STRENGTH ASSESSMENT of LIGHTWEIGHT CONCRETE

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by

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ABSTRACT

The Thesis is concerned with strength characteristics of lightweight concrete, with particular emphasis upon methods of assessment of insitu properties. Fundamental properties of lightweight concretes have been reviewed, and relevant aspects confirmed and augmented experimentally for concretes made with a range of commercially available aggregate types. Those considered have been Lytag (fully and semi-lightweight concretes), Leca and Pellite. Experimental studies have assessed the suitability of a range of available insitu testing methods for use with those concretes. Tests considered were rebound hammer, ultrasonic pulse velocity, small cores, Windsor Probe, pull out, internal fracture, and pull off methods. Failure mechanisms for the latter four approaches have also been considered analytically, in some cases with the aid of finite element techniques, in an attempt to explain and justify features observed experimentally. A series of large scale reinforced beams have also been made to enable results to be extended beyond small scale specimens.

It has been shown that most of the test methods can be successfully applied to lightweight concrete, but that strength correlations are different to those for normal weight concrete and in most cases vary between different lightweight materials. These may also be influenced by curing history in some cases. In some instances, accuracy of strength prediction may exceed that usually accepted as being possible for normal weight concrete. The analytical studies have generally supported these findings, and in the case of pull off tests have highlighted important practical aspects of the test method which have previously not been fully considered in practice. It has also been demonstrated that insitu variability of lightweight concrete may differ according to concrete type and may not necessarily be similar to that expected for normal weight concrete, with resulting implications for planning and interpretation of insitu investigations.

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GENERAL NOTATIONS

C	Capo force
D	depth of failure cone
d	core diameter
E_s	static modulus of elasticity
E_d	dynamic modulus of elasticity
e	base of natural logarithm
F	pull out force
f_c	cube compressive strength
f_{cc}	core strength
f_{cy}	cylinder compressive strength
f_t	tensile splitting strength
L	Lok force
ℓ	core length
ℓ_c	side length of failure cone
p	pull off strength
R	rebound number
T	torque
V	pulse velocity
W	exposed probe length

CHAPTER 1. INTRODUCTION

1.1 GENERAL

Compressive strength testing has long been accepted as the principal criterion for judging the quality of concrete. Most specifications for concrete therefore have a provision that for quality control purposes standard specimens shall be tested for compressive strength. In the U.K. these are commonly in the form of cubes, whilst in U.S.A. and elsewhere cylinders are used. The results of the tests thus fulfill a useful and necessary function, in that they allow concrete quality specifications to be checked. They do not, however, necessarily provide a good indication of the actual strength of concrete in structures. The reasons for this lie in three main factors which affect the strength of concrete at a given age. These are:

1. Variations of the concrete supply.
2. The degree of compaction achieved when the fresh mix was poured.
3. The curing condition.

Standard cube (cylinder) compressive strengths are sensitive to the first factor provided that sampling is sufficiently frequent. There may, however, be considerable differences between the compaction and curing conditions of the test specimens and the insitu concrete which could significantly affect the strengths.

Documentation of concrete strength in structures may be required in certain cases when;

i) there are no standard specimens or the test results do not comply with the specification

ii) doubts arise about the adequacy of the quality of the insitu structure

iii) functional changes and new loads are considered in old structures

iv) there are observed signs of deterioration of the concrete due to fire, chemical attack or other environmental effects.

There is therefore, a need for reliable methods to determine the strength of the concrete in its as-cast form. This method should necessarily leave the structure under test basically undamaged and still fit for its design purpose.

Many insitu testing techniques have been developed to evaluate the insitu strength of concrete. Currently three different categories of insitu test methods are used, non-destructive, partially destructive and destructive tests. Among these tests, some, like the rebound hammer and core tests, have been in use for many years whereas others, such as the B.R.E. internal fracture test and pull off test, are more recent developments. The proponents of each method invariably attempt to correlate the results of their tests to the compressive strength of concrete test cubes (cylinders) which enables the production of calibration graphs from which an estimated concrete strength may be obtained.

In recognition of the need for insitu testing, a great deal of research has been carried out to assess the reliability of a range of insitu test methods on concrete made from natural dense aggregates. As a result of the large consumption of aggregates by the construction industry, the continuing extraction of natural aggregates from the ground is likely to lead to severe environmental problems. The need to find another source of aggregates imposes itself therefore, if irreversible deterioration of the countryside is to be avoided. Subjecting certain industrial wastes to various manufacturing processes has the merit of supplying aggregates and providing an opportunity for the disposal of a waste material. Some such aggregates are porous and light in weight due to the introduction of internal voids during the process of manufacture. Owing to their low density, the concrete produced using these aggregates is called lightweight concrete because it is lighter in weight (up to 2000kg/m^3) than normal weight concrete (approximately 2400kg/m^3).

Structural lightweight concrete has become established as an important and versatile material in modern construction. The reasons for development taking place in this field are technical and economic. Its lower density means that a structure's dead weight can be reduced with a consequent use of smaller sections and the corresponding reduction in the size of foundations. Furthermore, lightweight concrete shows better insulating and fire resistance properties. Also drilling, cutting and chasing of lightweight concrete is easier and cheaper. Consumption of lightweight concrete has already grown in recent years, and it is believed that the use of insitu test methods will become increasingly important when assessing the insitu strength and quality of concrete. Regarding research in the field of insitu testing, very little attention has so far been paid to lightweight concrete.

