+SF =3.65	15	47%	62%	73%	95%

Table 3.13 Compressive Strength Rate of Gain with time as a Percent of 56-days Compressive Strength the effect of SF+MAS+A/C (see table 2.4 for details of mix proportions).

Mix Details	Mix type	7-d	28-d
6B W/C+SF = 0.23 MAS = 9.5mm A/C+SF = 4.0	Basalt	84%	99%

Table 3.14 Compressive strength Rate of Gain with time as a Percent of 56-days Compressive Strength A/C effect (Wadi mixes).

Mix Details	A/C	3-d	7-d	14-d	28-d
1W W/C =0.38 MAS = 19mm	3.71	67%	70%	84%	93%
2W W/C =0.38 MAS = 19mm	3.35	67%	69%	81%	90%

Table 3.15 Compressive strength Rate of Gain with time as a Percent of 56-days Compressive Strength A/C effect (Wadi mixes).

Mix Details	A/C	3-d	7-d	14-d	28-d
3W W/C =0.38 MAS = 19mm	2.88	45%	67%	82%	94%
4W W/C =0.38 MAS = 19mm	2.48	47%	62%	86%	93%

Table 3.16 Compressive strength Rate of Gain with time as a Percent of 56-days Compressive Strength MAS effect (Wadi mixes).

Mix Details	MAS	3-d	7-d	14-d	28-d
2W W/C =0.38 A/C =3.35	19 mm	67%	69%	81%	90%
5W W/C =0.38 A/C =3.35	12.5 mm	71%	81%	90%	93%

Table 3.17 Compressive Strength Rate of Gain with time as a Percent of 56-days Compressive Strength SF effect (Wadi mixes).

Mix Details	SF%	3-d	7-d	14-d	28-d
2W W/C =0.38 MAS = 19mm A/C+SF = 3.35	0.0	67%	69%	81%	90%
1SW W/C+SF =0.38 MAS = 19mm A/C+SF = 3.35	5	72%	74%	85%	98%

Table 3.18 Compressive Strength Rate of Gain with time as a Percent of 56-days Compressive Strength SF effect (Wadi mixes).

Mix Details	SF%	3-d	7-d	14-d	28-d
2W W/C =0.38 MAS = 19mm A/C =3.35	0.0	67%	69%	81%	90%
2SW W/C+SF = 0.38 MAS = 19mm A/C+SF =3.35	10	40%	69%	85%	98%
3SB W/C+SF =0.38 MAS = 19mm A/C+SF =3.35	15	40%	71%	87%	99%

Table 3.19 Compressive Strength Rate of Gain with time as a Percent of 56-days Compressive Strength the effect of SF+MAS+A/C (see table 2.4 for details of mix proportions).

Mix Details	Mix type	7-d	28-d
6W W/C+SF = 0.23 MAS = 12.5mm A/C+SF = 3.68	Wadi	90%	99%

❖ The test results indicate the following:

1. Rate of gaining strength for a higher A/C ratios (3.65 to 3.95) gives a higher rate of gaining strength more than (3.14 to 2.71) A/C ratios for Basalt (Tables 3.8, & 3.9). The same result for Wadi mixes with A/C ratio (3.35-3.71) shows a higher rate of gaining strength more than A/C ratio of (2.48 to 2.88), (Tables 3.14, & 3.15, Figures 3.15-18).

2. Lowering the MAS for both of aggregate mixes increases the rate of gaining strength (Tables 3.10, & 3.16, Figures 3.19, & 3.20).

3. Presence of 5% SF as a partial replacement level, increases the rate of gaining strength more than the reference mixes without SF & for both Basalt and Wadi mixes, from the age of 3-days till 28-days, the effect of SF mixes after 28-days are minimal or very small raising in strength (ACI 234R-96, 1996), the effect of SF on the rate of hydration is detailed in sec 3.1.3 (Tables 3.11, & 3.17, Figures 3.21, & 3.22).

4. Presence of 10-15% partial replacement of SF resulted in a higher rate of gaining strength but always after the age of 7-days until 28-days. It shows always a lower rate at the age of 3-days comparing with the reference mixes without SF, also the effect of SF of raising in strength after 28-days are very small (Tables 3.12, & 3.18, Figures 3.23, & 3.24).

5. A highest rate of gaining strength have been found in mixes number 6B, & 6W, which have almost the most preferable factors for accel-

3.2 Splitting Tensile Strength of HSC:

A 36 standard 150*300 mm cylinders have been tested for splitting tensile strength (Ft) at the age of 28-days & for both types of aggregates, according to ASTM C496, to investigate splitting tensile strength of HSC and its response of varying A/C, type of aggregates, MAS, & use of SF. Finally a recommended relation between SQRT f'c and splitting tensile strength will be drawn.

3.2.1 Effect of A/C & Type of Aggregates on Splitting Tensile strength (f_t) of HSC:

A/C ratio have been varied to investigate its effect on splitting tensile strength for both Basalt & Wadi mixes, the results are presented in Tables 3.20, & 3.21.

Table 3.20 f'c at 28-d Vs f_t at 28-d, the effect of A/C, for Basalt mixes.

Mix#	A/C	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
1B	3.95	60.0	5.48	9.1
2B	3.65	62.3	5.55	8.9
3B	3.14	51.6	5.13	10.0
4B	2.71	42.8	4.98	11.6

Table 3.21 f'c at 28-d Vs f_t at 28-d, the effect of A/C, for Wadi mixes.

Mix#	A/C	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
1W	3.71	48.4	3.32	6.9
2W	3.35	45.1	3.26	7.2
3W	2.88	43.4	3.06	7.0
4W	2.48	39.6	2.99	7.5

1. Test results shows generally & as predicted a direct proportioning of splitting tensile strength f_t with compressive strength f'c, that a higher average compressive strength value will produce a higher splitting tensile strength f_t.

   Hence its noticed that the effect of A/C on the compressive strength of concrete is again correct for splitting tensile strength, that a leaner mix (higher A/C ratio) up to optimum value shows a higher splitting tensile strength for both types of aggregate mixes which is -

proportional directly with compressive strength, the effect of A/C on compressive strength of concrete has been discussed in sec 3.1.1.

2. However results show a significant increase in splitting tensile strength when we change from wadi aggregate mixes to Basalt ones, that's clear from notice that the increase of (65-70%) in splitting tensile strength in Basalt mixes versus (8-38%) in compressive strength.

   This supports that the effect of shape and surface texture of aggregates have a large effect on tensile strength more than compressive strength - tripled in average here - (Neville and Brooks, 1990), because the change in fact was from rounded and smooth shape and surface texture of wadi aggregates toward angular and rough surface of Basalt aggregates, this irrespective of already the difference in strength between two materials which discussed in sec 3.1.1.

3.2.2 Effect of MAS on Splitting Tensile Strength (f_t) of HSC:

The MAS in mix 2B & 2W which stated as reference mixes, was reduced to 9.5, & 12.5 mm respectively to investigate the MAS effect on splitting tensile strength, the test results presented in Tables 3.22, & 3.23.

Table 3.22 f'c at 28-d Vs f_t at 28-days, the effect of MAS, for Basalt mixes.

Mix#	A/C	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
2B	19	62.3	5.55	8.9
5B	9.5	66.5	5.71	8.6

Table 3.23 f'c at 28-d Vs f_t at 28-d, the effect of MAS, for Wadi mixes.

Mix#	A/C	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
2W	19	45.1	3.26	7.2
5W	12.5	47.0	5.00	10.6

1. According to improvement in mechanical properties induced by lowering MAS as discussed in sec 3.1.2, splitting tensile strength posses higher values with lowering MAS.

2. How-ever Wadi mixes shows a significant increase in splitting tensile strength when lowering MAS more than Basalt ones, and this probably because lowering MAS in Wadi mixes decreases the deficiencies in shape and surface texture (lesser effect) for Wadi aggregates.

3.2.3 Effect of SF on Splitting Tensile Strength (f_t) of HSC:

Mixes 2B, & 2W are stated as reference mixes, and retested for splitting tensile strength at 28-days using the levels of 5, 10, & 15% partial replacement by weight of OPC, the results are presented in Tables 3.24, & 3.25.

Table 3.24 f'c at 28-d Vs f_t at 28-d, the effect of SF, for Basalt mixes.

Mix#	A/C	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
2B	0.0	62.3	5.55	8.9
1SB	5.0	65.6	5.66	8.6
2SB	10.0	77.4	6.44	8.3
3SB	15.0	75.6	6.22	8.2

Table 3.25 f'c at 28-d Vs f_t at 28-d, the effect of SF, for Wadi mixes.

Mix#	A/C	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
2W	0.0	45.1	3.26	7.2
1SW	5.0	47.5	4.53	9.5
2SW	10.0	61.1	4.72	7.7
3SW	15.0	60.8	4.94	8.1

1. The improvement in micro-structure of concrete provided by using SF have been discussed in sec 3.1.3, accordingly test results shows a higher splitting tensile strength with increasing SF% level up to optimum value which after no further strength could be gained with more SF% (sec 3.1.3).

2. Wadi aggregates mixes have been improved more significantly than Basalt aggregates mixes for the same reason discussed in sec 3.2.1.

3.2.4 Toward Very-High Splitting Tensile Strength Value (f_t) of HSC:

The mixes 6B, & 6W presented in sec 3.1.4, have been tested for splitting tensile strength at 28-days age to get a highest possible value in the view of test results, this helps to extending the trend of data for a higher values to make a more proper prediction of material behavior at high stress levels. Results are presented in Tables 3.26, & 3.27.

Table 3.26 f'c at 28-d Vs f_t at 28-d (highest value), for Basalt mixes.

Mix Details	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
6B W/(C+SF) = 0.23 MAS =9.5 mm A/(C+SF) = 4.0 SF% = 15.0	94.2	6.69	7.1

Table 3.27 f'c at 28-d Vs f_t at 28-d (highest value), for Wadi mixes.

Mix Details	f'c (MPa) 28-days	f_t (MPa) 28-days	f_t/f'c %
6W W/(C+SF) = 0.23 MAS =12.5 mm A/(C+SF) = 3.68 SF% = 15.0	88.0	6.25	7.1

- Note that for all conditions the ratio Ft/F'c was decreased by increasing compressive strength, and it was ranged between 7.1-10.6 % for Wadi materials, & 7.1-11.6% for Basalt materials which shows a higher potential for Basalt mixes into splitting tensile strength/ compressive strength ratio, knowing that the theoretical relation is 12.5 % (tensile strength/ compressive strength) for normal-strength concrete (Neville and Brooks, 1990).

- How-ever wadi mixes shows also increase in f_t/f'c when improve it by lowering MAS &/or using SF.

3.2.5 Regression Analysis f_t Versus f'c for HSC:

A regression analysis has been done to draw a relation between SQRT of f'c at 28-days and splitting tensile strength at the same date, & its summarized as following:

1. It has been founded that Basalt mixes likely to have same behavior relation between SQRT of F'c at 28-days and splitting tensile strength what-ever are the MAS, A/C, & SF% (The relation directly affected by compressive strength). The analysis presented in Table 3.28 and Figure 3.27.

Table 3.28 SQRT (f'c) Vs f_t (Regression analysis for Basalt mixes)

Mix#	SQRT (f'c) @ 28-d	f_t MPa @ 28-d
4B	6.54	4.98
3B	7.19	5.13
1B	7.75	5.48
2B	7.89	5.55
1SB	8.1	5.66
5B	8.15	5.71
3SB	8.69	6.22
2SB	8.80	6.44
6B	9.70	6.69

2. The case is different with Wadi materials mixes which shows a significant enhancement of splitting tensile strength when lowering MAS and/or using SF, so the two items will be analyzed separately as following:

a. Mixes without SF and with a MAS of 19mm (MAS recommended for HSC). The analysis presented in Table 3.29 & Figure 3.28.

b. Mixes with SF at any level of replacement or MAS, or mixes with low MAS (< 19 mm) , or both together. The analysis presented in Table 3.30 & Figure 3.29.

Table 3.29 SQRT (f'c) Vs f_t (Regression analysis for Wadi mixes, MAS = 19mm without SF)

Mix#	SQRT (f'c) @ 28-d	f_t MPa @ 28-d
4W	6.29	2.99
3W	6.59	3.06
2W	6.72	3.26
1W	6.96	3.32

Table 3.30 SQRT (f'c) Vs f_t (Regression analysis for Wadi mixes, MAS = 19mm with SF, or MAS = 12.5mm(<19mm) with or without SF)

Mix#	SQRT (f'c) @ 28-d	f_t MPa @ 28-d
1SW	6.89	4.53
2SW	7.82	4.72
3SW	7.80	4.94
5W	6.85	5.00
6W	9.38	6.25

Discussion of Regression Analysis:

1. For Basalt mixes the linear trend in Figure 3.27 outcomes the following relation between splitting tensile strength and compressive strength which is:

   $f_t = 0.72$ SQRT(f'c)........... 3.1

   , which is higher than recommended by ACI 363R-93 for normal and high-strength concrete which is:

   $f_t = 0.54$ SQRT(f'c)..................3.2 (ACI 363R-93, 1993)

2. For Wadi mixes without SF and MAS of 19mm, the linear trend in Figure 3.28 outcomes the following relation which is:

   $f_t = 0.47$ SQRT(f'c).....................3.3

   , which is also lower than recommended by ACI 363R-93 for normal and high-strength concrete (Eq. 3.2)

3. For Wadi mixes with SF at any level of replacement or MAS, or mixes with MAS = 12.5 mm (< 19mm) with or without SF, linear trend in Figure 3.29 outcomes the following relation which is:

   $f_t = 0.65$ SQRT(f'c) 3.4

   ,which is also higher than recommended by ACI 363R-93 for normal and high-strength concrete (Eq 3.2).

3.3 Flexure Tensile Strength of HSC:

A 36 Standard 100*100*500 mm test prisms have been tested with third point loading to investigate flexure tensile strength by means of modulus of rupture for HSC according to ASTM C78 at the age of 28-days, for different A/C, aggregates type, MAS, & SF%.
And to investigate the relation between SQRT of f'c and modulus of rupture based on linear regression analysis.

3.3.1 Effect of A/C & Aggregate type on Modulus of Rupture (f_r) for HSC:

The effect of varying A/C over modulus of rupture for both Basalt and Wadi mixes are presented in Tables 3.31, & 3.32.

Table 3.31 f'c at 28-d Vs f_r at 28-d (A/C effect for Basalt mixes).

Mix#	A/C	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
1B	3.95	60.0	6.43	10.7
2B	3.65	62.3	6.89	11.0
3B	3.14	51.6	6.04	11.7
4B	2.71	42.8	5.83	13.6

Table 3.32 f'c at 28-d Vs f_r at 28-d (A/C effect for Wadi mixes).

Mix#	A/C	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
1W	3.71	48.4	5.81	12.0
2W	3.35	45.1	5.20	11.5
3W	2.88	43.4	5.04	11.6
4W	2.48	39.6	4.98	12.6

1. Similar to splitting tensile strength, modulus of rupture is directly proportional to compressive strength (higher compressive strength gives a higher modulus of rupture), hence the higher A/C up to optimum value gives a higher modulus of rupture (sec 3.1.1 & 3.2.1).

2. Results shows that modulus of rupture is higher for Basalt than Wadi mixes; (11-33%) increase which correspond to (8-38%) for increase in compressive strength this leads to the following conclusions:

a. Note that (8-38%) increase in strength of Basalt mixes gives (65-70%) increase in splitting tensile strength versus (11-33%) increase in modulus of rupture, which means that splitting tensile strength is more sensitive than modulus of rupture for change of type, shape, & surface texture of aggregates (tripled), see sec 3.2.1.

b. Modulus of rupture is strongly related to compressive strength (nearly the per cent increase in compressive strength versus by the same per cent increase in modulus of rupture).

c. There is a positive advantage of adopting modulus of rupture as a uniform representative to indicate tensile strength according to the results of compressive strength what ever the MAS or SF% level for the same type of aggregates.

3.3.2 Effect of MAS on Modulus of rupture (f_r) for HSC:

The MAS in mix 2B, & 2W which stated as reference mixes, was limited to 9.5 & 12.5mm respectively to investigate the MAS effect on modulus of rupture, the results are presented in Tables 3.33, & 3.34.

Table 3.33 f'c at 28-d Vs f_r at 28-d (MAS effect for Basalt mixes).

Mix#	MAS mm	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
2B	19	62.3	6.89	11.0
5B	9.5	66.5	6.93	10.0

Table 3.34 f'c at 28-d Vs f_r at 28-d (MAS effect for Wadi mixes).

Mix#	MAS mm	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
2W	19	45.1	5.20	11.5
5W	12.5	47.0	6.43	13.7

1. Improving the mechanical properties induced by lowering MAS as discussed in sec 3.1.2 have the same effect on modulus of rupture (higher modulus of rupture with lowering MAS), and again relatively Wadi mixes shows more improvement than Basalt mixes.

2. Wadi mixes show less significant increase in flexure tensile strength represented by modulus of rupture versus splitting tensile strength (24%: 53%, doubled) when decreasing MAS, this indicate that splitting tensile strength is more sensitive for reducing MAS than modulus of rupture (doubled) for Wadi mixes, see sec 3.2.2.

3.3.3 Effect of SF on Modulus of Rupture (f_r) of HSC:

Mixes 2B & 2W are stated as reference and retested for modulus of rupture at 28-days age but using the levels 5, 10, & 15% partial replacement by weight of OPC. The results are presented in Tables 3.35, & 3.36.

Table 3.35 f'c at 28-d Vs f_r at 28-d (SF effect for Basalt mixes).

Mix#	SF%	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
2B	0.0	62.3	6.89	11.0
1SB	5.0	65.6	7.04	10.7
2SB	10.0	77.4	7.8	10.0
3SB	15.0	75.6	7.56	10.0

Table 3.36 f'c at 28-d Vs f_r at 28-d (SF effect for Wadi mixes).

Mix#	SF%	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
2W	0.0	45.1	5.20	11.5
1SW	5.0	47.5	5.42	11.4
2SW	10.0	61.1	5.66	9.3
3SW	15.0	60.8	5.50	9.0

1. According to improving of micro-structure of concrete provided from using SF discussed in sec 3.1.3, test results shows a higher modulus of rupture with increasing SF% level up to optimum value which after no further increase in strength could gained with more SF% (see sec 3.1.3).

2. Wadi mixes show less significant improvement in modulus of rupture versus splitting tensile strength when using SF ([4-9%] Vs [39-52%]), this supports that the sensitivity of splitting tensile strength is more than modulus of rupture for any improvement introduced to concrete micro-structure.

3.3.4 Toward a Very-High Modulus of Rupture Value:

The mixes presented in sec 3.1.4 have been tested for modulus of rupture at 28-days to get a highest possible value, according to test results. The results are presented in Tables 3.37, & 3.38.

Table 3.37 f'c at 28-d Vs f_r at 28-d (The highest obtained value for Basalt mixes)

Mix Details	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
6B W/C+SF=0.23 A/C+SF = 4.0 MAS = 9.5mm	94.2	8.41	8.9

Table 3.38 f'c at 28-d Vs f_r at 28-d(The highest obtained value for Wadi mixes)

Mix Details	f'c MPa 28-d	f_r MPa 28-d	f_r/f'c %
6W W/C+SF=0.23 A/C+SF = 3.68 MAS =12.5mm	88.0	7.50	8.5

- For all conditions the ratio Fr/F'c decreased with increasing compressive strength and it was ranged from (8.5-13.7%) for Wadi mixes, & (8.9-13.6%) for Basalt mixes, which shows the same potential for Basalt and Wadi mixes into flexure strength/compressive strength ratio.