The present day lightweight aggregates manufactured in the U.K., namely Lytag, Pellite and Leca, are capable of providing the engineer with better strength/density ratios together with increased ease in placing, cutting, grinding and fixing. The uses of lightweight concretes made with these aggregates have been demonstrated by researchers including Swamy and Lambert (1983), Anon (1989), and Mayfield and Louati (1990), who affirm that they have satisfactory properties for structural applications.

1.2 PURPOSE AND SCOPE OF THE PRESENT INVESTIGATION

Previous studies of insitu strength assessment of lightweight concrete, have typically been restricted to consideration of only a limited range of insitu test techniques. In each individual investigation, one typical lightweight concrete has generally been used to represent lightweight concrete in general. In many such investigations, lightweight concrete was considered to be a secondary material and a limited number of tests performed to compare behaviour with that of normal weight concrete.

Research carried out by Teychenné (1967) on the influence of lightweight aggregate characteristic on the properties of concrete, showed however that each type of lightweight aggregate has its own characteristics and is, in general, different from that of other lightweight aggregates. This suggests that the properties of one lightweight concrete cannot be generalized to all types. Thus it is desirable to investigate the properties and behaviour of each type of available lightweight concrete more comprehensively in relation to insitu strength testing.

Following a literature review of previous investigations four principal areas of research were selected for the present study.

1. Investigate the principal properties of different types of lightweight concretes concerning compressive strength, tensile strength and stiffness. This study was considered necessary as a basis to understanding the behaviour of different lightweight concretes while under insitu tests.

2. Apply a range of insitu tests to lightweight concretes to derive calibration graphs. The parameters to be studied included

   i) type of lightweight aggregates
   ii) effect of curing regime
   iii) effect of age up to 1 year

   It was also intended that the accuracy and reliability of the test methods on lightweight concretes be established from a statistical analysis of the results of this part of the study.

3. Assessment of the mechanism of failure for a number of insitu test methods. This aspect has been given only very limited consideration in previous investigations. In the present study, the aim was to demonstrate the mode of failure and hence explain aspects of the influence of concrete type on insitu test measurements.

4. Assessment of concrete variability in full scale elements. Despite numerous previous studies of strength variations within different types of concrete element (column, wall, beam and slab), the possible influence of concrete type (i.e. lightweight and normal weight concretes) has not been considered. It was decided to examine this feature by using results from insitu tests on full scale concrete beam elements made from different types of concrete to study insitu strengths and their variability.

A comprehensive experimental programme was undertaken to examine the performance and reliability of eight different insitu tests namely,

Rebound Hammer

Ultrasonic Pulse Velocity

Windsor Probe

Pull-Out

B.R.E. Internal Fracture

Direct Pull Internal Fracture

Pull-Off

Cores (50mm)

applied to a range of lightweight concretes made from Lytag, Leca and Pellite.

1.3 STRUCTURE OF THESIS

A short description of the structure of the thesis is outlined below:

A general description of the various structural properties of lightweight concretes is given in Chapter 2.

Chapter 3 reviews the available insitu tests for estimating concrete strength and discusses their implications. The major factors which influence the test results are outlined based on available data for normal weight concrete. Accuracies of strength prediction based on 95% confidence limits are also presented for normal weight concrete.

The materials used in this investigation and the method of manufacture of test specimens along with details of the experimental programme are presented in Chapter 4. Several mixes were designed for each of the five different types of concrete. These being four types of lightweight concrete and one type of normal weight concrete for comparison. The lightweight concretes were either fully or semi lightweight. Only one type of fully lightweight concrete was used which was made from coarse and fine

Lytag. The three types of semi lightweight concrete consisted of natural sand and a coarse lightweight aggregate (Lytag, Leca or Pellite). In the normal weight concrete, North-Notts crushed gravel and sand were used, with the same sand being used in the semi lightweight concretes.

Chapter 5 covers the measured short term and long term properties of the different concrete types. The properties investigated included compressive strength, tensile strength and elastic moduli (static and dynamic). Results were obtained for ages up to 360 days under different curing conditions. Particular attention has been given to the influence of aggregate characteristics as well as the effects of sand replacement for fine lightweight aggregate. These features are believed to be important to a proper understanding the insitu test measurements.

Chapter 6 is concerned with the assessment of the insitu test techniques on a number of lightweight concretes. The study was again focussed on the influence of aggregate characteristics which may influence the test mechanism and results. Correlations between insitu test measurements and compressive strength were examined under different curing conditions at different ages. Accuracies of the strength prediction expressed by various types of empirical correlations were examined by regression analysis. The influence of concrete type on the correlation has been taken into account by comparing the correlations obtained. These correlations are also compared with those existing for normal weight concretes, or obtained in this study. Coefficients of variation obtained in the study have been used to examine the effect of concrete type on variability of the insitu test measurements.