- Modulus of rupture shows always a higher ratio and for all conditions and type of materials, more than splitting tensile strength this can be shown in the following table:

Comparison between the range of f_t/f'c & f_r/f'c , for both types of mixes under all conditions

	f_t/f'c Range	f_r/f'c Range
Basalt Mixes	7.1-11.6 %	8.9-13.6 %
Wadi Mixes	7.1-10.6 %	8.5-13.7 %

3.3.5 Regression Analysis f_r Versus f'c for HSC:

_A regression analysis have been done to draw a relation between SQRT of f'c at 28-days and modulus of rupture f_r & its summarized as following:

1. It has been found that Basalt mixes likely to have a same relation between SQRT of f'c and modulus of rupture f_r at 28-days age, what-ever the A/C, MAS, & SF%.

2. Test results discussion in sec 3.3.1 and 3.3.3,show that for Wadi mixes the same relation between F'c & Fr could be held what-ever A/C, MAS, & SF%.

3. The relation and for both types of materials are directly proportioned to compressive strength & the effect of varying the mix ingredient properties to enhance micro-structure of concrete could be minimized if it is related to the value of compressive strength for the same type of aggregates used.

4. That is opposite to the case which founded in studying splitting tensile strength for wadi materials which states that the behavior of Wadi mixes into splitting tensile strength affected significantly when the properties of mix ingredient and micro-structure of concrete varied i.e. (lowering MAS, using SF).

5. Basalt aggregate mixes shows a higher relative proportioning with SQRT of f'c than Wadi aggregate mixes.

- The regression analysis are presented in Tables 3.39, & 3.40, Figures 3.30, & 3.31 which resulted in the following:

Table 3.39 SQRT (f'c) Vs f_r (Regression analysis for Basalt mixes)

Mix#	SQRT (f'c) @ 28-days	f_r MPa @ 28-days
4B	6.54	5.83
3B	7.19	6.04
1B	7.75	6.43
2B	7.89	6.89
1SB	8.10	7.04
5B	8.15	6.93
3SB	8.69	7.56
2SB	8.80	7.80
6B	9.70	8.41

Table 3.40 SQRT (f'c) Vs f_r (Regression analysis for Wadi mixes)

Mix#	SQRT (f'c) @ 28-days	f_r MPa @ 28-days
4W	6.29	4.98
3W	6.59	5.04
2W	6.72	5.2
5W	6.85	6.43
1SW	6.89	5.42
1W	6.96	5.81
3SW	7.80	5.5
2SW	7.82	5.66
6W	9.38	7.5

Discussion of regression analysis

1. For Basalt mixes the linear trend in Figure 3.30 outcomes the following relation between SQRT of f'c and Fr which is:

f_r = 0.87 SQRT(f'c).......... 3.5

, which is lower than recommended by ACI 363R-93 for high-strength concrete which is:

f_r = 0.94 SQRT(f'c).............3.6 (ACI 363R-93, 1993)

, and greater than recommended ACI 381-95 for normal strength concrete which is:

$f_r = 0.7$ SQRT(f'c) ....3.7 (ACI 318-95, 1995)

2. For Wadi mixes the linear trend in Figure 3.30 outcomes the following relation between SQRT of f'c & f_r which is:

   $f_r = 0.8$ SQRT (f'c) 3.8

   , which is also less than recommended by ACI 363R-93 (Eq 3.6) for high-strength concrete, and greater than recommended by ACI 318-95 for normal strength concrete (Eq 3.7).

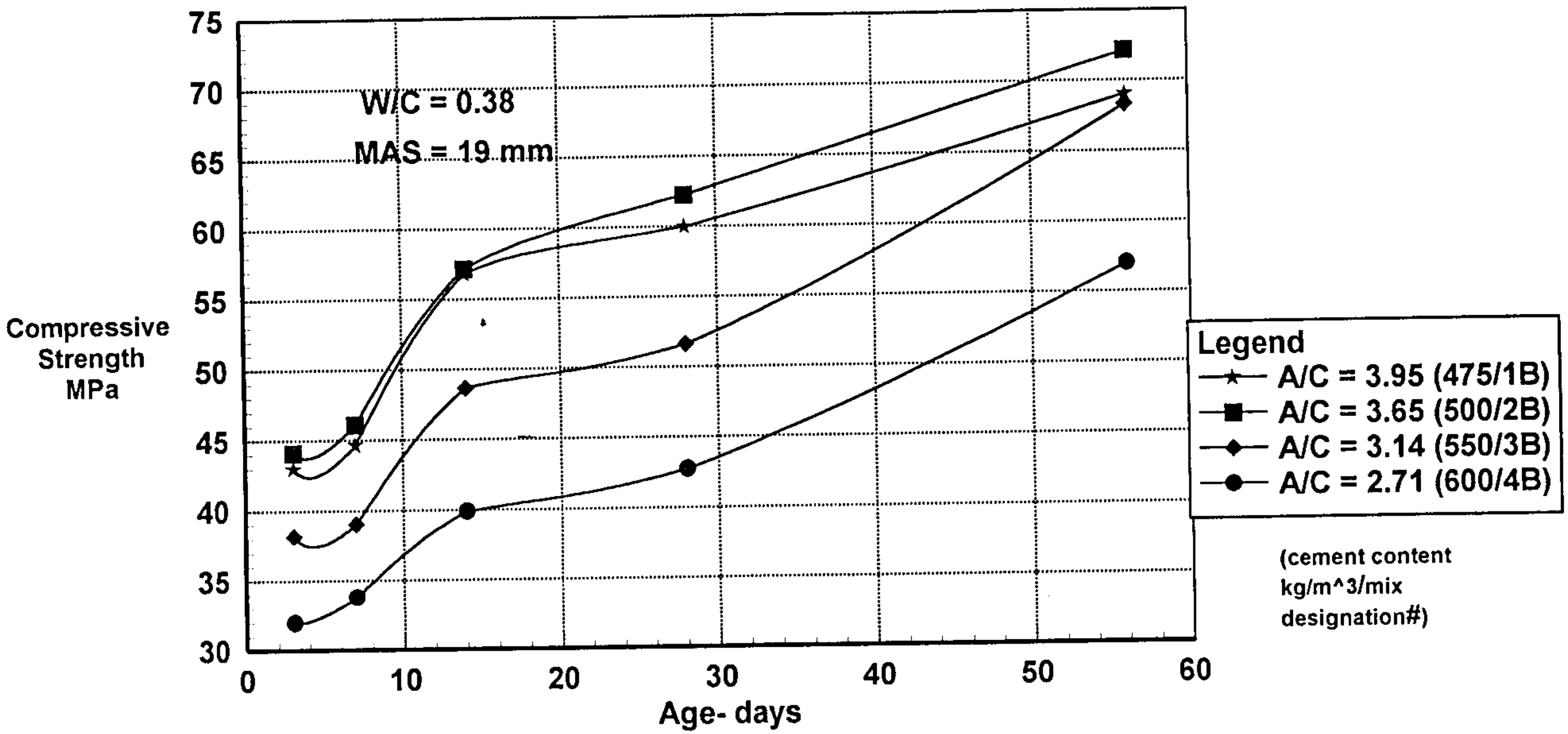

Figure 3.1 compressive strength gain with time for different A/C basalt mixes proportions.

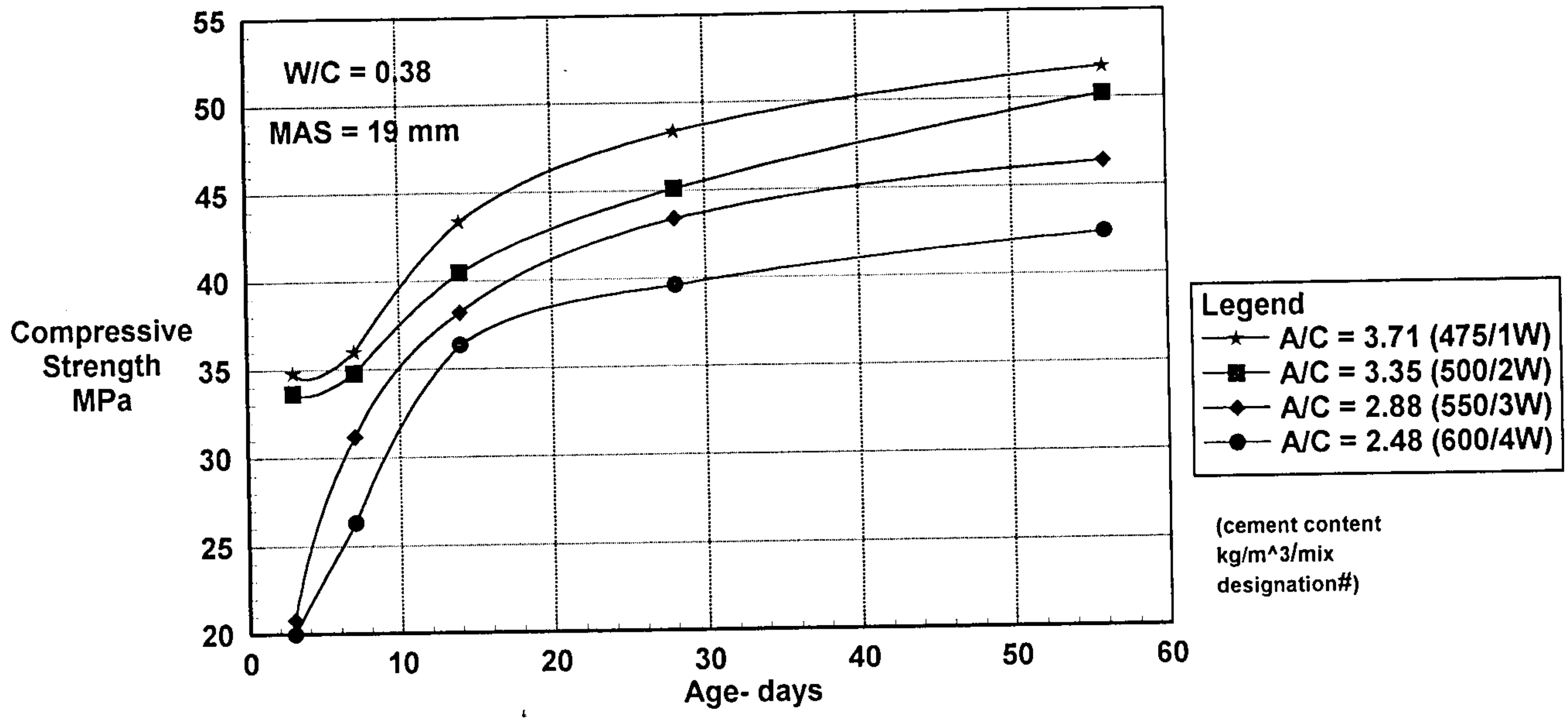

Figure 3.2 compressive strength gain with time for different A/C Wadi mixes proportions.

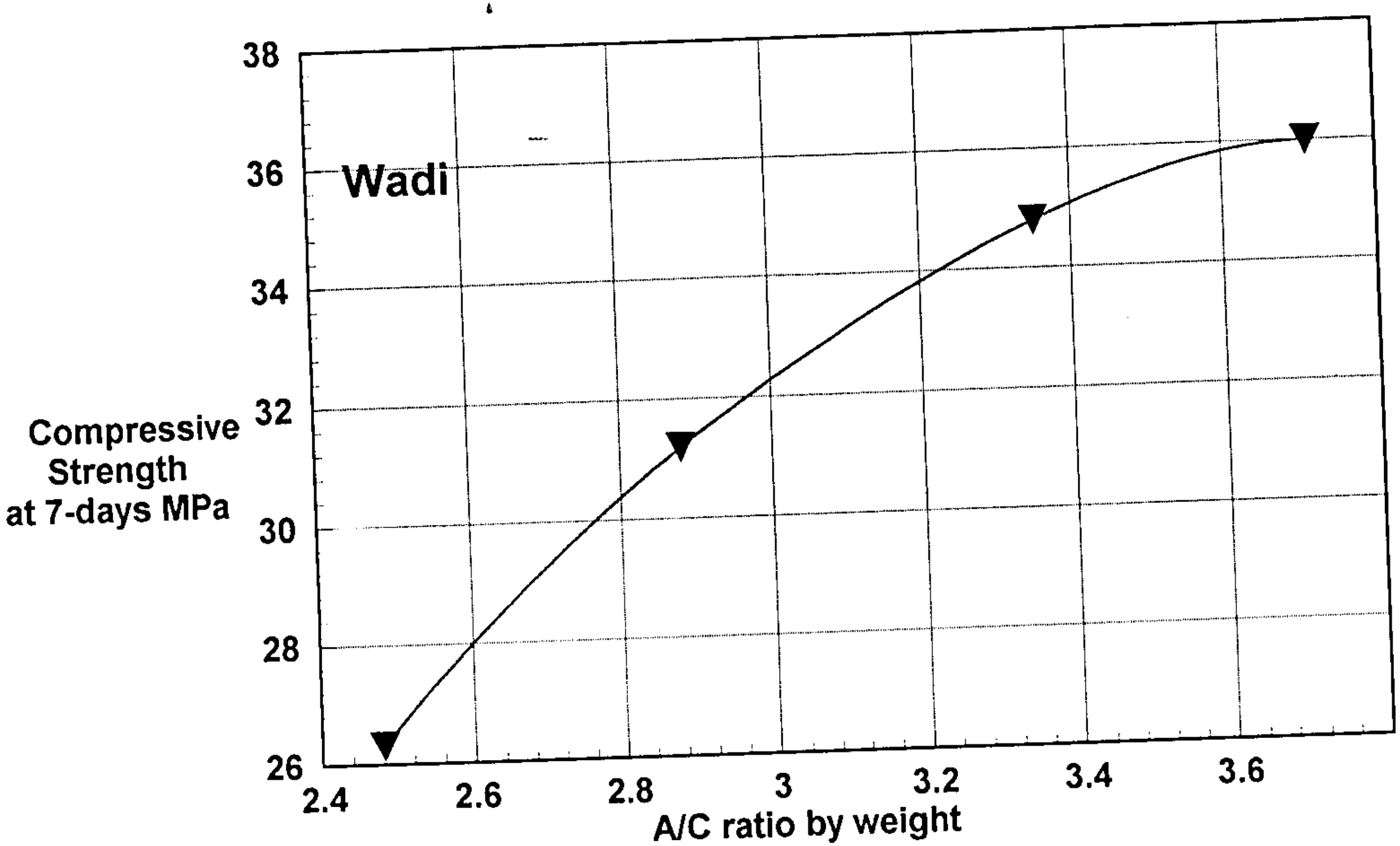

Figure 3.3 A/C ratio by weight Vs Average Compressive Strength at 7-days For Wadi materials mixes

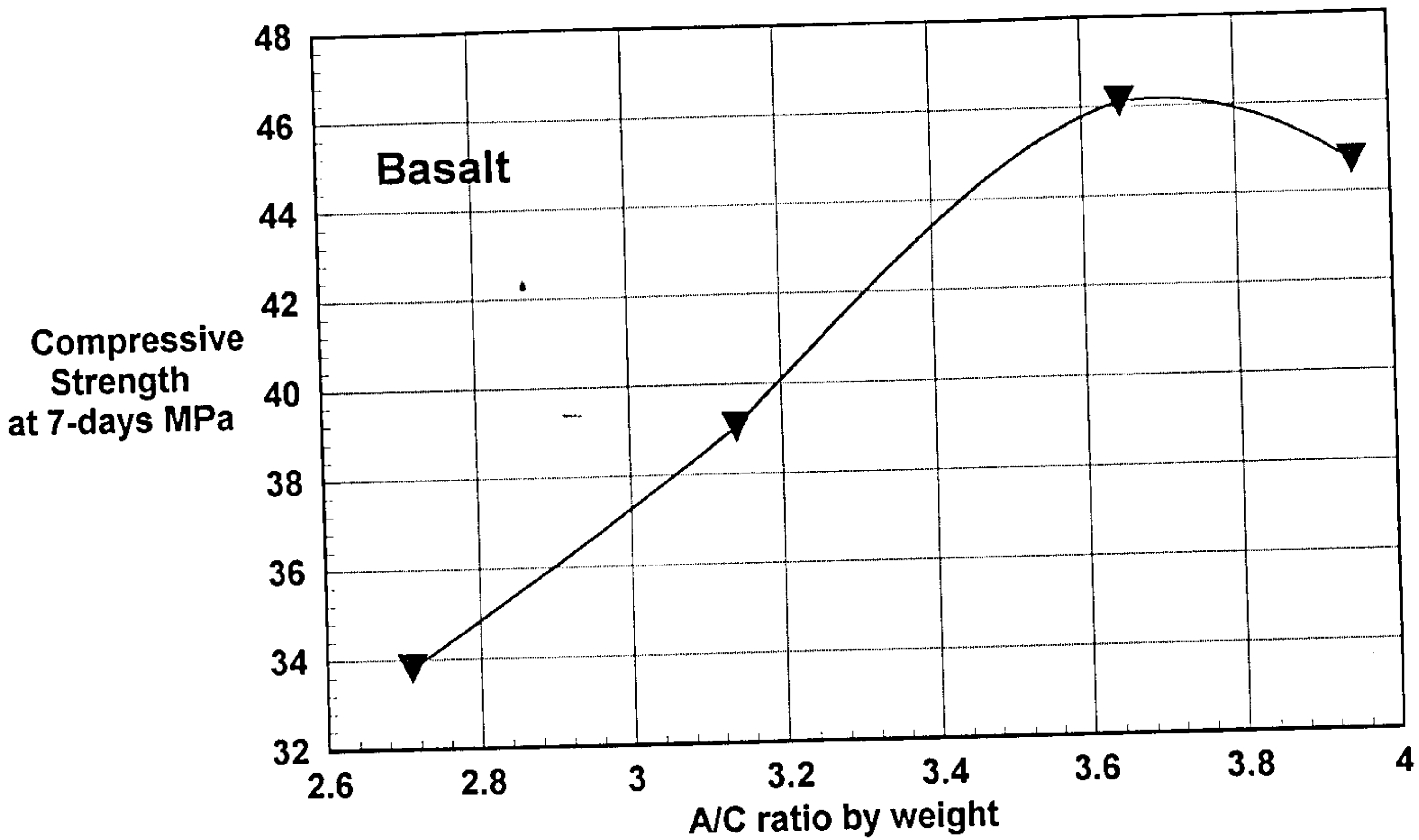

Figure 3.4 A/C ratio by weight Vs Average Compressive Strength at 7-days For Basalt materials mixes