The development of the failure mechanisms of selected partially destructive tests namely, Windsor Probe, pull out, direct pull internal fracture and pull off (surface and partial cored) are detailed in Chapter 7. In the Windsor Probe tests, the internal mode of fracture was examined by observing the internal fracture zone by cutting sections of the concrete specimens. For the direct pull internal fracture tests, a theoretical relationship between failure force and concrete strength is presented using basic engineering mechanics, assuming a tensile failure. The analytical correlations between failure force and cube strength for different types of lightweight concrete were compared with empirical correlations obtained in the experiments. For the pull out and pull off tests, theoretical analyses of stress distributions were carried out using a linear finite element computer code called ANSYS (Swanson Analysis Systems, 1989). In the pull out tests, the influence of concrete types (lightweight and normal weight concretes) were examined at the stage of precracking. In the pull off tests, the theoretical analyses were applied extensively by loading the model up to 50% of expected failure load. The parameters investigated were concrete type, material type and size of disks, and the effect of partial coring.

Chapter 8 describes a programme of insitu tests including rebound hammer, pulse velocity and pull out tests on full scale reinforced beams (2.2x0.3x0.5m) to determine the insitu concrete strengths and variability. The beams were cast from five available concretes (lightweight and normal weight concretes). The merits of the different insitu test methods were compared in relation to strength estimation and its variation across the member depth. An approach is also presented to estimate mean concrete strength in beams and cube specimens using statistical methods. The latter estimation was given within prescribed confidence limits, based on limited sampling of insitu tests and cubes.

Finally in Chapter 9 the conclusions with regards to the present investigation are presented, together with recommendations for future work.

CHAPTER 2. LIGHTWEIGHT CONCRETE

2.1 HISTORICAL DEVELOPMENT

The benefit of lightweight aggregate in concrete as a structural material has been recognized as far back as Roman days. In the second century AD, the Romans built the 44m diameter dome of the Pantheon in Rome using natural pumice aggregate. The modern lightweight industry dates back to 1917 when S.J. Hayde developed a process for expanding shale and clay to form hard lightweight material called Haydite, suitable for making concrete of substantial strength and low density (ACI, 1979).

One of the earliest uses of reinforced lightweight concrete was in the construction of ships and barges by the Emergency Fleet Building Corporation of World War I. The concrete weighed about 1700 kg/m^3 and had a compressive strength of about 35 N/mm^2 using expanded shale. The performance of such concrete was shown to be excellent in service, particularly as regards durability and resistance to abrasion and fatigue. Richart and Jensen (1931), who were probably the first investigators, conducted a very extensive study at the University of Illinois in 1930 on Haydite (expanded shale) lightweight concrete. Their research is regarded as a major contribution to the early acceptance of lightweight concrete as a structural material.

In the U.K., the use of lightweight aggregates for structural concrete was proposed as early as 1936 (Lea, 1936), but this was not followed up until the latter 1950's. In 1957 a programme of research was started at the Building Research Station to investigate the properties of lightweight concrete.

The first processed lightweight aggregate in the U.K., foamed blast furnace slag, came into commercial production in 1935 and is currently governed by the British Standard BS 877 : part 2 (1973) which was first published in 1939. However this was not recognised as allowable aggregrate for use in structures until the first publication in 1957 of British Standard CP 114 (1969), "The Structural use of reinforced concrete in buildings".

The first building frame of reinforced lightweight concrete in the U.K. was a three-storey office block built at Brentford, near London in 1958. Since then many structures have been built of precast or insitu, reinforced or prestressed, lightweight concrete; thus indicating that lightweight concrete has the same wide adaptability as normal weight concrete.

The use of lightweight aggregate in the U.K. according to the late 1970's figure showed in the order of 1.9 million m^3 per annum compared with some 50 million m^3 of natural dense aggregate used in normal weight concrete (Spratt, 1980). According to a recent publication (Dhir, et al, 1989) demand for lightweight concrete seems to have dropped due to high initial cost of materials and insufficient knowledge of the properties of lightweight concrete. However, demand is expected to increase again in the future due to growing scarcity of natural dense aggregate in this country and a growing awareness of the advantages to be gained by the use of lightweight concretes.

2.2 LIGHTWEIGHT AGGREGATES

Lightweight aggregates can be either natural or artificial materials. Natural lightweight aggregates are found only in some areas and their performance in fresh and hardened concrete is generally not satisfactory as compared to artificial lightweight aggregates. These materials have not been included in this study.

The most significant feature of manufactured lightweight aggregates is their high internal porosity, which is caused by internal expansion of gases during heating followed by a rapid cooling of the aggregate so that the gases can not escape. The existence of these pores is the prime reason for their low density.

The shape and structure of lightweight aggregate can be quite variable and will be a consequence of the processing techniques used in its production. Aggregates may be angular or highly irregular in shape, and this will affect the workability of concrete made with the aggregate. Lightweight aggregates have high absorption values, because of the large internal inter-connected porosity and this requires a modified approach to concrete proportioning.

The characteristics and assessment of lightweight aggregates for use in the U.K. are covered by British Standard BS

877 : part 2 (1973), BS 1165 (1985), BS 3681 : part 2 (1973) and BS 3797 : part 2 (1976).

2.2.1 Structural Lightweight Aggregates and their Production

BS 3681 : part 2 (1973) defines lightweight aggregates as aggregates having a loose bulk density not exceeding 950 kg/m^3 for coarse aggregate or 1200 kg/m^3 for fine aggregate when determined on the material oven dried at a temperature of 105 ± 5° C.