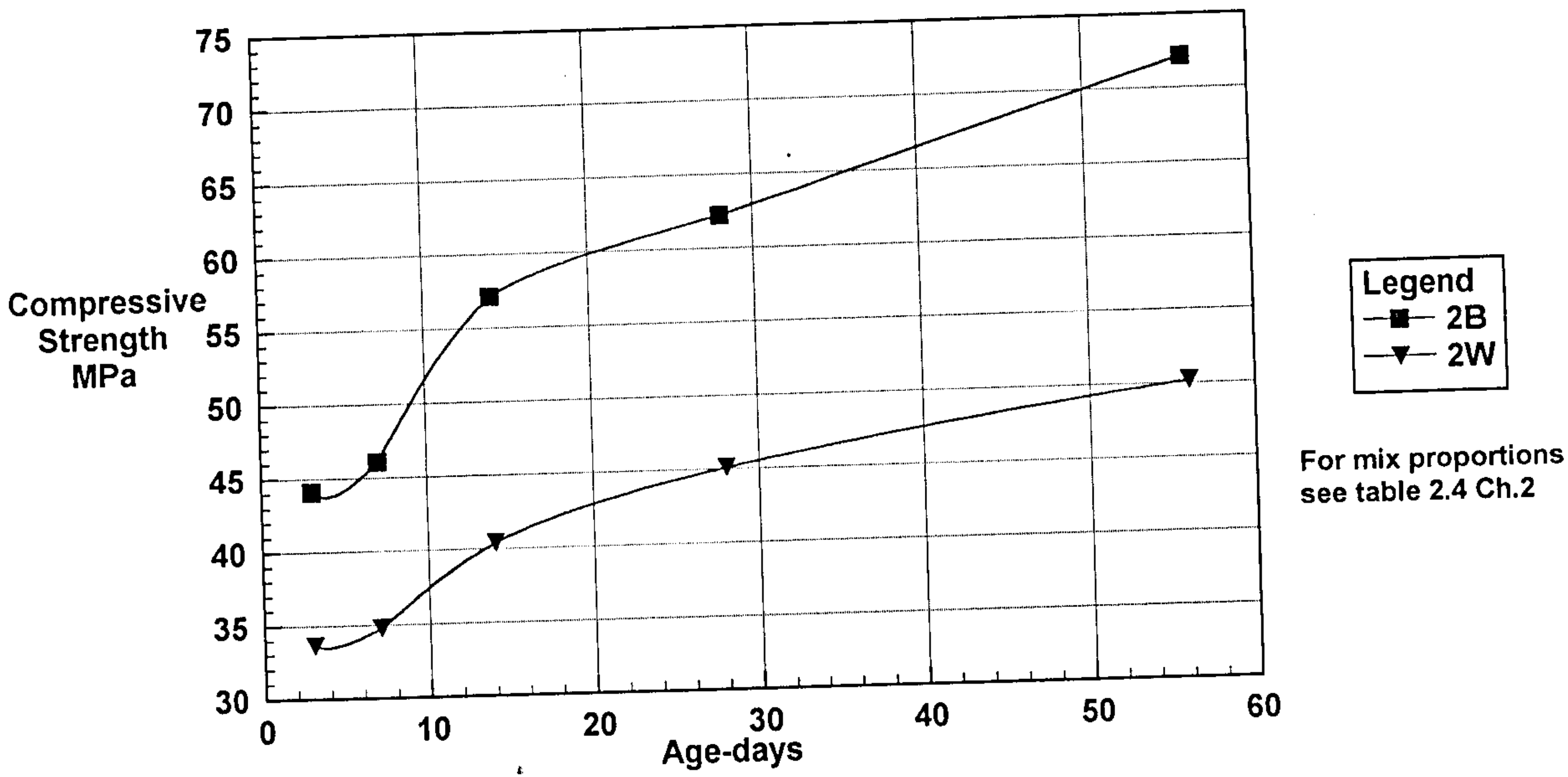

Figure 3.6 Effect of Different Source Type of Aggregates on Compressive Strength

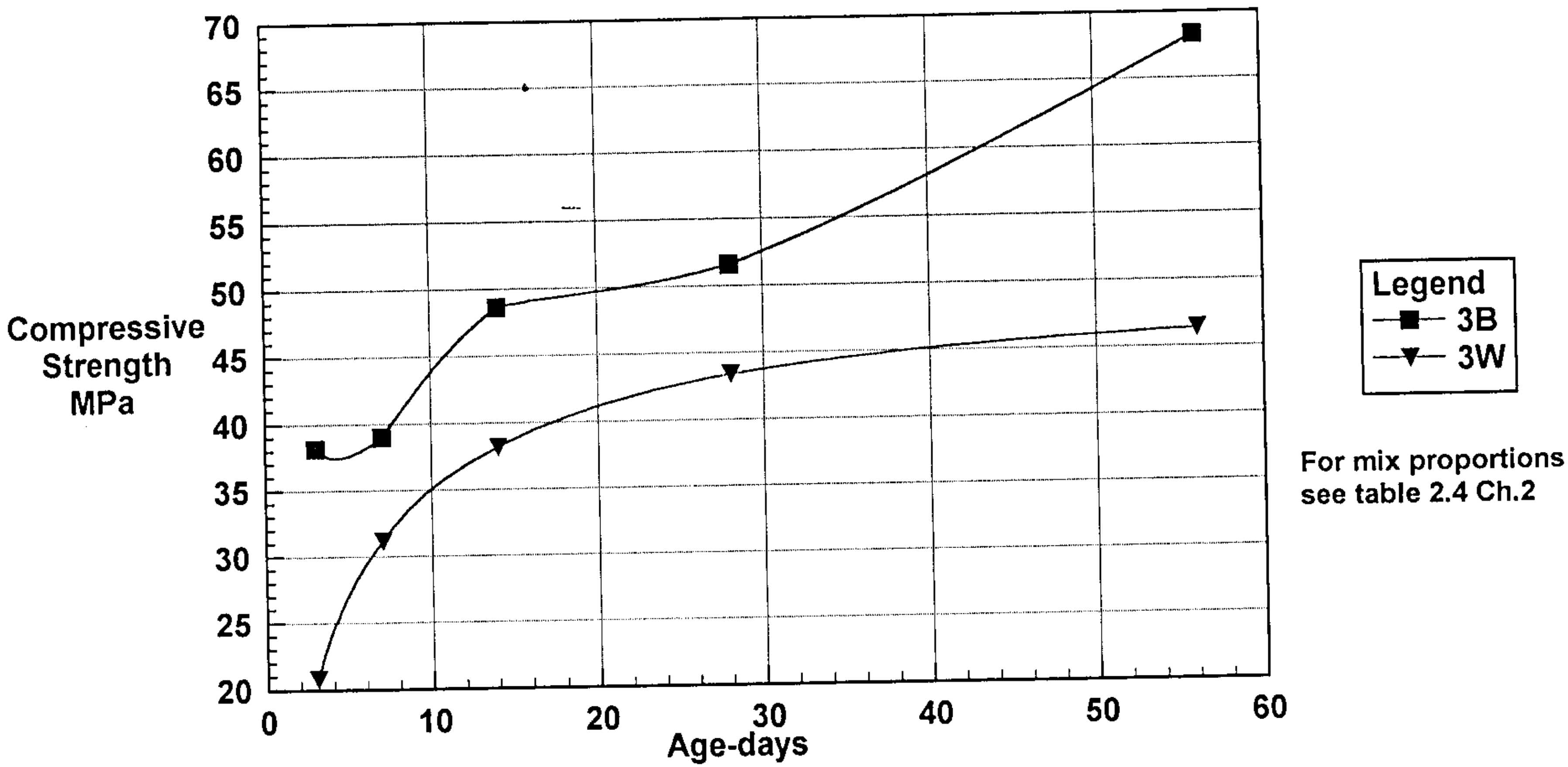

Figure 3.7 Effect of Different Source Type of Aggregates on Compressive Strength

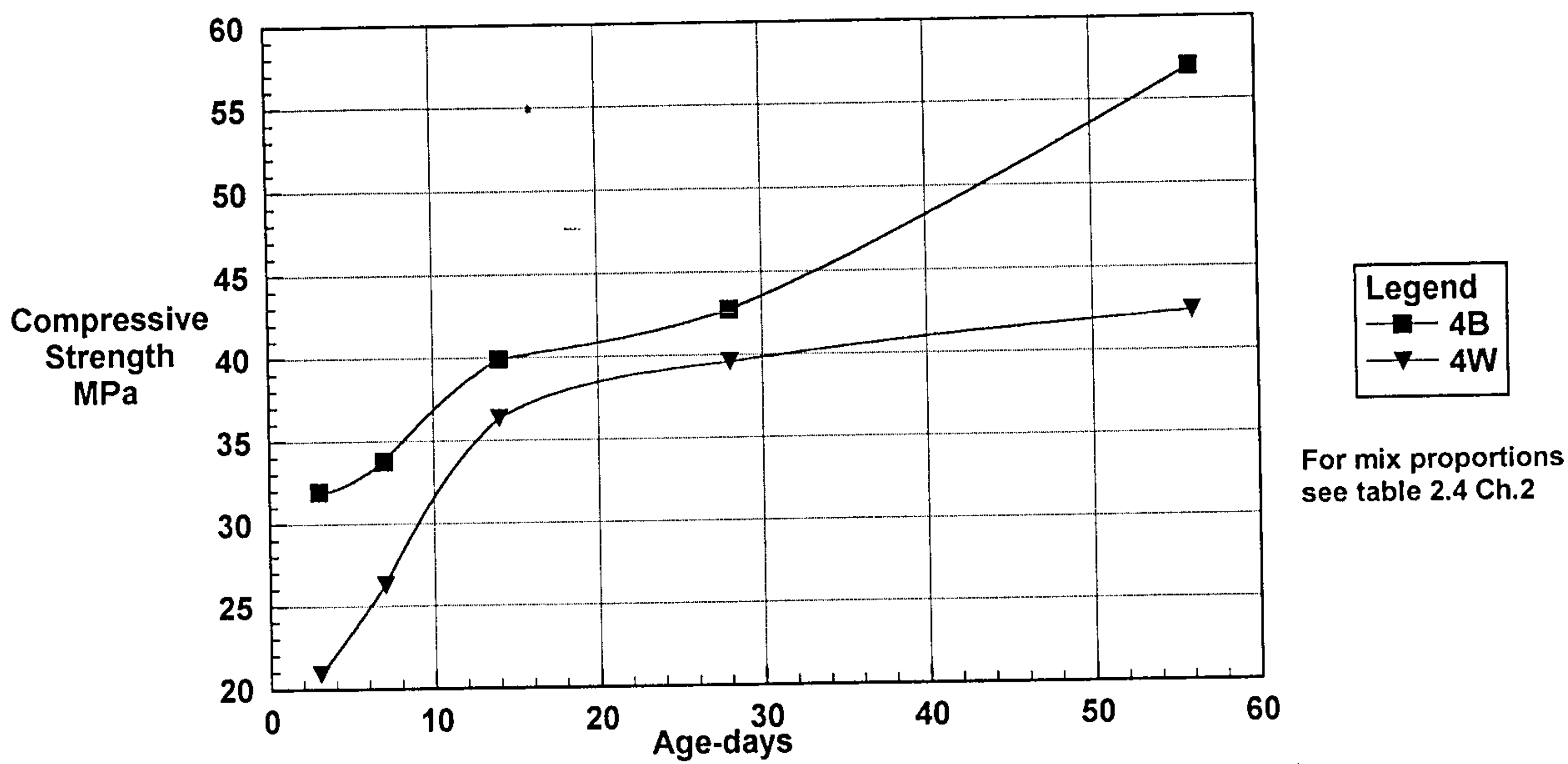

Figure 3.8 Effect of Different Source Type of Aggregates on Compressive Strength

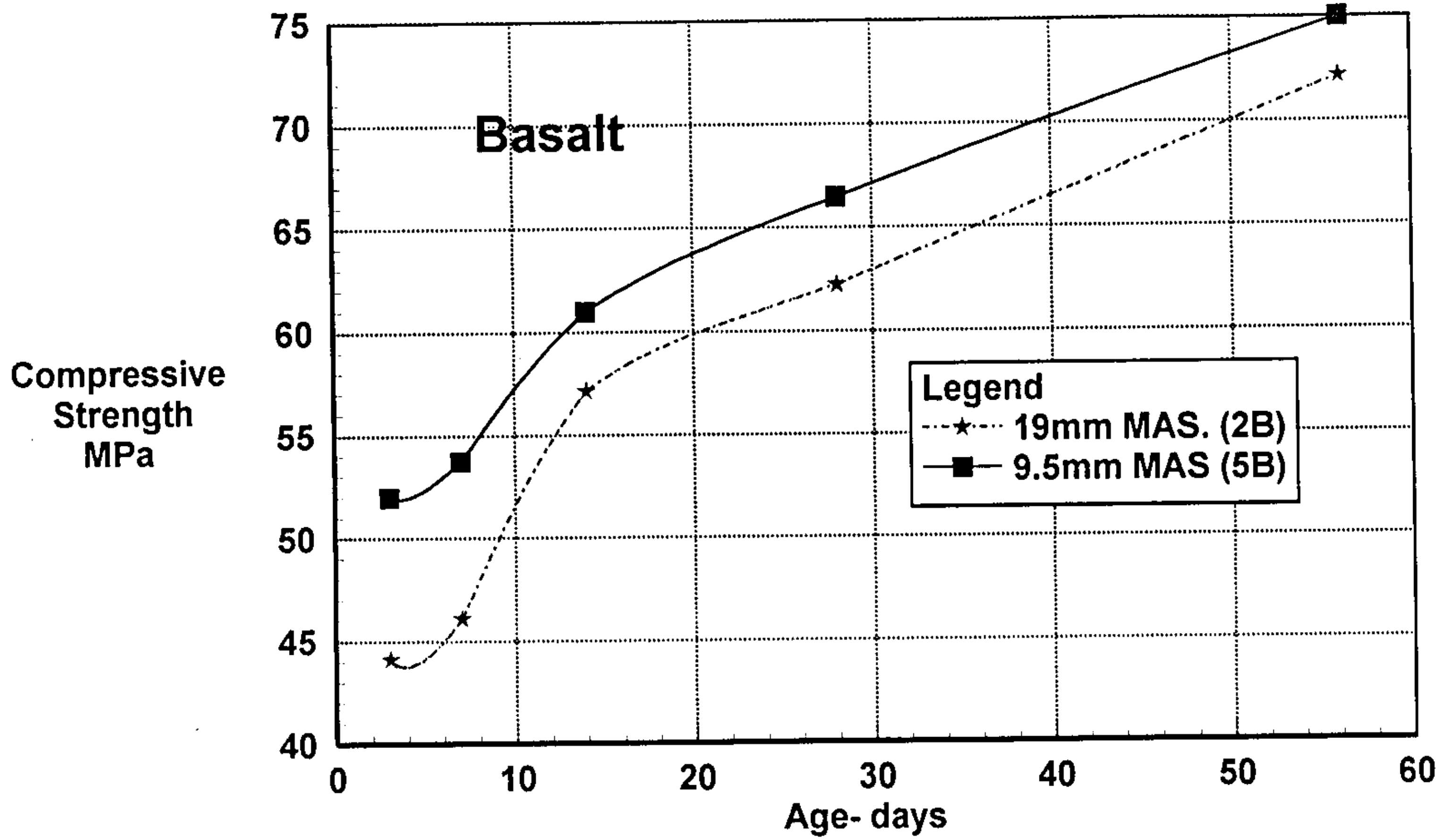

Figure 3.9 Effect of maximum Agg. Size. (MAS) on strength for Basalt Agg.

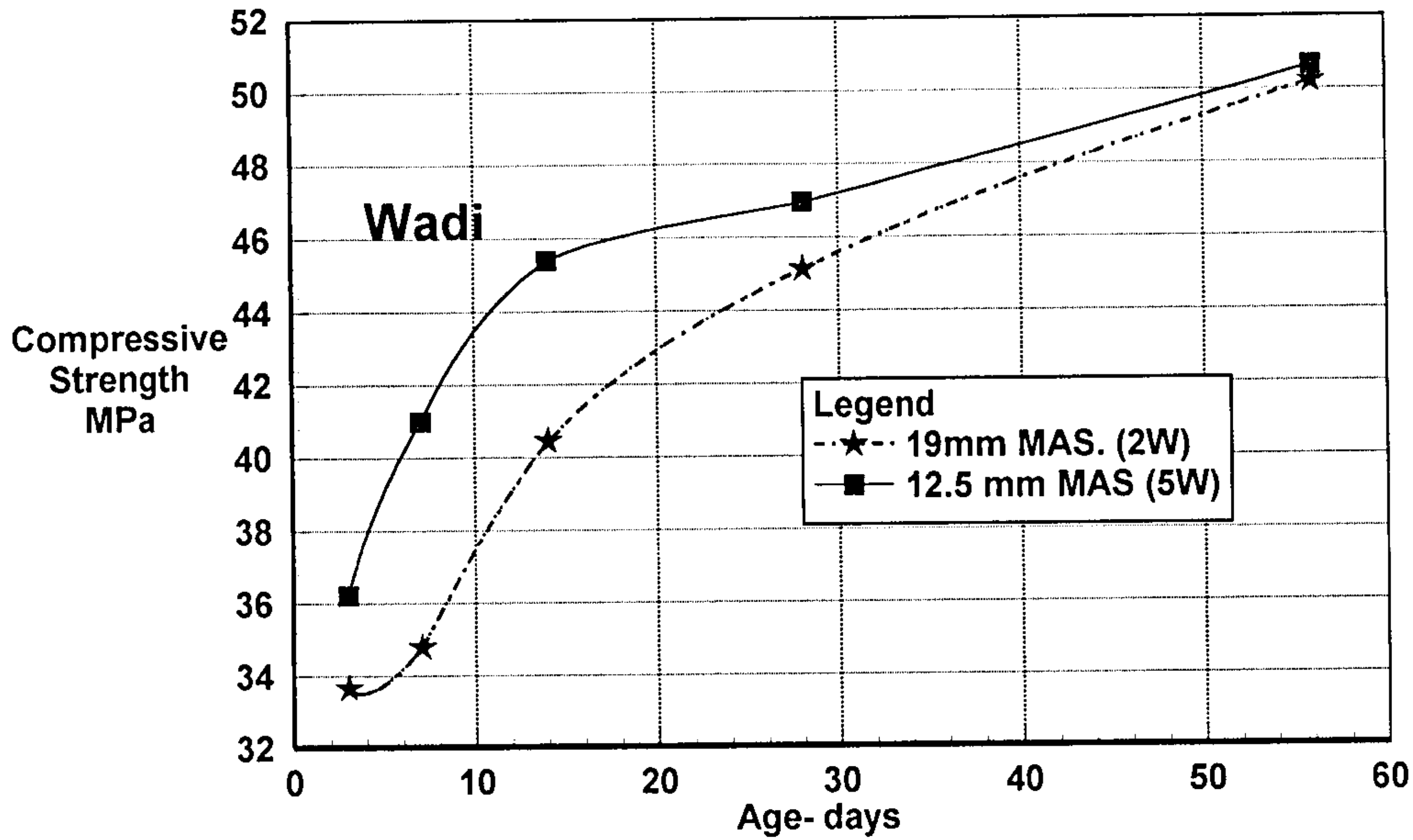

Figure 3.10 Effect of maximum Agg. Size. (MAS) on strength for Wadi Agg.

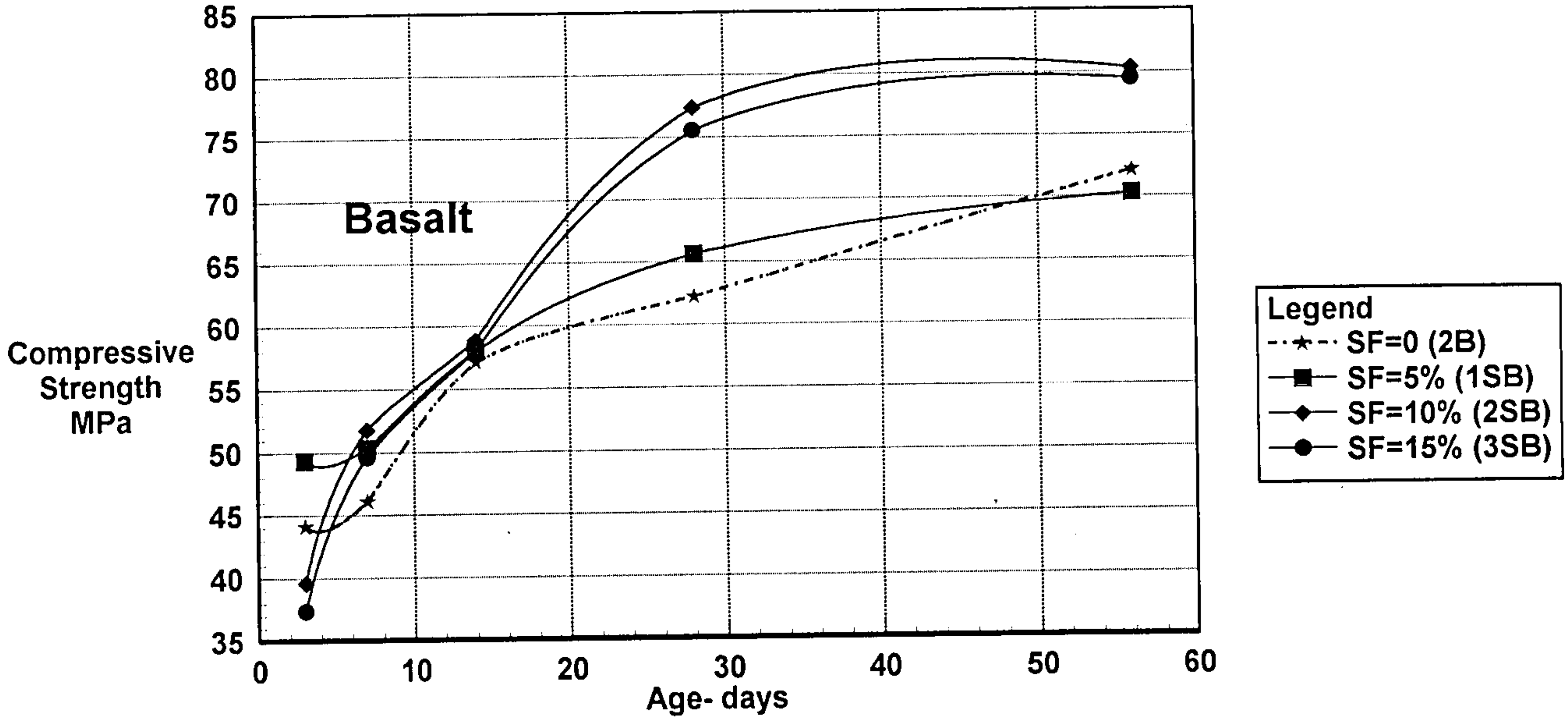

Figure 3.11 Effect of silica fume (SF) on strength for basalt.

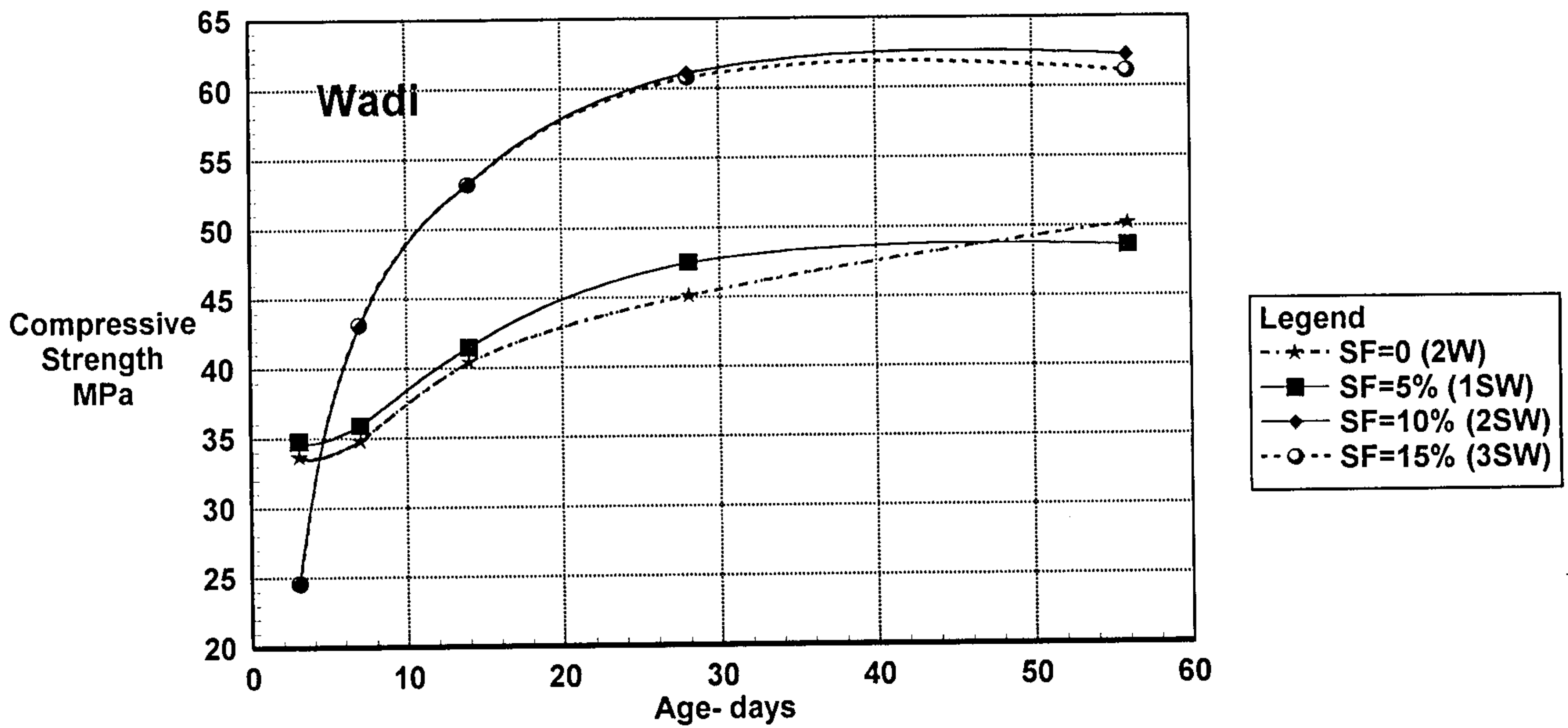

Figure 3.12 Effect of silica fume (SF) on strength for Wadi Agg.