In the U.K., the commercial production of lightweight aggregates suitable for structural concrete is summarized in Table 2.1 including the approximate annual productions of lightweight aggregate manufacturers based on 1979 survey (Horler, 1980). Some of these materials, as indicated in Table 2.1, are now no longer available. These aggregates are suitable for load bearing structures in combination with steel reinforcement or in composite steel-concrete construction. The materials are chemically inert and can produce concrete of sufficiently high strength and low density. Structural lightweight aggregates are produced by expanding, foaming or sintering clays, shales and slate or industrial by-products such as pulverized fuel ash and blast-furnace slag. The production of Lytag 'sintered pulverized fuel ash' (Boral Lytag, 1987), Leca 'expanded clay' (ARC Conblock, 1987) and pellite 'pelletised expanded blast-furnace slag' (Tarmac Pellite, 1983) which have all been used in this present investigation are described in detail in the following sections. Typical samples of each type of these coarse aggregates are shown in Figure 2.1.

2.2.1.1 Lytag

Lytag is pelletised sintered pulverized fuel ash, P.F.A.. The P.F.A. is the residue from the combustion of pulverized coal used as fuel in modern power stations. It is a grey powder much resembling portland cement in finess and it consists of very tiny spherical particles. By heat treatment (about 1200° C) these small particles can be made to cohere forming pellets having considerable mechanical strength which may be used as lightweight aggregate.

The process of causing this cohesion is called sintering. The amount of fuel which is necessary for sintering to take place is about 8% (Kinniburgh, 1956). Although the P.F.A. contains some unburnt fuel it is unlikely to be as high as 8%. This fuel deficiency is corrected by the addition of coal dust in the form of slurry. The coal slurry is mixed with the P.F.A. in a screw mixer, and then the prepared material is fed into the pelletizers. The pelletizers consist of large, inclined pans which are rotated with a scraper blade. The moist mixture is fed onto the lower part of the pan, carried upward by the rotation and swept across by the scraper blade. A light spray of water suppresses dust. The mixture coheres into pellets which spill over the lip of the pan at the bottom onto a belt conveyor. The 'green' pellets are then transferred by means of a belt conveyor to a storage hopper where they are fed onto the moving grate of the sinter machine. The pellet bed thickness is kept constant at approximately 300mm. The top surface of the pellet bed is ignited by an oil fired ignition hood which has a temperature of about 1400°C. Once ignited, the pellets are in a partially sintered state and the bed continues to burn as it moves along the sintering machine, with combustion eventually progressing right to the bottom of the 300mm bed. By the time the pellets reach the end of the sintering machine they are fully sintered. After sintering, the pellets are hard and red-brown coloured. After cooling the pellets are screened and graded. Figure 2.2 shows a flow diagram of the manufacturing process of the Lytag aggregate.

2.2.1.2 Leca

Leca is lightweight expanded clay aggregate produced from a special grade of clay which is suitable for bloating. In the U.K., the best clay which could be considered for this purpose is London clay (Harrison, 1974).

The clay coming from the quarry is crushed and passed through grinding in a wet pan mill. Water with an additive which encourages bloating are added to it. The fine and plastic paste of clay is then forced through perforated plates in an extrusion press. The hole diameters are chosen according to the aggregate diameter required. The extruded clay paste is cut up into pieces of the required length and the pellets obtained are first dried in a rotary kiln giving them a round shape. The burning is then done, mainly in a rotary kiln, but sometimes a vertical shaft kiln is used as well. The kiln is fired by a mixture of pulverized coal and oil and reaches a temperature of about 1200°C. The material formed by this process consists of round particles with a dense skin and a honeycomb interior. The aggregates are brownish to reddish in colour. The aggregates are then cooled, screened and stored in different particle sizes. Figure 2.3 shows a simple presentation of the manufacturing process of Leca.

2.2.1.3 Pellite

Pellite is pelletised expanded blast furnace slag aggregate as a by-product of iron production. It is manufactured by passing a controlled molten slag (up to 3 ton per minute) direct from the blast furnace down a vibrating feed plate through water jets and then onto a rapidly rotating convoluted drum. The molten slag expands on contact with the water and is in a semi-plastic state when it hits the drum. Fins on the drum break the expanded slag into small particles as they are thrown through a water mist, forming them into semi rounded shapes with a smooth glassy surface. The bigger particles however tend to be irregular in shape and slightly more vesicular. After cooling, the material is screened and graded. The aggregates are light grey to buff in colour. Figure 2.4, diagramatically shows the process of manufacturing Pellite.

2.2.2 Properties of Lightweight Aggregates

The properties of lighweight aggregates depend on the raw material and the operating procedure used in their manufacture, and hence can differ greatly from each other.

The most notable property of all lightweight aggregates is their porosity which in turn affects many other properties. The amount of pores may vary from about 25 to 75% of the aggregate volume depending on the raw materials, the process of manufacture and the particle size of the finished material.

Each of the properties of lightweight aggregates may have some influence on the properties of the fresh and hardened concrete. However, it must be remembered that the properties of lightweight concrete, in common with those of normal weight concrete, are also greatly influenced by the quality of cement paste, i.e. water-cement ratio.