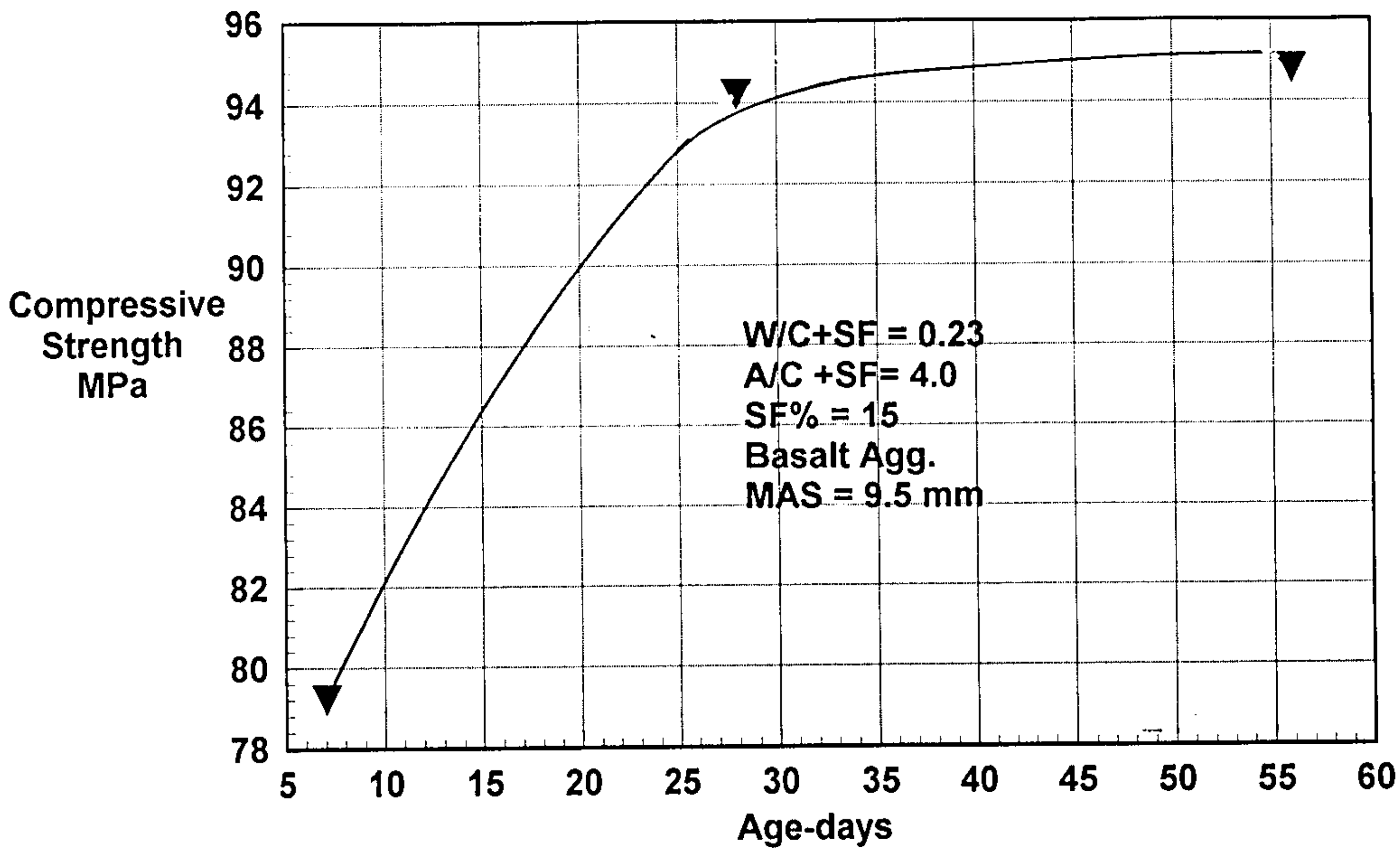

Figure 3.13 Strength gain with time for a Very High-Strength Mix No.6B

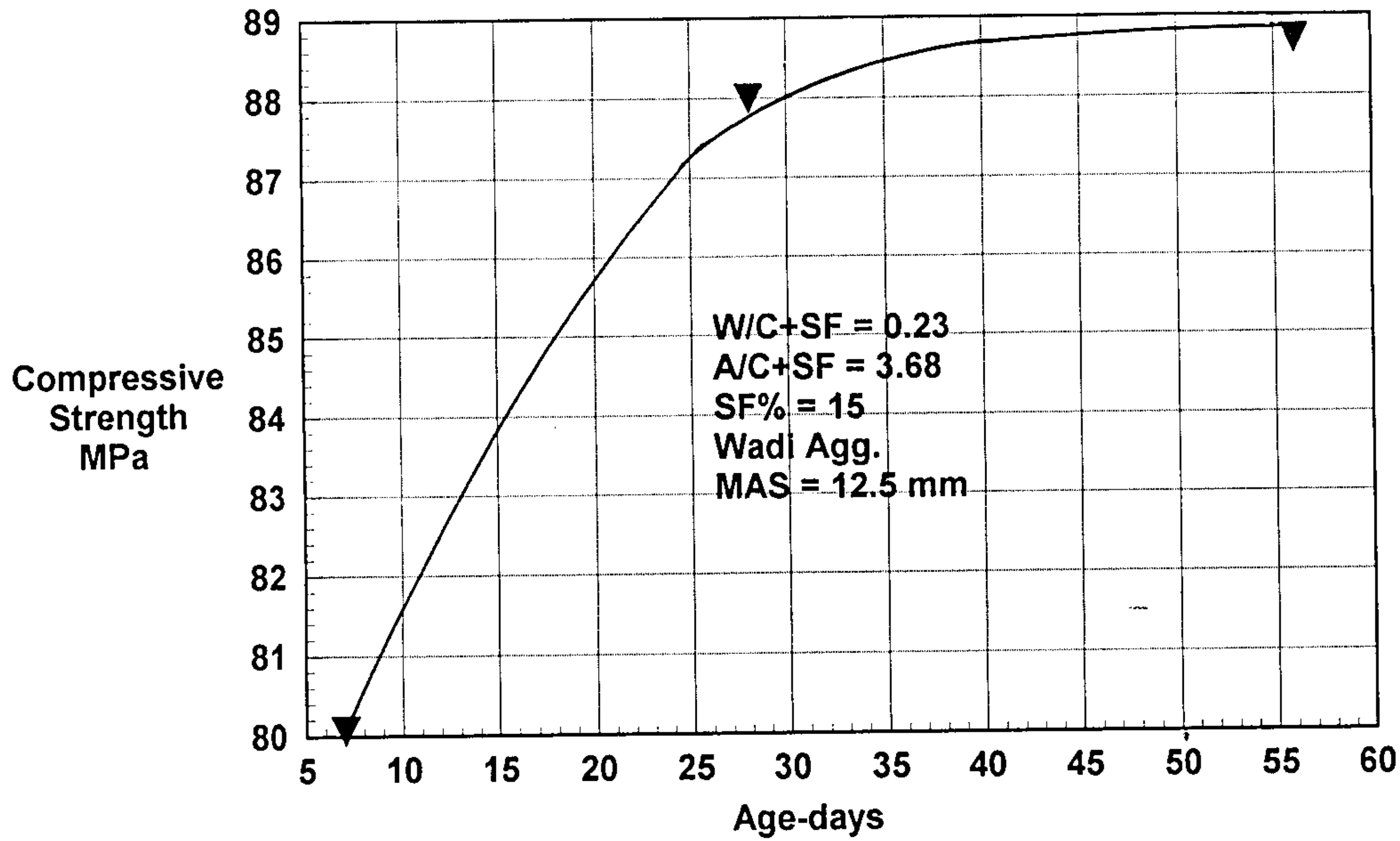

Figure 3.14 Strength gain with time for a Very High-Strength Mix No.6W

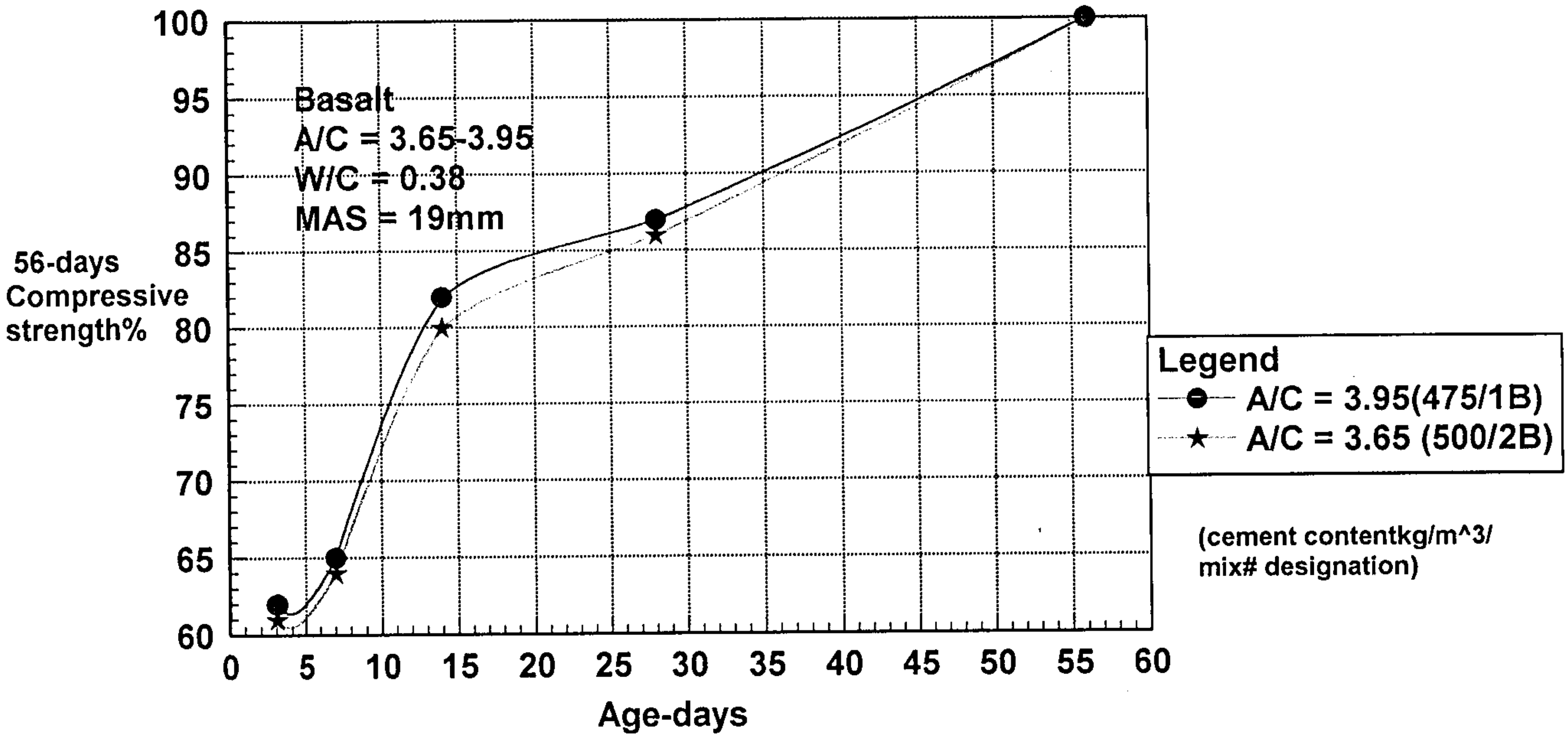

Figure 3.15 Compressive Strength gain with time as a percent of 56-days strength

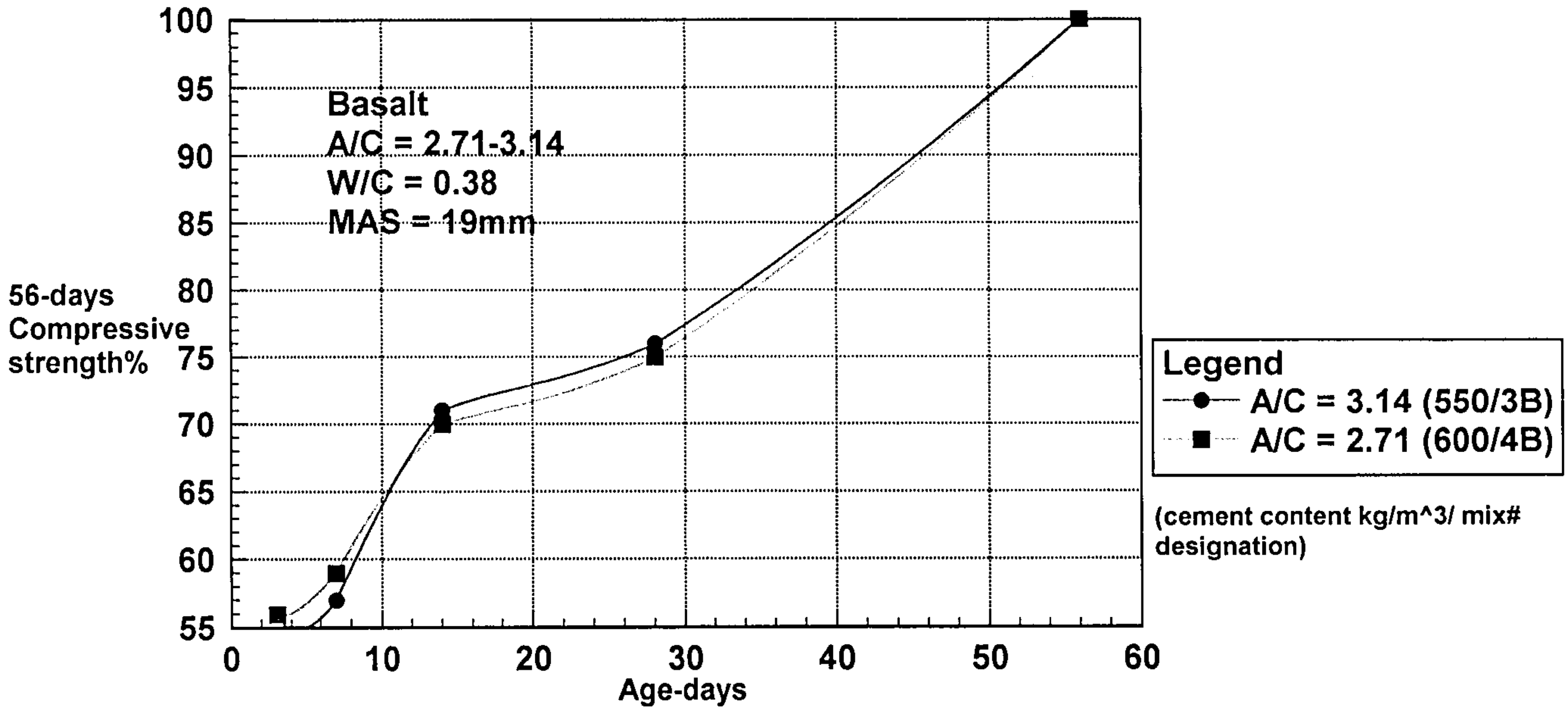

Figure 3.16 Compressive Strength gain with time as a percent of 56-days strength

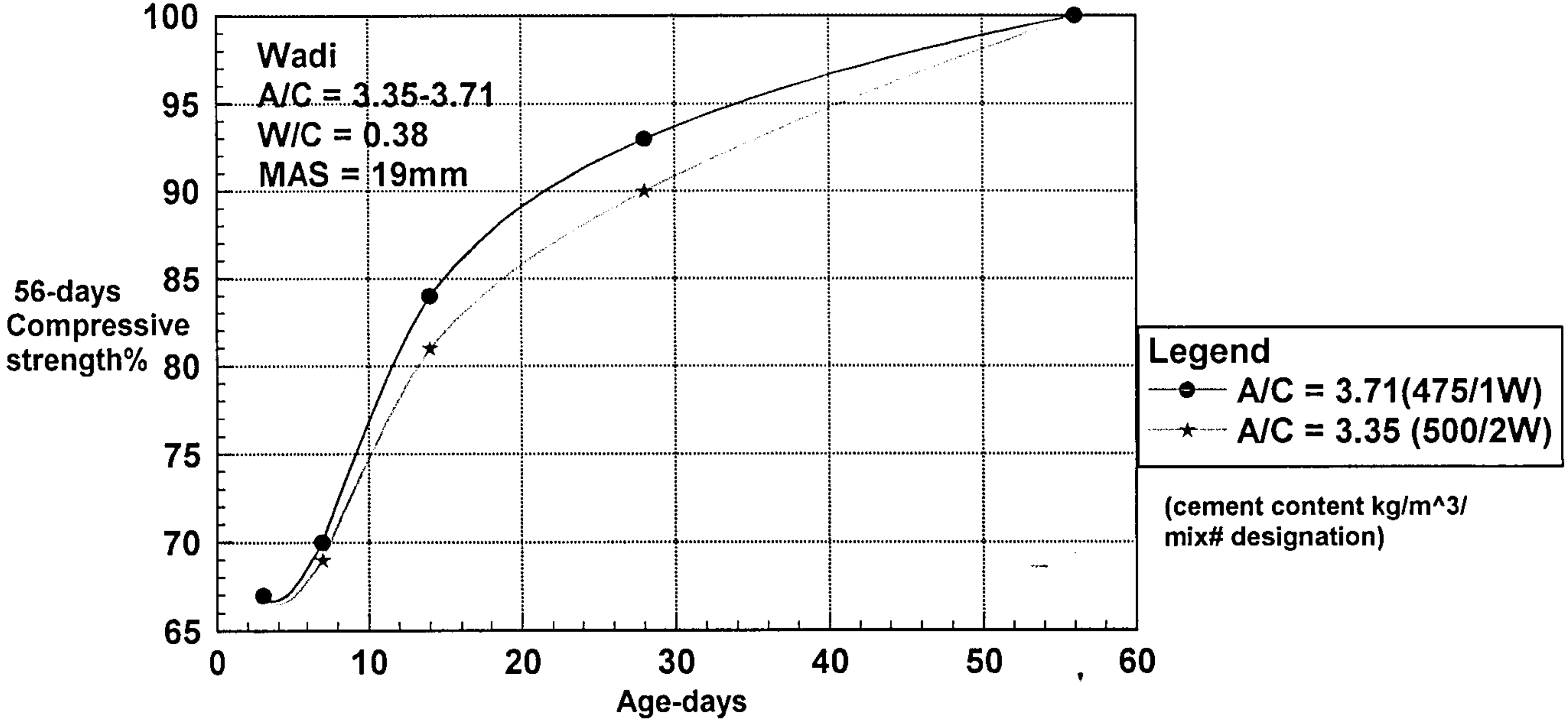

Figure 3.17 Compressive Strength gain with time as a percent of 56-days strength

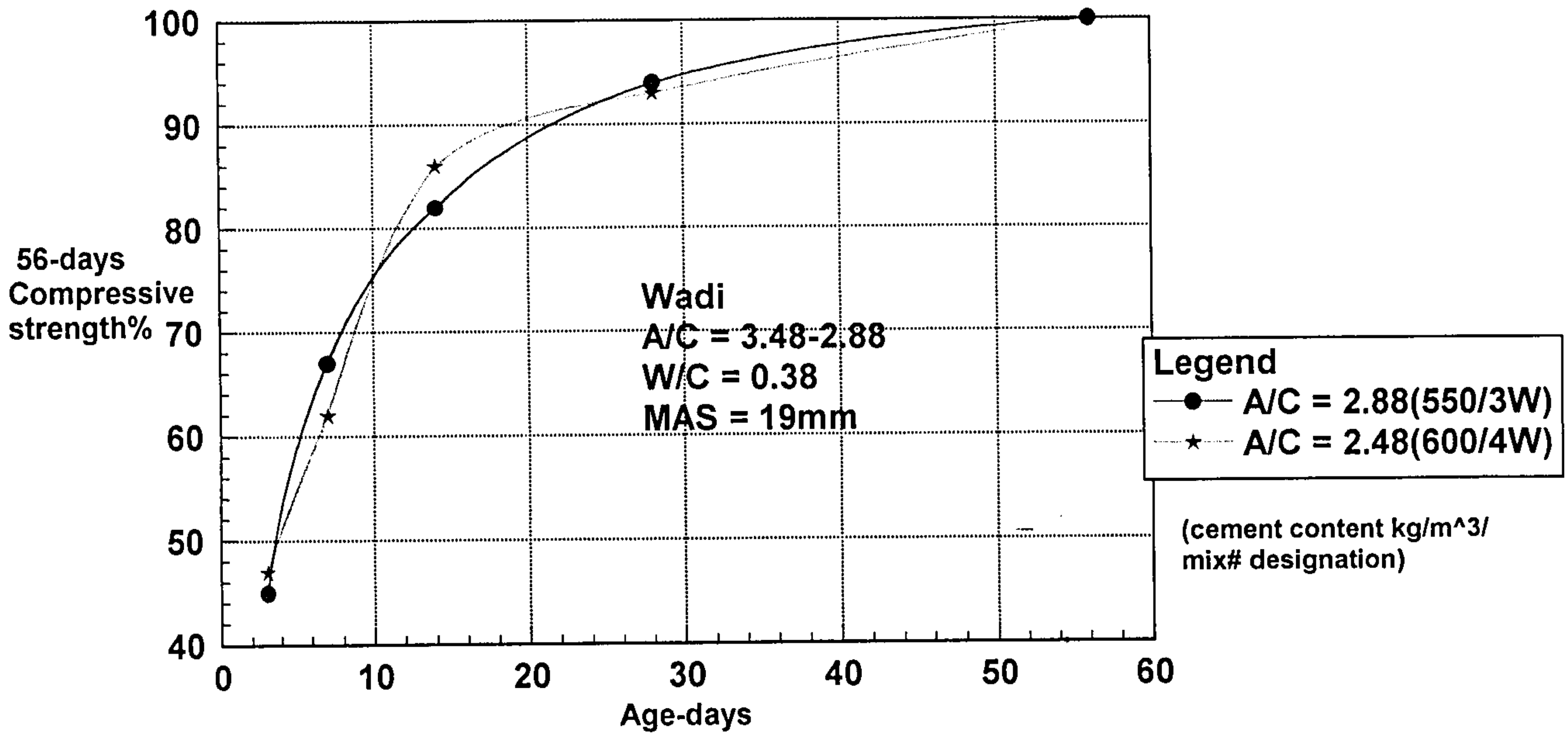

Figure 3.18 Compressive Strength gain with time as a percent of 56-days strength

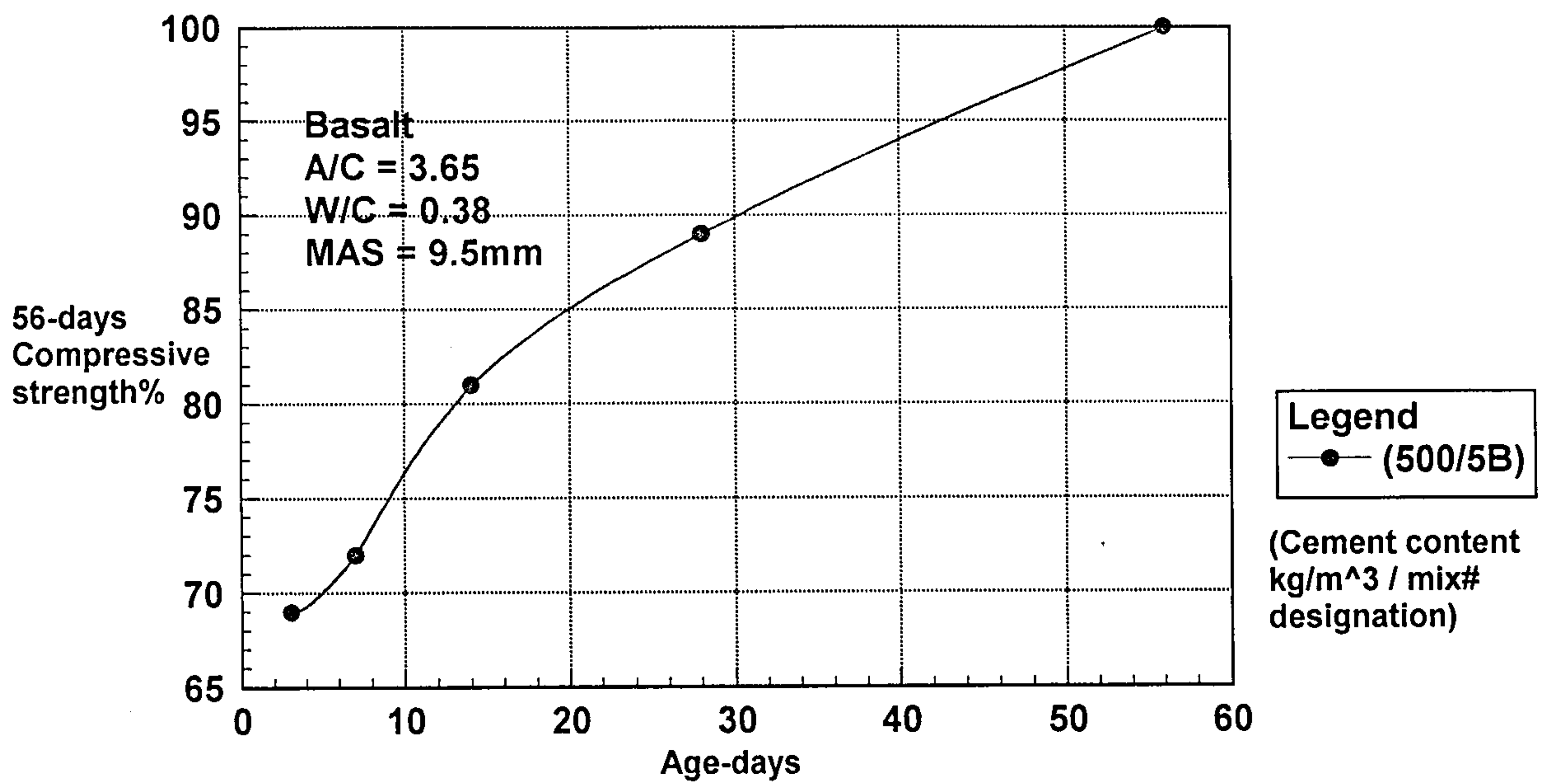

Figure 3.19 Compressive Strength gain with time as a percent of 56-days strength

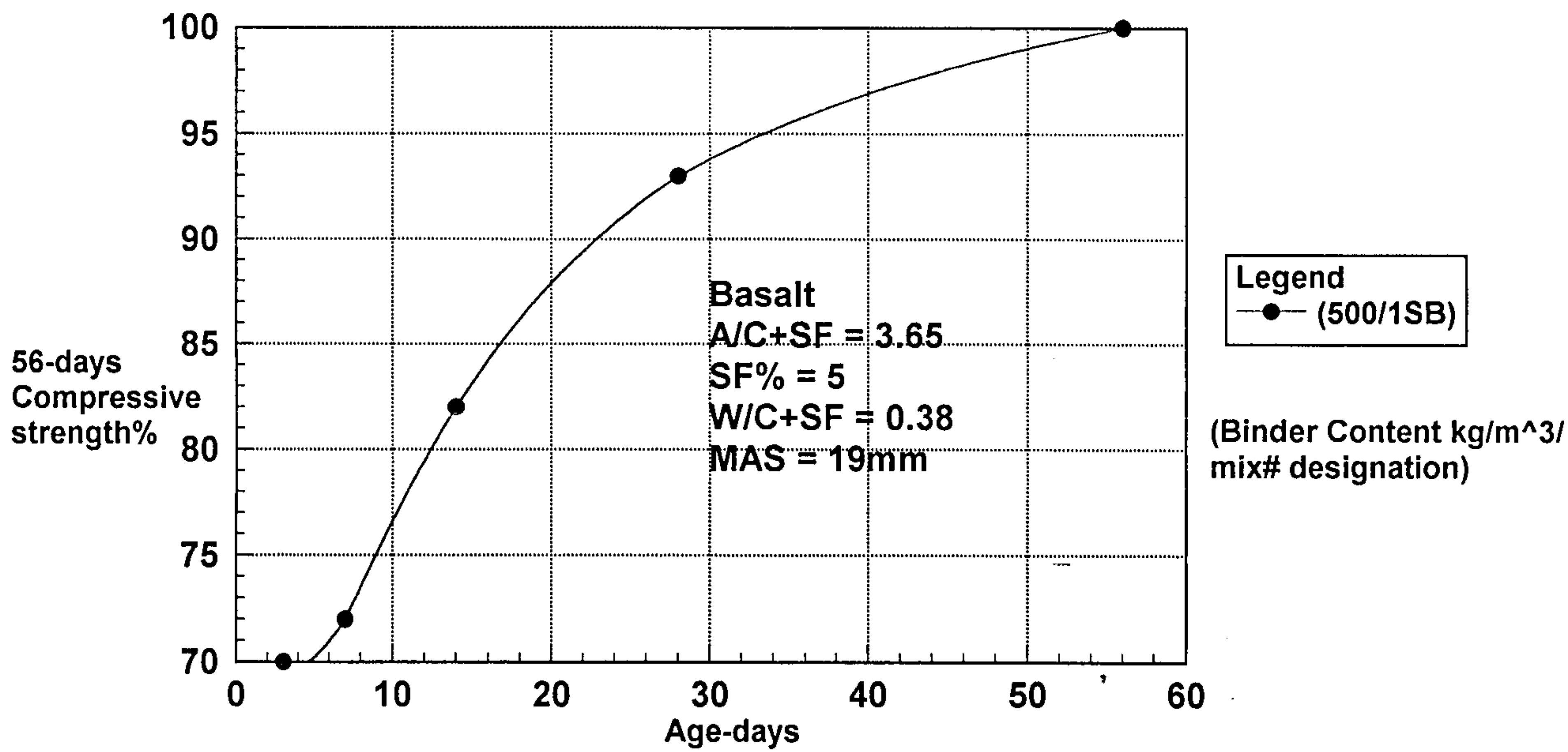