2.2.2.1 Bulk Density

Due to their porous structures, the bulk densities of lightweight aggregates are considerably less than those of natural dense aggregates which are usually of the order of 1600 kg/m^3. The bulk density varies with particle size, being highest for the fine particles and lowest for the coarse particles. The dry loose bulk density of U.K. structural lightweight aggregates varies from 350 to 950 kg/m^3 for coarse aggregate and 700 to 1200 kg/m^3 for fine aggregates. Figure 2.5 shows the typical bulk density of coarse lightweight aggregates available in different countries.

2.2.2.2 Water Absorption

Lightweight aggregates, due to their porous nature, have greater capacity for absorbing water than natural dense aggregates. The amount of water absorbed by the aggregate depends on the pore structure and the surface condition. The water absorption is time-dependent and also very much depends on the type of aggregate and to some extent on particle size.

Initially, it may be extremely fast or slow, but in either case, it continues for a long time. Based on a 24 hour absorption value, the water absorbed by lightweight aggregates may vary from 5 to 25% by weight of the dry aggregate (Spratt, 1974) as compared to about 2% for natural dense aggregate. The amount of water absorbed by the aggregates has a significant influence on mix proportions, handling and control of concrete.

2.2.2.3 Strength of Lightweight Aggregates

In a two-phase material such as lightweight concrete, the strength of concrete depends partly on the strength of aggregate contained therein. The strength of lightweight aggregate particles varies considerably with type and its source. Some particles may be strong and hard and others weak and friable. There are many ways which the aggregate strength can be determined (FIP, 1983), although British Standards do not include a strength test for lightweight aggregates, such as the 10% fines value for natural dense aggregate given in BS 812 : part 3 (1975). The 10% fines value test involves placing a known mass of aggregate in a standard cylinder and compressing the contents of the cylinder by a certain amount for 10 minutes under a uniform loading rate, followed by screening the contents after crushing. This test is generally not considered to be a good guide for the strength potential measurement of lightweight aggregates. However, for comparison with natural dense aggregate strength, Teychenné (1968) reports that the 10% values for U.K. lightweight aggregates are between 1 to 11 tons force as against 20 to 30 tons force for natural dense aggregate.

2.2.2.4 Modulus of Elasticity of Lightweight Aggregate Particles

The modulus of elasticity of concrete is a function of the moduli of its constituents, aggregate and matrix. Dynamic modulus of elasticity values from ultrasonic pulse velocity measurements (Muller, 1979) on lightweight aggregates show that usual lightweight aggregates have a range of 3-18 kN/mm^2 whereas for natural dense aggregates values may range from about 30 kN/mm^2 for quartz to 100 kN/mm^2 for basaltic rock (Holm, 1983).

2.3 LIGHTWEIGHT CONCRETES

Lightweight concretes made from suitable structural lightweight porous aggregates have been widely used in a variety of structures throughout the world. In the U.K., guidance and recommendations concerning their use are given by The Institution of Structural Engineers and Concrete Society (1987), and by BS 8110 : part 2 (1985) and also by the earlier codes namely CP 110 (1972), CP 114 (1969), CP 115 (1969) and CP 116 (1969). The basic feature of lightweight concrete is the low density which is 15-40% lower but with strength equal to that normally achieved by normal weight concrete. While density depends primarily on the density of the lightweight aggregate, it is also influenced by the cement, water and air content, and to a small degree by the proportions of coarse to fine aggregate.

The water-cement ratio law applies to concrete made with lightweight aggregate in the same way as to normal weight concrete. However, due to the high value of water absorption of lightweight aggregates and their variable rate of absorption, free water-cement ratio is not generally used in mix design. In lightweight concrete mix design it is therefore necessary to use the total water/cement ratio to account for the water absorption by the aggregate. This is primarily due to uncertainty of calculating that portion of the total water in the mix which is applicable to the free water-cement ratio. The water absorbed in the aggregate prior to mixing is not included as part of the cement paste, and a further complication is introduced by absorption of some indeterminate part of the water added at the mixer. However, it is quite probable that this absorbed water is available for continued hydration of the cement after normal curing has ceased.

In practice there are two very distinct types of lightweight concretes, the first is made of lightweight coarse and fine aggregates and called fully lightweight concrete, whilst the other is termed semi lightweight concrete in that the coarse aggregate is lightweight and all or part of the fine aggregate is replaced by natural sand. From the site point of view, there is little benefit in a partial replacement, because this will add to the complexities of control and full sand replacement is thus most commonly used in practice. Previous studies by Hanson (1964) and Pfefier and Hanson (1967) on the replacement of fine lightweight aggregate with natural sand have indicated that an improvement in mix characteristics and concrete properties may result, but this improvement is achieved at the cost of an increase in the density.

Some notable examples of these two categories of structural lightweight concretes are the Maintenance Hanger V at

Frankfurt am Main airport and the BMW Administrative Building in Munich which were made of fully lightweight and semi lightweight (with full sand replacement) concretes respectively. In current construction, the majority of lightweight concrete structures are made of semi lightweight.

2.3.1 Properties of Fresh Concrete

The most important requirement in providing a suitable structural lightweight concrete mix is that it should be workable so that it can be fully compacted. The shape and texture of the aggregate particles, and the coarse nature of the lightweight fine aggregate tend to produce harsh mixes, particularly with relatively lean mixes, which may result in poor workability. Surface condition of the aggregate (open or closed surface pores) has an influence on the properties of fresh concrete. Aggregates with spherical shape, smooth surface, and closed surface pores require relatively less mortar compared to those with crushed and angular surfaces with open surface pores. For the latter type of aggregate, the interlocking of the sharp-edged particles hinders the compaction of the concrete. In this case a very high mortar content is required because a portion of the mortar penetrates into the open surface pores. Also, due to high water absorption of lightweight aggregates when they are in contact with the cement paste, the workability will decrease from the time of mixing until the time of placing and compaction. In order to obtain a good workability at the time of placing, additional water must be added to the mix which must be related to the length of time between mixing and placing. However, if the additional quantity of water would be so great that the initial consistency of the concrete would be too wet, the aggregate can be pre-wetted. It must however be borne in mind that pre-wetting the aggregates results in increased fresh density, a reduction in frost resistance and strength, and increased creep and shrinkage (FIP, 1983).

Partial or complete replacement of fine lightweight aggregate with natural sand in the form of semi lightweight concrete was found to assist in promoting workability of the concrete. A general appreciation of the effects of the natural sand replacement on fresh lightweight concrete may be obtained by referring to the work by Hanson (1964) and, Mayfield and Louati (1990). Replacing the complete fine lightweight aggregate with natural sand tends to improve cohesiveness and workability of the mix along with slightly lower cement content requirement. In semi lightweight concrete, due to low water absorption, there is a lower water demand in comparison with fully lightweight concrete. For the case of complete replacement of fine lightweight aggregate, a reduction in water content of 12-24% was reported (Hanson, 1964). Sand replacement, however, will increase the difference in density between matrix and coarse aggregate which may cause segregation (FIP, 1983). Air entrainment was found to prevent segregation of the matrix from the coarse lightweight aggregate particles and 6% air content was proposed by Hanson (1964).

2.3.2 Properties of Hardened Concrete

A range of significant properties of hardened lightweight concrete such as strength, modulus of elasticity, dimensional stability, durability and permeability are considered. In lightweight concrete in general, the behaviour of these properties can be viewed in a similar way to normal weight concrete, although quantitatively there are significant differences. Figures 2.6 to 2.9 summarize some important hardened properties for a number of typical fully and semi lightweight concretes.

2.3.2.1 Compressive Strength

The normal compressive strengths required by the construction industry for usual design purposes can be obtained with a variety of structural lightweight aggregates (Swamy, 1978). The compressive strength development of lightweight concrete, as in normal weight concrete, is related to the water/cement ratio (Figure 2.6) and test results have shown that strength of lightweight concrete is less sensitive to changes in water/cement ratio than normal weight concrete (Dhir et al, 1989a). Differences in properties of particular lightweight aggregates may require different cement contents for a given compressive strength of concrete. In general, for similar compressive strengths, a higher cement content is required with lightweight concrete compared to normal weight concrete and this is more apparrent at higher strength levels.

Aggregates normally have a maximum achievable strength ceiling, above which further additions of cement content would not appreciably increase the concrete strength (Holm, 1980). This ceiling strength is dependent on the vitreous material and the quantity, size, shape and the distribution of the enveloped pores. The strength ceiling generally can be increased at the given cement content by reducing the maximum size of the coarse aggregate. This effect is more pronounced for the weaker and more friable aggregates (Spratt, 1974). In the U.K., among the different types of lightweight aggregate, Lytag and Leca were found to be the strongest and the weakest respectively. Teychenné (1967) showed that Lytag is capable of producing concrete with a cube strength in excess of 60 N/mm^2 whereas in concrete made with Leca a ceiling strength of $30N/mm^2$ was found. However, apart from Leca, there is no reliable correlation between aggregate strength and concrete strength and lower strength of lightweight aggregate does not necessarily produce a lower concrete strength. For example, Teychenné (1967, 1968) showed that Aglite with lower crushing strength would produce concrete strength comparable to Lytag, and similar observations have been made by Evans and Hardwick (1960), and Dhir et al (1989).

The improvement in compressive strength of lightweight concrete from replacement of fine lightweight aggregate with natural sand has been investigated by many researchers, such as Hanson (1964), Teychenné (1967), Louati (1988) and Dhir et al (1989a). A variety of concretes made with different types of manufactured coarse lightweight aggregate in conjunction with natural sand from different sources have been covered by these researchers. Hanson (1964) showed an increase in compressive strength of up to 35 per cent for complete replacement of fine lightweight aggregate with natural sand. Recent investigations by Dhir et al (1989a) showed that for a given estimated free water/cement ratio, higher compressive strength would be obtained for semi lightweight concrete compared to fully lightweight concrete and this is more pronounced at lower water/cement ratios.

Strength design of concrete and the assessment of the quality of concrete will normally be judged on the basis of the compressive strength reached at the age of 28 days. Although, with a view to rapid rate of construction, the strength at an earlier age is also often of importance. On the other hand, in some cases the strength at a later age can be of interest.

In the short term, for low strength design, the early age strength development of lightweight concrete is similar to normal weight concrete. However at high strength levels, the rate of strength development in lightweight concrete is much higher than in normal weight concrete. This is because of the higher cement demand of lightweight concrete.