Figure 3.21 Compressive Strength gain with time as a percent of 56-days strength

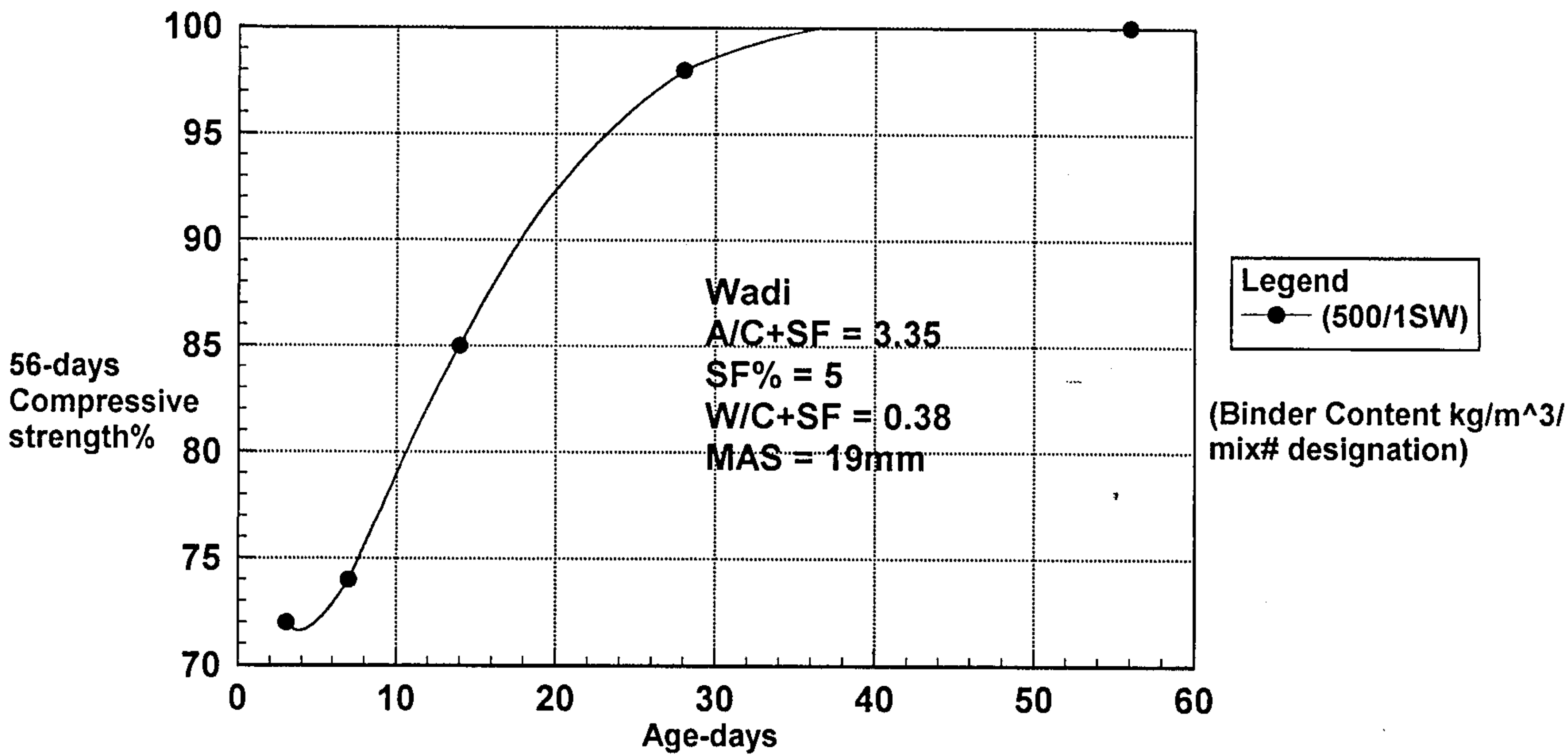

Figure 3.22 Compressive Strength gain with time as a percent of 56-days strength

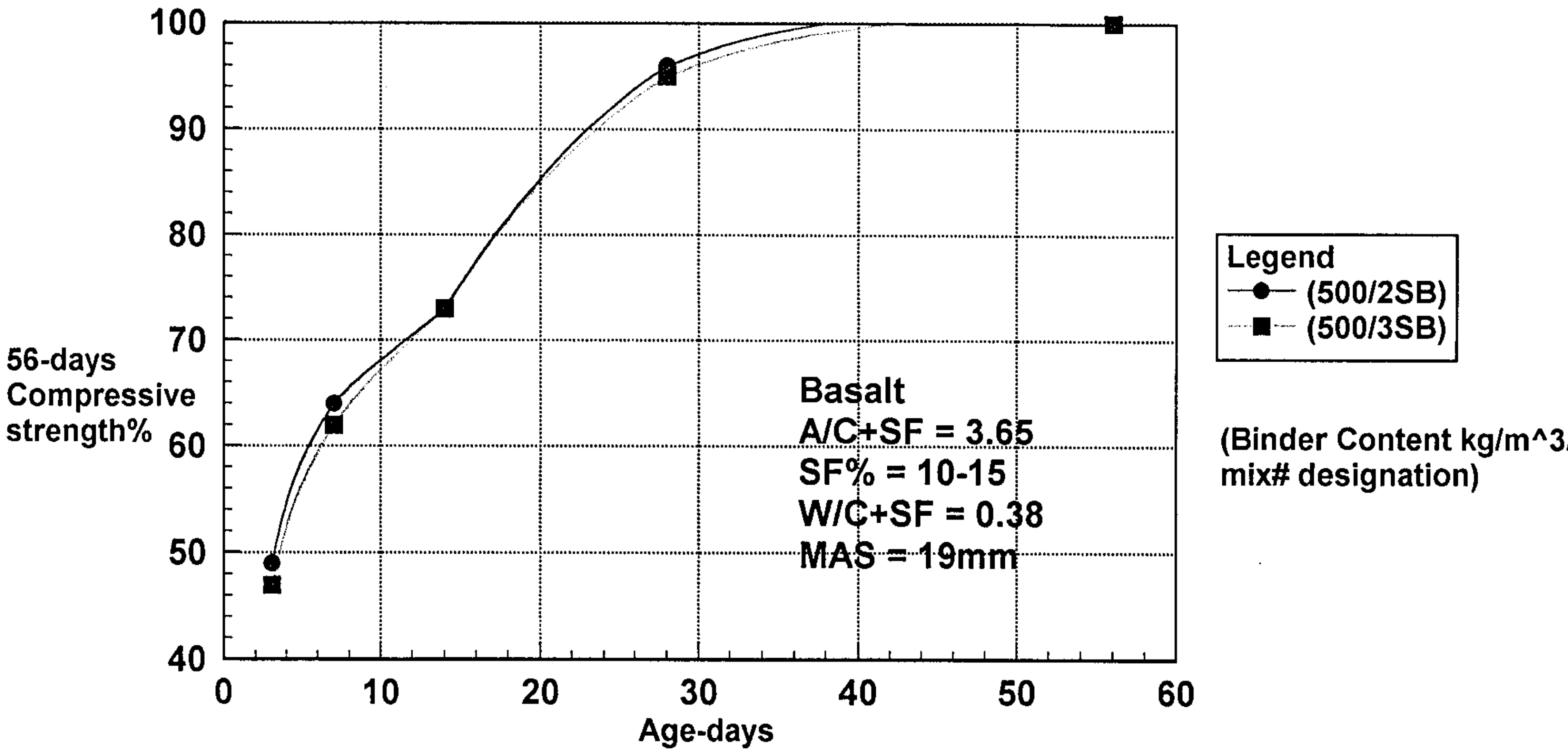

Figure 3.23 Compressive Strength gain with time as a percent of 56-days strength

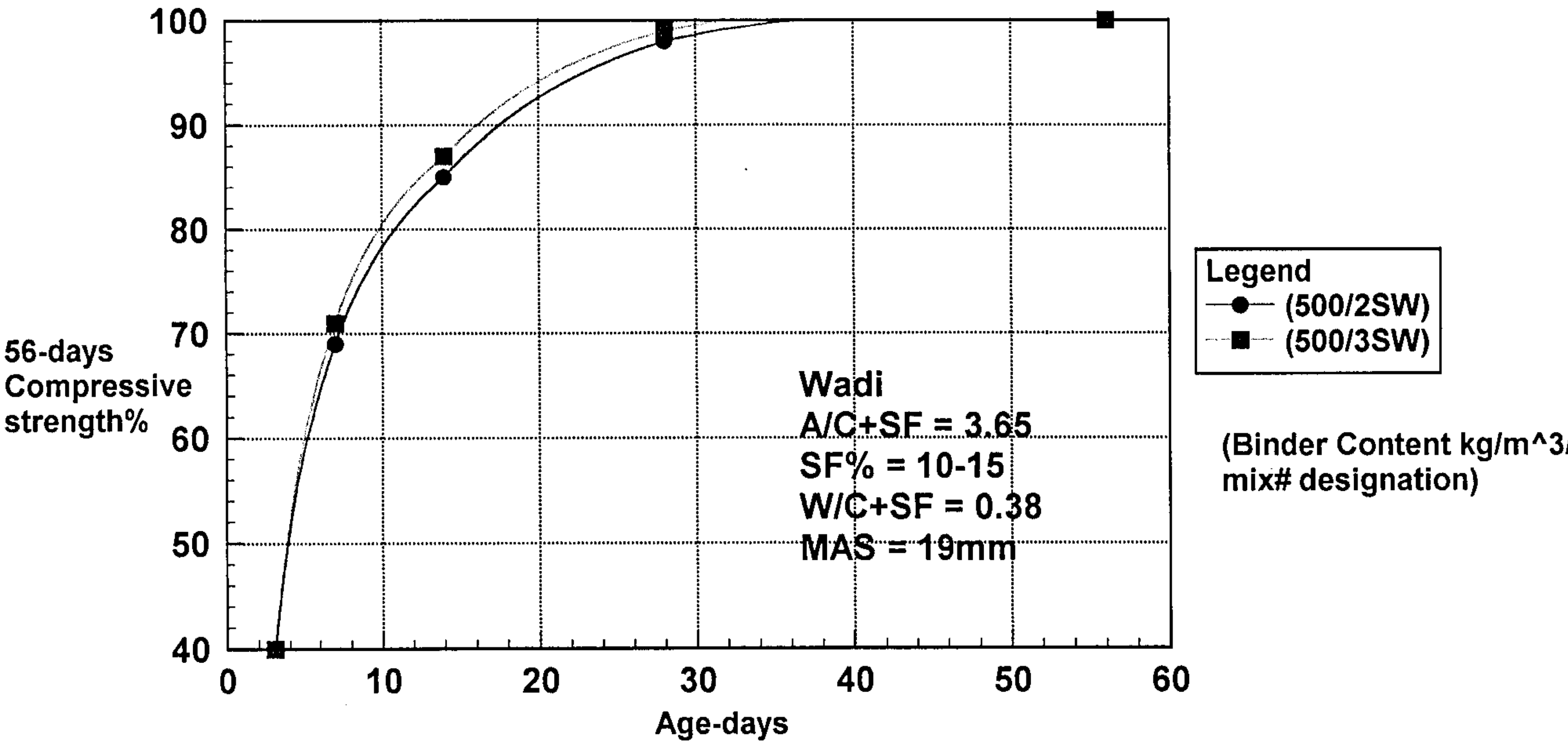

Figure 3.24 Compressive Strength gain with time as a percent of 56-days strength

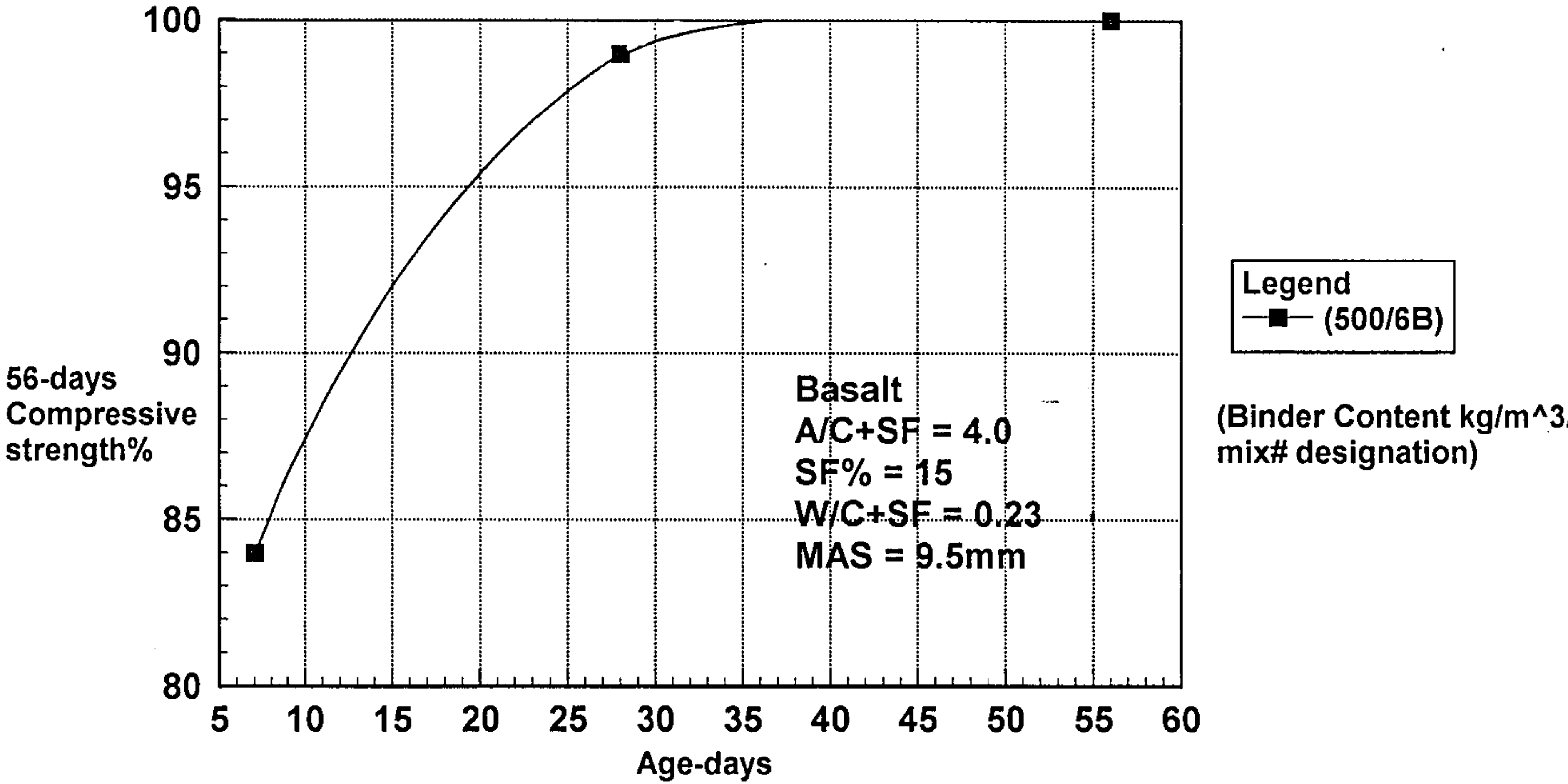

Figure 3.25 Compressive Strength gain with time as a percent of 56-days strength

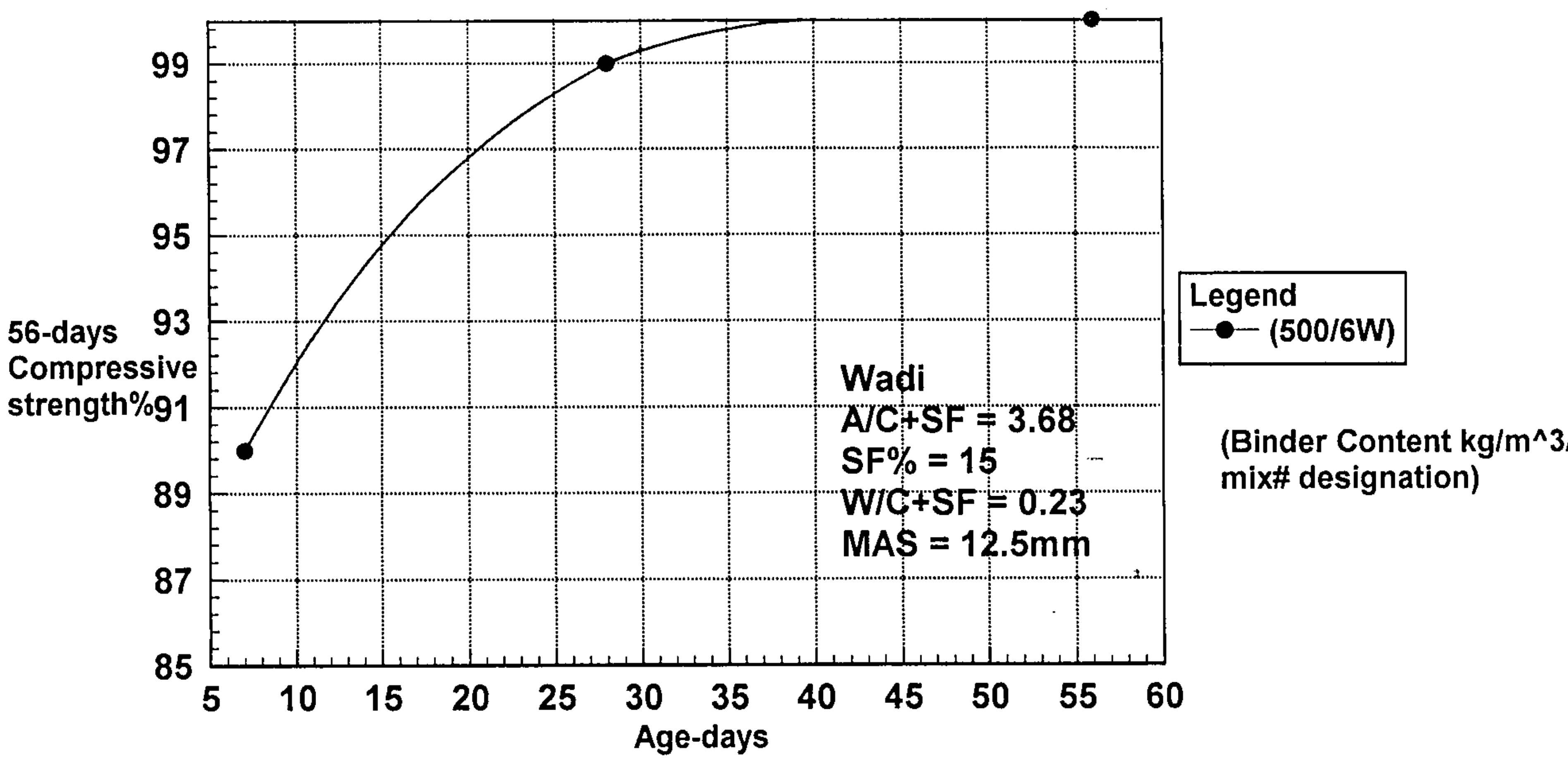

Figure 3.26 Compressive Strength gain with time as a percent of 56-days strength

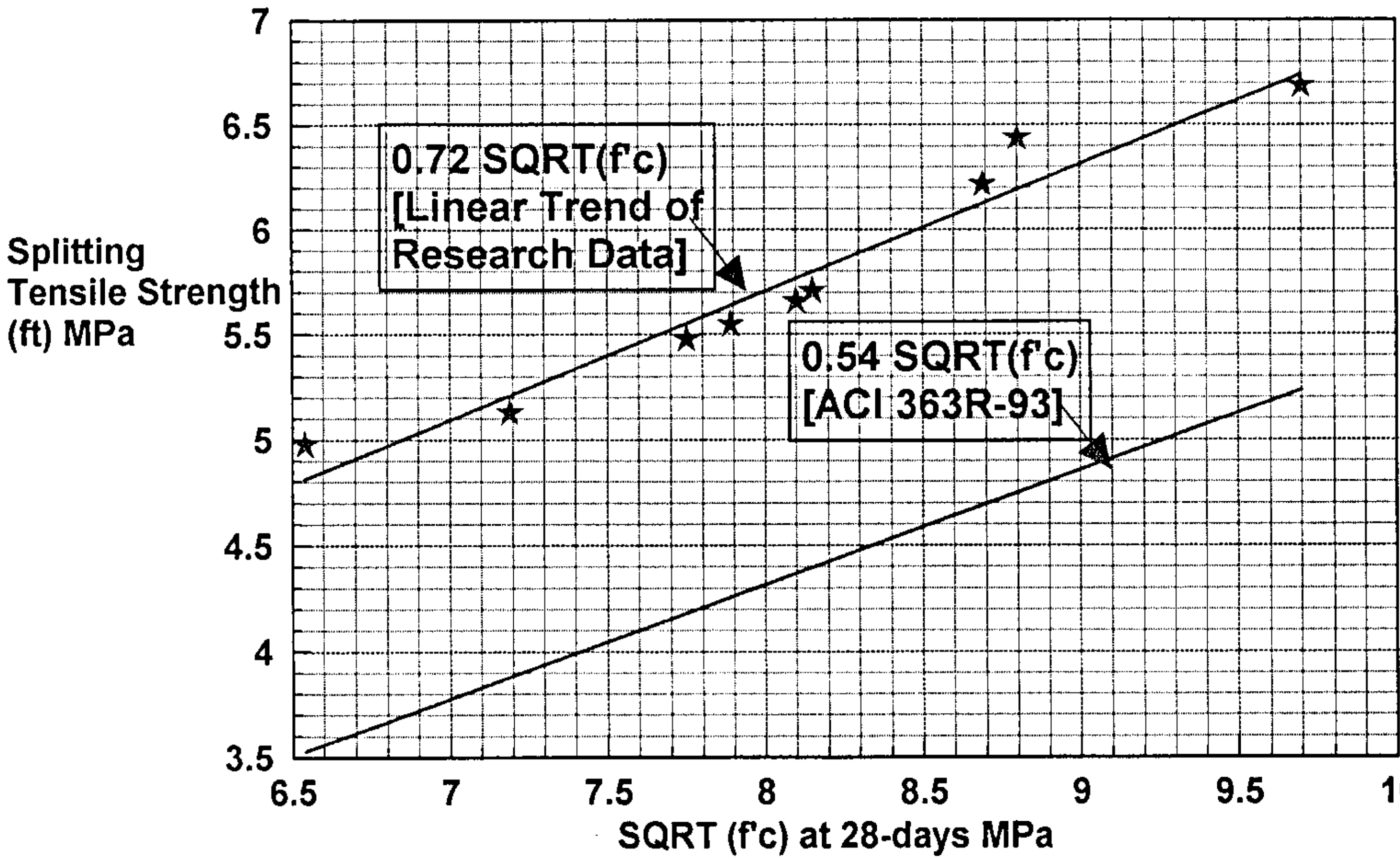

Figure 3.27 SQRT (F'c) at 28-days age Vs Splitting tensile strength for Basalt MAS = 9.5-19mm, with or without Silica Fume

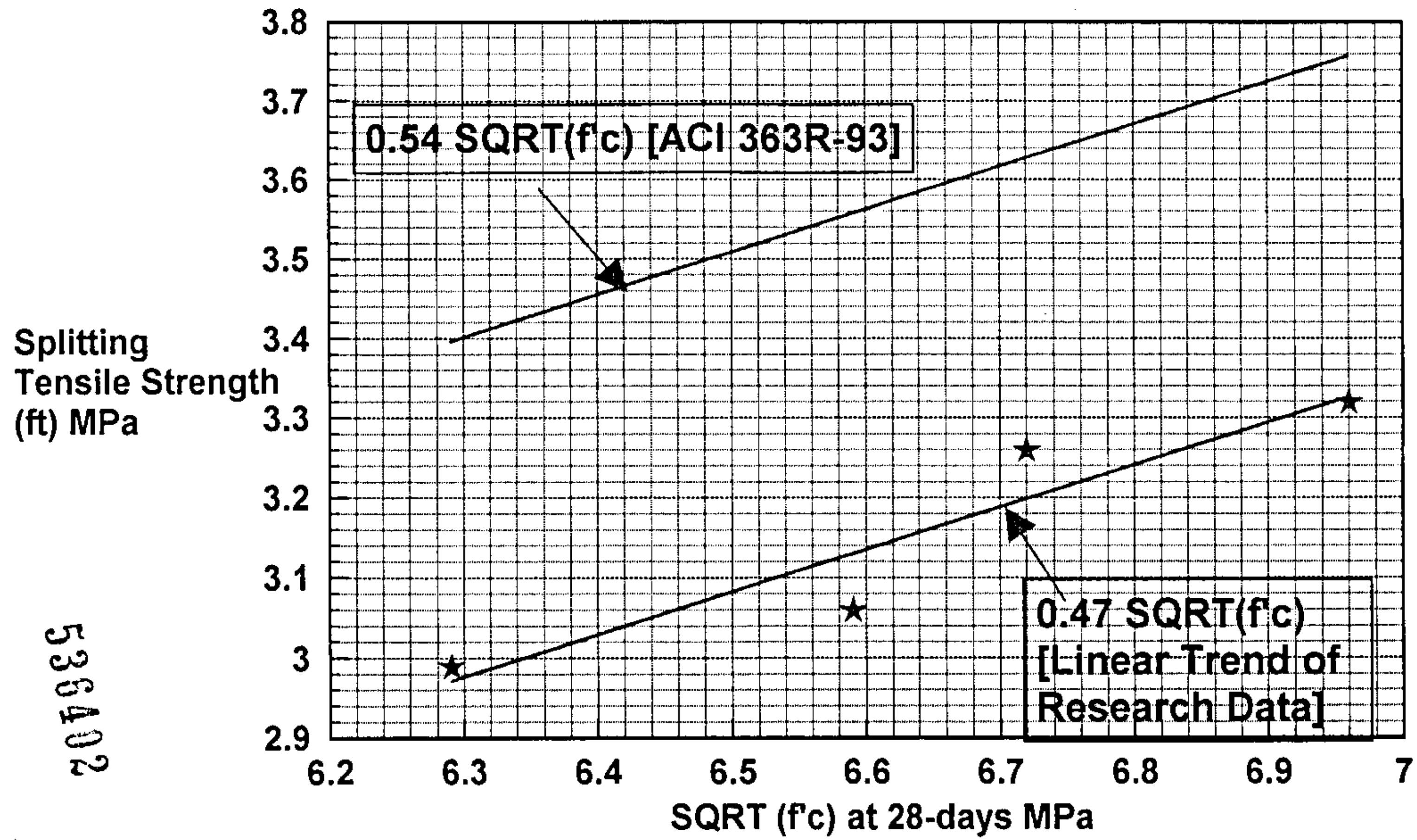

Figure 3.28 SQRT(F'c) at 28-days age vs Splitting tensile strength for Wadi
MAS = 19mm without Silica Fume

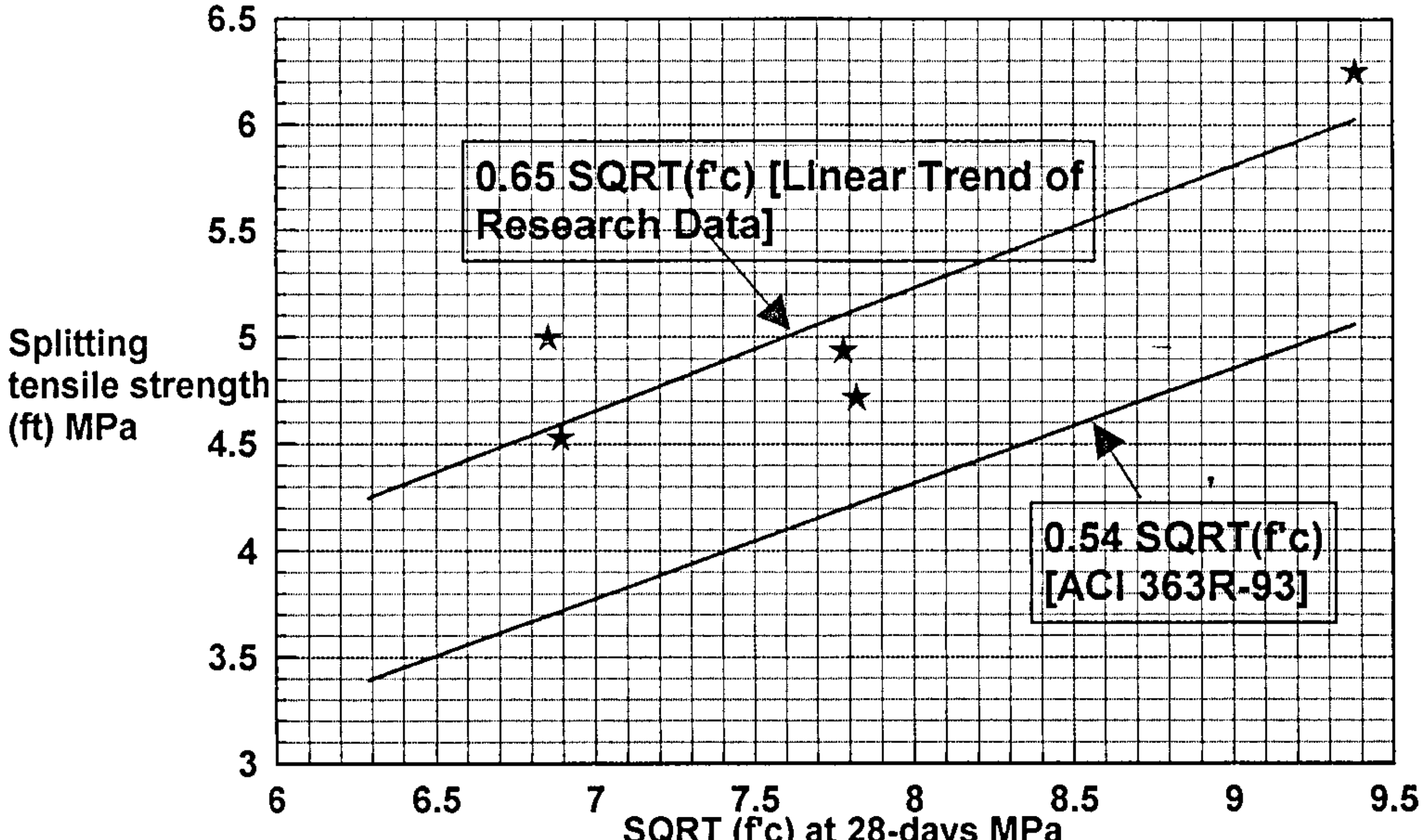

Figure 3.29 SQRT (F'c) at 28-days age Vs Splitting tensile strength for Wadi MAS = 12.5mm with or without Silica Fume / or MAS =19mm with Silica Fume

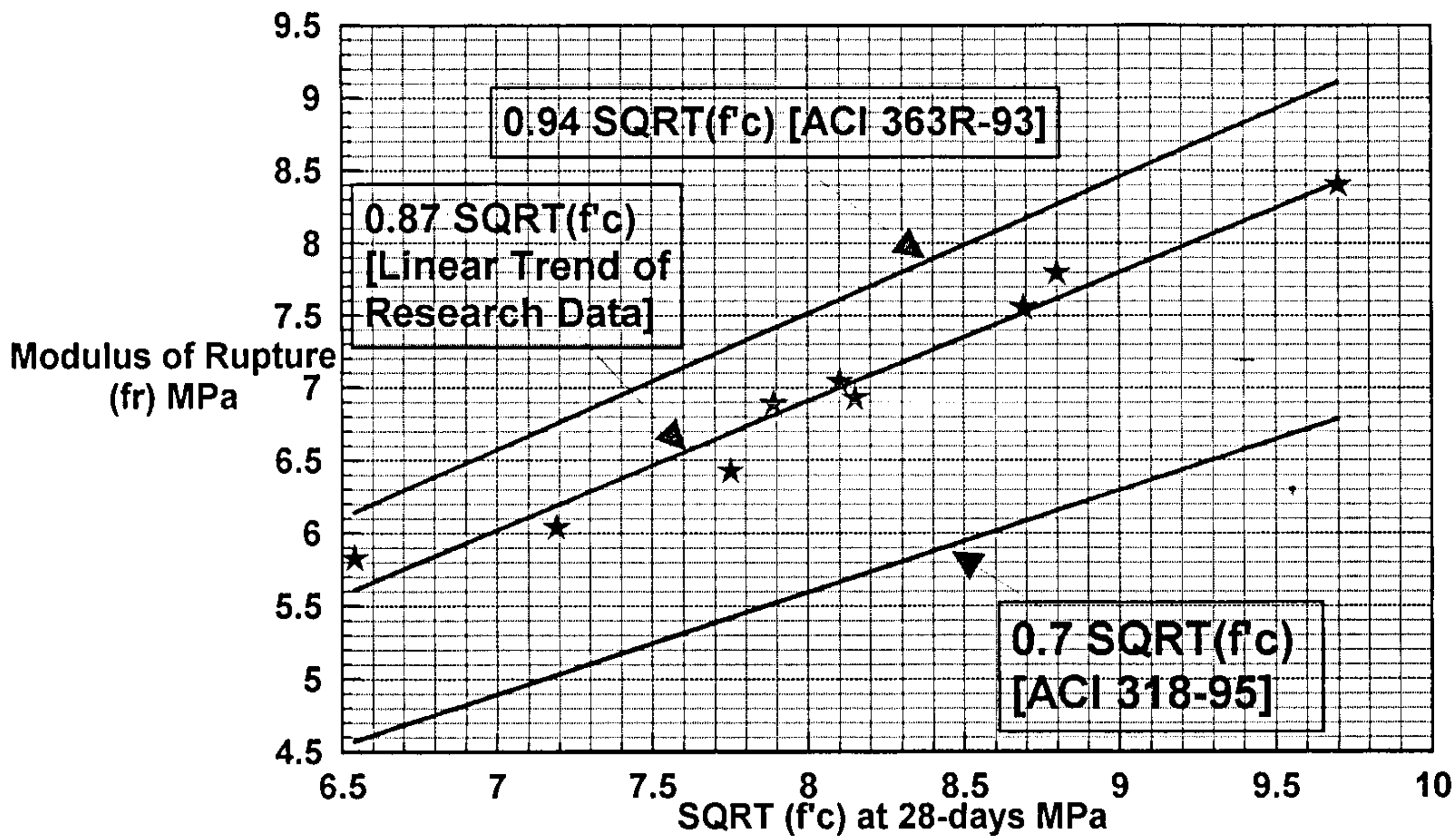

Figure 3.30 SQRT (F'c) at 28-days age vs Modulus of Rupture for Basalt MAS 9.5-19mm, with & without Silica Fume

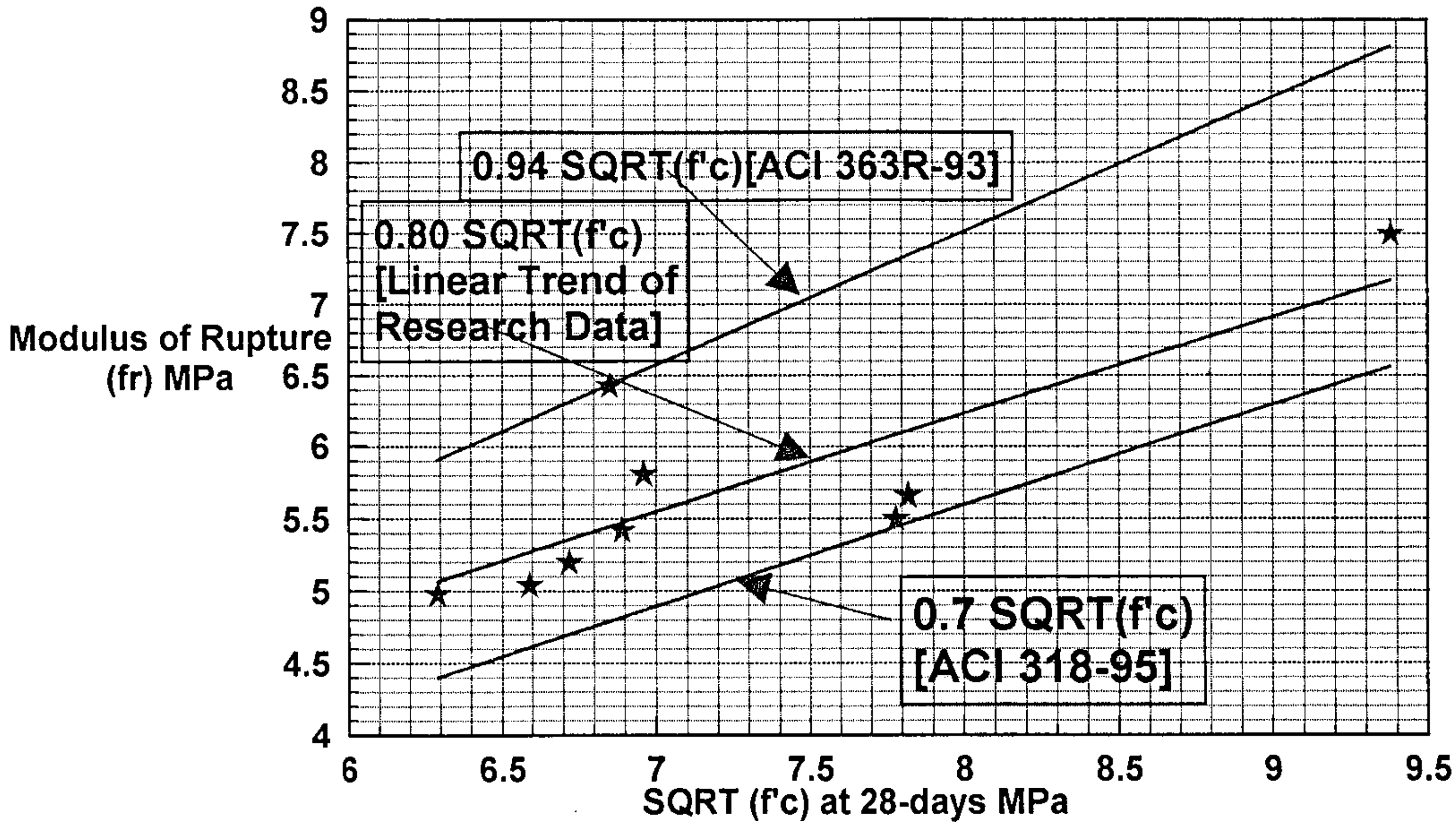

Figure 3.31 SQRT (F'c) at 28-days age Vs Modulus of Rupture for Wadi MAS 12.5-19mm, with or without Silica Fume

4- Conclusions & Recommendations

4.1 Conclusions:

In the view of research results and discussions the following conclusions and recommendations are given. For detailed mix proportions for each mix number designation see Table 2.4.

4.1.1 Compressive Strength of HSC:

1. HSC could be produced locally, just using properly suited and strong aggregates, along with low W/C ratio & HRWRA, without any additional considerations i.e.(using Silica Fume or lowering MAS below 19mm).

2. The highest compressive strength value obtained without use of SF or lowering MAS below 19mm at W/C = 0.38, was for Basalt mix # 2B, which have 62.25 & 72.15 MPa specified 150*300 mm standard cylinder compressive strength at 28 & 56 days respectively.

3. However HSC mixes without SF show always a potential to have more compressive strength after the age of 56-days whatever MAS is, Basalt mixes have a higher potential than Wadi mixes do.

4. Lowering MAS for Basalt from 19mm to 9.5 mm, and for Wadi from 19mm to 12.5mm, increases the compressive strength but slightly. The highest compressive strength value obtained for Basalt was with 9.5mm MAS without SF, was for mix #5B, of 66.49, & 75 MPa standard 150*300 mm cylinder compressive strength at the age of 28, & 56 days respectively. However test results show that it is feasible to use MAS of 19mm in production of HSC.

5. Using Silica Fume as a partial replacement by weight instead of OPC increases compressive strength significantly for levels of replacement > 5% up to optimum replacement level which after no additional compressive strength will be gained with further increase in the level of replacement.

6. The highest obtained compressive strength using SF was for Basalt mix# 2SB of W/C+SF = 0.38, & 10% SF level of replacement which has a 77.38, & 80.36 MPa standard 150*300 cylinder compressive strength at the age of 28, & 56 days respectively.

7. Mixes with SF have minimal potential to gain more strength after the age of 28-days.

8. Wadi mixes show a relatively more gain in compressive strength when using SF more than Basalt mixes.

9. Basalt aggregates mixes have a higher compressive strength values more than Wadi aggregates mixes significantly and under all conditions. The values of compressive strength obtained from Wadi aggregates mixes were for mix# 2W, 5W, & 2SW of 50.22, 50.66, & 62.25 MPa 56-days 150*300 mm standard cylinder compressive strength, versus Basalt same aggregates volume & constitutions proportions mix# 2B, 5B, & 2SB of 72.15, 75, & 80.36 MPa 56-days 150*300 standard cylinder compressive strength.

   The difference in strength as noted is to the right of Basalt mixes of (44%, 48%, & 29%).
   How-ever A/C for Wadi aggregate mixes may be optimized to got a higher results (Figure 3.3), because the A/C ratios in Wadi mixes have been designed to maintain the same volume of corresponding Basalt mixes to held a comparison of aggregate type effect alone.

10. A very-high value of compressive strength obtained by very-low W/C+SF ratio of 0.23, MAS = 9.5mm, SF% = 15, & A/C+SF = 4.0, for mix# 6B from Basalt group, which got a 94.22, 94.79 MPa 28, & 56-days standard 150*300mm cylinder compressive strength. Versus the highest value for Wadi group mix#6W of W/C+SF =0.23, MAS = 12.5mm, SF% = 15, & A/C+SF = 3.68, which got a 88, & 88.70 MPa 28, & 56-days standard cylinder compressive strength.

11. How-ever in the view of test results the values obtained in mix# 6B, & 6W, may be improved more if more trials have been done to optimize A/C+SF ratio, & SF% at the same W/C+SF, but this is beyond the scope of this research. Also it should be mentioned that a very-low W/C+SF ratio gives a not practical mix what-ever the dose of HRWRA, this clear when notice the slump of 50mm only(Table 2.4)

for both mixes, how-ever this may be resolved with more development in HRWRA technology.

4.1.2 Splitting Tensile strength of HSC:

1. Splitting tensile strength directly proportional to compressive strength in general.

2. Splitting tensile strength is largely affected by type, shape, & surface texture of aggregates (65-70%) increase in splitting tensile strength noticed with using Basalt aggregates over Wadi aggregates Versus (8-38%) increase in compressive strength(Tables 3.1, 3.2, 3.20, & 3.21).