The 28-day compressive strength related to curing conditions shows that in many cases where the aggregate is used in a dry condition, dry curing provides better strength development than companion water cured concrete (Teychenné, 1967), (Dhir et al, 1989a). Bandyopadhyay (1974) carried out an investigation on solite lightweight concrete and found that the air dry-cured strength is lower than the water cured strength in the case of 28 day compresssive strengths less than 41 N/mm^2, but for higher compressive strengths, the air cured strength is higher (1-9%) than the wet cured. He also studied the effect of pre-wetted aggregate on the compressive strength of solite concrete, and concluded that the compressive strength is reduced up to 8% at 28 days.

In the long term, the strength gain of lightweight concrete is generally higher than normal weight concrete. The rate of strength gain after 28 days for dry and wet curing is in just the opposite sense to the short term behaviour. Swamy and Lambert (1983) investigated the strength development of semi lightweight concrete made with Lytag subjected to wet and dry curing conditions. For water curing, a maximum increase in strength between 10-40% at about 1-year and for dry curing an increase of 1-12% after two years were obtained. Similar long term strength development were observed by many other investigators, for example: Bandyopadhyay (1974), Balendran (1980) and Louati (1988).

2.3.2.2 Tensile Strength

The assessment of the tensile strength of concrete may be obtained by two standard methods, namely, the splitting and the flexural tensile tests. Both of these two test methods measure the tensile strength of concrete in an indirect manner. The tensile strength values obtained from each test are not equal, and the latter method yields a higher value. The tensile strength is an important criterion in estimating the load under which cracking will develop. The appearance of a crack in lightweight concrete is quite different from that in normal weight concrete. In lightweight concrete under tension the fracture is caused by tensile stresses in the aggregate particles as well as by fracture of the matrix, since the tensile strength of the aggregate is usually less than that at the matrix. In the case of normal weight concrete, the fracture commonly occurs by breaking the bond between the matrix and the surface of the aggregate.

The tensile strength of concrete is more sensitive to different curing conditions than the compressive strength. For continuously wet curing, Hanson (1961) has indicated that the wet tensile strength of lightweight concrete falls in a rather narrow range, whereas wide variation is found for dry tensile strength. Under wet curing, research carried out by Hanson (1961), Pfiefer (1967) and Teychenné (1967) showed that the tensile strength of lightweight concrete is comparable to that of normal weight concrete of similar compressive strength. However FIP (1983) reports more recently that with most lightweight aggregate, the tensile strength of concrete is less than that of normal weight concrete for the same compressive strength under wet curing. Similar trends have also been reported by Balendran (1980) and Louati (1988). The tensile strength of lightweight concrete under dry curing, which is more relevant to behaviour of concrete in real structures, is considerably lower due to non-uniform moisture loss. The reduction in tensile strength is much more significant at higher strength levels. Hanson (1961) showed that dry curing of lightweight concrete decreases the tensile strength by amounts up to 40% depending on the aggregate type and compressive strength. This loss in tensile strength is found to be higher for flexural tensile strength than for splitting strength, however with time, concrete tensile strength may be regained as the specimen becomes uniformly dry, (Swamy and

Lambert, 1983).

Replacement of fine lightweight aggregate with sand was found to have only a small effect on the tensile strength for wet curing (Pfeifer, 1967). However, it will increase the tensile strength of lightweight concrete when the concrete is subjected to dry curing (Brewer et al, 1962), (Ivey and Buth, 1966), (Pfeifer, 1967).

2.3.2.3 Modulus of Elasticity

The modulus of elasticity used in concrete design may be determined by two standard methods, namely, the static and the dynamic test methods. The static modulus of elasticity is the measurement of chord modulus which is the slope of the line drawn between two points on the stress-strain curve. The dynamic modulus of elasticity is likely to represent the initial tangent modulus which is the slope of the tangent to the stress-strain curve at the origin. The dynamic moduli is of little practical significance, since it applies only to small stresses and strains. However it may be converted to the more practically useful static modulus by means of linear relationship which will be discussed later in section 5.4.2. The dynamic moduli of elasticity measurements are relatively higher than the static moduli of elasticity because the concrete is subjected to very small displacements.

The modulus of elasticity of concrete depends on the modulus of elasticity of its components, that is, the aggregate and the hardened cement paste and their relative proportions

(Figure 2.8). The moduli of elasticity of lightweight aggregates are much lower than those of natural dense aggregate. Thus the modulus of elasticity of lightweight concrete is generally much less than that of normal weight concrete. Typically, this ranges between about one-third to two-thirds that of normal weight concrete (Short and Kinniburgh, 1978). The exact value depends on the nature of the aggregates used, the compressive strength of the concrete, and on its density. Shideler (1957) evaluated eight lightweight aggregates produced in the U.S. and compared them with one normal weight concrete. He found that the modulus of elasticity of the lightweight concretes was from 53 to 82% of the modulus of the normal weight concrete of 24 kN/mm^2 at 28 days, and from 44 to 63% of the modulus of the normal concrete of 35kN/mm^2 at six months. Comparison of those lightweight concretes available in the U.K. with normal weight concrete, by Teychenné (1967) showed that foamed slag concrete has a modulus of elasticity of about 70% of that of normal weight concrete. For Aglite and Lytag concretes, the corresponding value varies between 50 to 60%. Also, Swamy and Bandyopadhyay (1975) carried out an investigation on the elastic properties of solite lightweight concrete and they found that the modulus of elasticity of this was about 62.5% of the normal weight concrete. The latter investigators also considered the effect of curing condition and age on the modulus of elasticity and reported that this is not very significant. The increase in modulus of elasticity after 28 days is marginal at about 5%.