3. Lowering MAS increases splitting tensile strength. Rounded and smooth surface Wadi aggregates are more significantly enhanced for f_t with lowering MAS than angular and rough surface Basalt aggregates (Tables 3.22, & 3.23).

4. Increasing SF% level as partial replacement increases f_t. Rounded and smooth surface Wadi aggregates are more significantly improved than angular and rough surface Basalt aggregates with the use of SF(Tables 3.24, & 3.25).

5. The highest values of f_t were for the highest values of f'c which are summarized as following:

    a. 5.55 MPa for Basalt mix# 2B (without SF, MAS = 19mm) Versus 3.32 MPa for same conditions Wadi mix# 1W.

    b. 5.71 MPa for basalt mix# 5B (without SF, MAS=9.5mm) versus 5 MPa for Wadi mix# 5W(without SF, MAS=12.5mm).

    c. 6.44 MPa for Basalt mix# 2SB(with 10% SF, MAS = 19mm), versus 4.94 MPa for Wadi mix# 3SW(with 15% SF, MAS =19mm).

    d. 6.69 MPa for Basalt mix# 6B (with 15% SF, MAS=9.5mm, W/C+SF = 0.23), versus 6.25 MPa for Wadi mix# 6W(with 15% SF, MAS = 12.5mm, W/C+SF = 0.23).

6. Generally the ratio of $f_t/f'c$ decreased with the increase of compressive strength. Also $f_t/f'c$ increased in Wadi mixes when using SF more than the same constituents but without SF, this is not correct for Basalt which still related to compressive strength only (the higher f'c the lower is $f_t/f'c$) (Tables 3.20-27).
   The ratio of two strengths depends on the general level of strength of the concrete. How-ever, there are other factors which affect the relation, the main ones being the shape and surface texture of coarse aggregate, the method of testing concrete in tension, the size of specimen, and the moisture conditions of the concrete (Neville and Brooks, 1990).

4.1.3 Flexure Tensile Strength of HSC:

1. Flexure tensile strength represented by modulus of rupture Fr directly proportional to compressive strength (the higher f'c, the higher is f_r).

2. Modulus of rupture is less sensitive to change in type, shape, and surface texture of aggregates than splitting tensile strength that, (11-33%) increase in Fr to the right of basalt versus (65-70%) increase in Ft (Tables 3.20, 3.21, 3.31, & 3.32).
   Note also that increase in Fr (11-33%) is more correlated with the increase in compressive strength (8-38%), which is not the case in Ft, this clear in comparison of the results for mixes number 1B, 2B, 3B, & 4B, Basalt, and 1W, 2W, 3W, & 4W, Wadi (Tables 3.1, 3.2, 3.31, & 3.32).

3. Lowering MAS increases f_r, rounded and smooth Wadi aggregates behaves less significant increase of f_r than do in f_t that improvement of 24% into f_r versus 53% into f_t when decreasing MAS from 19mm to 12.5mm(Tables 3.22, & 3.33), hence splitting tensile strength is more sensitive to improve when lowering MAS than modulus of rupture when using rounded and smooth surface Wadi aggregates.

4. Increasing the level of SF% replacement increases f_r, a rounded smooth surface Wadi aggregates shows a less significant increase in f_r versus f_t when using SF that (4-9%) increase in Fr versus (39-52%) in Ft, hence again splitting tensile strength more improved

due to using SF than modulus of rupture for Wadi mixes (Tables 3.25, & 3.36).

5. The highest obtained values for f_r are as following:

   a. 6.89 MPa for Basalt mix#2B (without SF, MAS = 19mm),versus 5.81 MPa for same condition Wadi mix# 1W

   b. 6.93 MPa for Basalt mix# 5B (without SF, MAS = 9.5mm), versus 6.43 MPa for Wadi mix# 5W(without SF, MAS = 12.5mm).

   c. 7.8 MPa for Basalt mix# 2SB (with 10% SF, MAS = 19mm), versus 5.66 MPa for same conditions Wadi mix# 2SW.

   d. 8.41 MPa for Basalt mix# 6B (with 15% SF, MAS = 9.5mm, W/C+SF = 0.23), versus 7.56 MPa for Wadi mix# 6W (with 15% SF, MAS = 12.5mm, W/C+SF = 0.23).

6. Generally the ratio $f_r/f'c$ decreases when compressive strength increases under all conditions and type of aggregates (Tables 3.31-38).

4.1.4 Gain of Strength with Time for HSC:

___ It has been found that HSC possess high-rate of gaining strength with time, and irrespective to the dose of superplasticizer it affected by the following factors:

1. A/C ratio & type of aggregates, that a greater A/C posses a higher rate of gain strength, Wadi aggregate mixes show a higher rate of gaining strength than Basalt ones do (Tables 3.8, 3.9, 3.14, & 3.15, Figures 3.15 - 18)

2. Lowering MAS leads to a higher rate of gaining strength (Tables 3.10, & 3.16, Figures 3.19, & 20).

3. Introduce SF up to 5% partial replacement will accelerate the rate of gaining strength especially at high-early age (3-days) (Tables 3.11, & 3.17, Figures 3.21, & 3.22).

4. Increasing SF% of replacement greater than 5% will retard rate of gaining strength at high-early age (3-days), but this effect will not extend more than the age of 7-days, that all SF mixes posses a higher rate of gaining strength than mixes without SF after the age of 7-days (Tables 3.12, & 3.18, Figures 3.23, & 3.24).

4.2 Recommendations:

In the view of test results it is obvious that HSC (f'c > 42 MPa) behave in fundamentally different ways from normal-strength concrete so, engineers should be alerted when treating with HSC applications. It is also obvious that Wadi gravel mixes are less strength behavior than Basalt ones, especially in tensile strength. Hence using Basalt aggregate is more recommended.

The following recommendations may be held for each item discussed in this research:

4.2.1 Relation Between f'c & f_t for HSC:

Regression analysis held in sec 3.2.5 recommends the following relations between f'c & f_t for HSC that may be applied using local materials:

Item	Basalt HSC Mixes	Wadi HSC Mixes
MAS =19mm, without SF	**f_t = 0.72 SQRT(f'c)**	**f_t = 0.47 SQRT(f'c)**
MAS = 9.5-12.5 mm, with or without SF	**f_t = 0.72 SQRT(f'c)**	**f_t = 0.65 SQRT(f'c)**
MAS = 12.5-19 mm With SF	**f_t = 0.72 SQRT(f'c)**	**f_t = 0.65 SQRT(f'c)**

❖ Note that ACI 363R-93 which recommends f_t = 0.54 SQRT(F'c) for normal and high-strength concrete is 33% more conservative than recommended for Basalt HSC mixes, & 20% more conservative than recommended for Wadi HSC mixes with 12.5 mm MAS, and/or mixes with SF what-ever is MAS. At the same time its over estimate Ft for Wadi HSC mixes with MAS 19mm and without SF by 15%.

4.2.2 Relation Between f'c & f_r for HSC:

Regression analysis held in sec 3.3.5 recommends the following relations between f'c & f_r for HSC that may be applied using local materials:

f_r = 0.87 SQRT (f'c) .... For Basalt HSC mixes
f_r = 0.80 SQRT (f'c) .... For Wadi HSC mixes

- Note that ACI 363R-93 which recommends that, f_r = 0.94 SQRT(f'c), for high-strength concrete is over estimate the recommended Fr for HSC Basalt mixes by 8%, and the recommended for Wadi HSC mixes by 17.5%.

- Note also that ACI 318-95 which recommends that, f_r = 0.7 SQRT(f'c) for normal strength concrete is more conservative than recommended for Basalt HSC mixes by 24%, and 14% for Wadi HSC mixes.

4.2.3 Rate of Strength gain for HSC:

Because HSC shows a potential to have higher gain of strength with time it may be feasible and recommended to specify compressive strength at the age of 56-days, different than 28-days which is recommended always [ACI 318-95] for normal-strength concrete. However SF mixes which shows minimal potential to gain strength after the age of 28-day, which is still a recommended date to specify compressive strength in the case of HSC using SF.
Considering only the optimum A/C ratios which will be adopted in any mix-design of HSC mix at the basis of highest compressive strength, and in the range of test results the following recommended rate of gaining strength for HSC using local materials may be adopted based on average results of gaining strength for each item related to the compressive strength at 56-days for both types of aggregate mixes & using HRWRA :

MAS (mm)	SF%	Percent of Compressive Strength at 56-days			
		3-d	7-d	14-d	28-d
19	0.0	64%	66%	81%	89%
9.5-12.5	0.0	70%	76%	85%	91%
19	< or = 5.0	71%	73%	83%	95%
19	10.0-15.0	44%	66%	81%	97%

4.2.4 A proposed Technique for Mix-Design of HSC Using Local Materials:

__In the view of test results the following technique for mix-design may be applied for production of HSC using local materials:

1. The preferable aggregates shall have the following properties:

   a. Abrasion resistance of coarse aggregates should be high (not more than 20-30% loss by weight using Los Angeles Machine [ASTM C131], that to insure and distinguish strong aggregate from other.

   b. F.M of fine aggregates shall be in the range of (2.5-3.2) according to ACI 211.4R-93.

   c. Grading of coarse and fine aggregates shall comply with the limits specified by ASTM-C33.

   d. Angular shape and rough surface texture are preferable for coarse aggregates (Basalt aggregate are recommended).

2. Choose the optimum volume of coarse aggregate per unit volume of concrete using the recommended volumes by ACI 211.4R-93 as following:

ACI 211.4R-93 Table 4.3.3-Recommended volume of coarse aggregate per unit volume of concrete.

Optimum coarse aggregate contents for nominal MAS to be used with sand with F.M of 2.5-3.2			
MAS (mm)	9.5	14	19
Fractional volume* of oven dry rodded coarse aggregate	0.65	0.68	0.72

* Volumes are based on aggregates in oven-dry rodded condition as described in ASTM C29 for unit weight of aggregates.

- Moisture state of aggregate should be considered when deciding the quantity needed of coarse and fine aggregate.

3. Apply 475-500 kg/m^3 cement content , for not greater than 19mm MAS, are recommended in production of HSC using local materials to maintain the following:

a. Increase rate of gaining strength (high-early gain of strength) which is needed always in HSC applications.

b. To maintain a high-range of workability, because use of high-cementitious materials content resulted in more water content, and the presence of high-cementitious materials content will be offset by using HRWRA.

c. Also cement content in excess of 500 kg/m^3 may affect A/C or A/(C+SF) ratio at a MAS of 19mm (which is feasible to use in production of HSC in the view of research conclusions) to lower values which resulted in increase paste thickness which adversely affect strength, and/or the risk of thermal cracks will increase.

4. Choose a W/C or W/(C+SF) ratio using recommended maximum values in ACI 211.4R-93 for each strength grade required.

ACI 211.4R-93 Table 4.3.5(b) –Recommended maximum W/C+P ratio for concretes made with HRWRA.

Field Strength F'c MPa		W/C or W/(C+P)* MAS (mm)		
		9.5	14	19
48	28-d	0.5	0.48	0.45
	56-d	0.55	0.52	0.48
55	28-d	0.44	0.42	0.40
	56-d	0.48	0.45	0.42
62	28-d	0.38	0.36	0.35
	56-d	0.42	0.39	0.37
69	28-d	0.33	0.32	0.31
	56-d	0.37	0.35	0.33
76	28-d	0.30	0.29	0.27
	56-d	0.33	0.31	0.29
83	28-d	0.27	0.26	0.25
	56-d	0.29	0.28	0.27

- *P is any mineral admixture used with OPC including silica fume, the letter P is after Pozzolana which are the most common mineral admixture used with concrete.

- Note that the above maximum values are for field so the lab results for a W/C, or W/C+SF which used in research which is mainly 0.38 gives a higher average compressive strength than presented at the same W/C or W/C+P especially for Basalt mixes, and nearly the same for Wadi mixes with SF.

5. Water demand could be determined by knowing W/C or W/C+SF, and using a cementitious material content between 475-500 kg/m^3. Moisture State of aggregate should be considered when deciding final water needed.

6. Fine aggregate content could be estimated ignoring entrapped air volume by subtracting all previous ingredient from 1m^3 of concrete [1-(V_C + $V_{C.A}$ + V_W)].

7. A trial mixes should be held three mixes at least to obtain optimum A/C or A/C+SF ratio, each time the cementitious material content varied between 475-500 kg/m^3.

8. When using SF as partial replacement a further trial mixes should be held differing the level of replacement, each time 3-mixes at least to obtain the optimum level of replacement, however test results show for the range of 475-500 kg/m^3 cementitious materials 10% level at 19 mm MAS is recommended.

9. The dose of HRWRA should be according to manufacture informa-tion, and to the level of required workability, and it should be conside-red into the final water content required to maintain the same W/C or W/(C+SF) especially at high doses.

10. If the required strength was not gained lower the W/C or W/C+SF and redo the steps 5 to 9.

4.2.5 A recommended experimental extension in the same field:

HSC still not applied well using Jordan local materials and construction practice, so its open area of many researches needed to recommends a local practice of using HSC evenly in design structures and into field practice. However the following items should be tested well for that reason:

1. Modulus of elasticity of HSC.
2. Ductility of HSC.
3. Reinforced HSC behavior under flexure, compression, shear, & torsion.
4. Long-term behavior of HSC (creep, Shrinkage, & deflection under service load).
5. Long-term durability of SF concretes (pore structure and permeability, freeze and thaw resistance, sulfate attacks, & steel corrosion).

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5- References

American Concrete Institute, ACI 318-95, Building Code Requirements for Reinforced Concrete and Commentary, Detroit, 1995.

American Concrete Institute, ACI 211.4R-93, Guide for selecting Proportions for High-Strength Concrete with Portland Cement and Fly Ash, Detroit, 1993.

American Concrete Institute, ACI 234R-96, Guide for the Use of Silica Fume in concrete, Detroit, 1996.

American Concrete Institute, ACI 363R-93, State-of-the-Art Report on High-Strength Concrete, Detroit, 1993.

American Society for Testing and Materials, ASTM-93, Concrete and Aggregate, 100 Barr Harbor Drive, 1993.

Ashour, S. A., Mahmood, K. and Wafa, F. F. 1997. Influence of Steel Fibers and Compression Reinforcement on Deflection of High-Strength concrete Beams. *ACI Structural Journal*, 90 (6): 632-642.

Carrasquillo, R. L., Nilson, A. H. and Slate, F. O. 1981. Properties of High-strength Concrete Subjected to Short-Term Load. *ACI Journal*, 78 (3): 171-178.

Cetin, A. and Carrasquillo, R. L. 1998. High-Performance Concrete: Influence of Coarse Aggregates on Mechanical Properties. *ACI Materials Journal*, 95 (3): 252-261.

Cordon, W. A. and Gillespie, H. A. 1963. Variables in Concrete Aggregates and Portland Cement Paste Which Influence the Strength of Concrete. *ACI Journal*, 65 (4): 293-298.

De Larrad, W. and Belloc, A. 1997. The Influence of aggregate on the compressive Strength of Normal and high-strength Concrete. *ACI Materials Journal*, 94 (5): 417-426.

Goldman, A. and Benture, A. 1989. Bond Effects in High-Strength Silica Fume Concretes. *ACI Materials Journal*, 86 (5): 440-447.

Khayat, K. H., Wachon, M. and Lanctot, M. C. 1997. Use of Blended Silica Fume Cement in Commercial Concrete Mixtures. *ACI Materials Journal*, 94 (3): 183-192.

Kilpatrik, A. E. and Vijaya, B. 1999. Tests on High-Strength Concrete-Filled Steel Tubular Columns. *ACI Structural Journal*, 96 (2): 268-274.

Kuennen, T. 1996. Silica Fume Resurges. Reprinted with permission from Concrete Products, London.

Kumar, P. and Monterio, P. J. 1993. *Concrete Structure, Properties, and Materials, 2nd edition.* Englewood Cliffs: Prentice-Hall, London.

Mahmood, K. M., Ashour, S. A. and Al-Noury, S. I. 1995. Effective Moment of Inertia and Deflections of Reinforced Concrete Beams Under Long-Term Loading. *Engineering Journal of University of Qatar*, 94 : 101-125.

Neville, A. M. and Brooks, J. J. 1990. *Concrete Technology, 2nd edition.* Longman Singapore Publishers (Pte) Ltd, Singapore.

Setunge, S., Attard, M. M. and Darvall, P. L. 1993. Ultimate Strength of Confined Very-High-Strength Concretes. *ACI Structural Journal*, 90 (90): 632-642.

Trende, U. and Buyukozturk, O. 1998. Size Effect and Influence of Aggregate Roughness in Interface fracture of Concrete Composites. *ACI Materials Journal*, 95 (4): 331-338.

Wafa, F. F., Shihata, S. A., Ashour, S. A. and Akhtaruzzaman, A. A. 1995. Prestressed High-Strength Concrete Beams Under Torsion. *ACI Structural Journal*, 121 (9): 1280-1286.

Well, T. C. 1998. Addressing Parking Garage Corrosion with Silica Fume. *Transportation Research Record* (1204).

Xie, J. A. and MacGregor, J. G. 1995. Mechanical Properties of Three High-Strength Concretes Containing Silica Fume. *ACI Materials Journal*, 92 (2): 135-146.

ملخص

خواص الخرسانة العالية الإجهاد باستخدام المواد المحلية

إعداد
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المشرف
الأستاذ الدكتور بسام أبو غزالة

تم استكمال برنامج بحثي شامل لدراسة إمكانية إنتاج خرسانة ذات مقاومة عالية باستخدام المواد المحلية، حيث تم الحصول على قيم مرتفعة من المقاومة و بمستوى مشغولية مرتفع باستخدام المضافات الملدنة. كما أنة تم العمل على دراسة اثر نسبة محتوى الركام إلى المواد الأسمنتية (A/C)، الحجم الأعظمي لحبيبات الركام (MAS)، واستخدام مضافات معدنية وذلك باستبدال وزني نسبي للأسمنت البورتلاندي العادي بمثيلة من السيليكافيوم (Silica-Fume)، اثر ذلك على الخواص الميكانيكية للخرسانة وهي مقاومة الإجهاد بالضغط (Compressive Strength)، مقاومة إجهاد الشد بالانفلاق (Splitting Tensile Strength)، مقاومة إجهاد الشد بالانحناء (Flexure Tensile Strength)، ومعدل اكتساب المقاومة مع الزمن.

أيضا" تم استخدام نوعين من الركام، ركام بازلتي و ركام وادي، مما دعم البحث لدراسة أثر شكل وبنية سطح حبيبات الركام على الخواص الميكانيكية للخرسانة، بغض النظر عن مدى قوة الركام نفسه، والتي أظهرت أثرا" فعالا" على مقاومة الإجهاد بالشد للخرسانة. وأظهرت نتائج البحث أن الخلطات التي استخدم فيها ركام بازلتي تتمتع بمقاومة أكبر ولمختلف الخواص الميكانيكية من الخلطات التي استخدم فيها ركام وادي.

أعلى مقاومة إجهاد بالضغط تم الحصول عليها على عمر خرسانة 28 و 56 يوم بالترتيب هي 77,4 و 80,4 نيوتن/ملم^2 للأسطوانة القياسية، باستخدام ركام بازلتي وسيليكافيوم على نسبة ماء إلي الأسمنت والسيليكافيوم (W/C+SF) مقدارها 38%، ومقاومة إجهاد بالضغط 94,2 و 94,8 نيوتن/ملم^2 للأسطوانة القياسية على عمر خرسانة 28 و 56 يوم بالترتيب، باستخدام ركام بازلتي و سيليكا فيوم وحجم اعظمي صغير للركام (9.5 ملم) على نسبة ماء إلى الأسمنت والسيليكافيوم (W/C+SF) مقدارها 23%.