The replacement of the fine lightweight aggregate with natural sand increases the modulus of elasticity of lightweight concrete (see Figure 2.8). Hanson (1964) observed that full replacement of fines with natural sand will increase the modulus of elasticity by 10 to 30%, depending on the aggregate used and the compressive strength level.

2.3.2.4 <u>Shrinkage and Creep</u>

Concrete is not dimensionally stable and it is subjected to time-dependent deformation under shrinkage and creep. Shrinkage is defined as the change in deformation of an unloaded specimen as a result of moisture loss while creep is the increase in deformation under sustained load. They can both result in appreciable loss of prestress in prestressed concrete elements and may reduce the tensile strength of concrete as well as affect the long term deformation and warping.

Shrinkage and creep are often attributed to the cement paste. However, concrete with dense natural aggregate shows relatively smaller shrinkage and creep as the paste movements are restrained by the rigidity of the aggregate. In lightweight concrete much less restraint is imposed by the aggregates due to their lower modulus of elasticity and this may be expected to lead to higher shrinkage and creep. Many investigators (Evans and Patterson, 1967), (Brooks and Neville, 1975, 1978), (Swamy and Ibrahim, 1973), (Dhir et al, 1989a), have, however shown that in practice shrinkage and creep in lightweight concrete may be either greater or less than for normal weight concrete.

Shrinkage and creep are mainly affected by the quantity and quality of cement and aggregate, and in the case of creep the stress/strength ratio. The effect of cement content and water-cement ratio on shrinkage and creep have been investigated by Lambert (1982) using Lytag semi-lightweight concrete. He discovered that an increase in cement content and a decrease in water-cement ratio may result in an increase in shrinkage by approximately 50% and decrease in creep by 25%. Similar behaviour has also been observed by Teychenné (1967), Pfeifer (1968), Bandyopadhyay (1974) and Balendran (1980). Figure 2.9 shows the dependency of shrinkage of lightweight concrete on cement content and aggregate type.

The effects of aggregate on shrinkage and creep properties of lightweight concrete is a phenomenon of great importance. Pfeifer (1968) showed that an increase in coarse aggregate volume reflects reduced cement paste and fine aggregates which also reflects reduced shrinkage and creep, and the reduction of these two latter properties could be as high as 30%. The significance of stiffness and shrinkage of aggregate on the shrinkage properties of lightweight concrete have been considered by Hobbs (1974). He showed that the effect of change in aggregate stiffness has a large influence on shrinkage. Also, he stated that lightweight aggregates of similar stiffness can nevertheless produce concretes of markedly different shrinkage behaviour as a result of differing aggregate shrinkage characteristics.

The effect of specimen size upon shrinkage and creep has been considered by Arnaouti and Sangakkara (1984). They concluded that for shrinkage at the short term stage, the rate of shrinkage for the smaller size of specimen, is higher, as expected in normal weight concrete. However, at the later stage (after 4 months), different specimen sizes were equalized to similar shrinkage values. Creep is affected to a lesser extent. This feature has also been observed elsewhere (Gamble and Parrott, 1978) on normal weight concrete, hence, it has been suggested that for practical purposes the size effects on creep can be ignored.

Creep may be significantly reduced by low pressure steam curing and very greatly reduced by high pressure steam curing. The reduction for low pressure steamed cured concrete made with American lightweight aggregate may be from 25-40% of the creep of similar concrete subjected to moist curing (ACI, 1979). The corresponding values for high pressure steam cured concrete may be from 60-80%. Similar influences may reduce shrinkage by as much as 40% for steam curing.

Partial or full replacement of the fine lightweight aggregate by natural sand usually reduces shrinkage (Figure 2.9) and creep for concrete made with most lightweight aggregates (Hanson, 1964), (Pfeifer, 1968). The amount of this reduction appears to a great extent to be proportional to the reduction of total water and cement content, that is, to the reduction of paste in the concrete. Pfeifer (1968) reported that for semi lightweight concrete with full replacement of the fines, the shrinkage and creep reduction could be 30% at compressive strength level of $35N/mm^2$. However it seems that for some types of aggregate, such as solite, sand replacement has no significant reduction in the shrinkage or the creep properties (Swamy and

Ibrahim, 1973).

2.3.2.5 Durability

The durability of internally sound concrete may be defined as its ability to resist adverse external influences of environmental conditions (such as climatic), fire, chemical attack and mechanical damage. Lightweight concrete, in spite of using aggregate with a cellular structure, has been shown to exhibit adequate durability in many cases.

2.3.2.5.1 Frost Resistance

The resistance of concrete to freezing and thawing is of particular importance when considering its use in exposed applications. Concretes made with natural dense aggregates have been shown to have excellent resistance to this type of weathering when they contain air entraining agents. Similarly, air entrainment provides a high degree of protection to lightweight concretes exposed to freezing and thawing (Klieger and Hanson, 1961). The same authors found that most lightweight concretes used in the investigation exhibited durability independent of air entrainment if dry aggregate was used, as opposed to normal weight concrete where this is not the case. The possible reason for this might be due to the porosity of the aggregate particles, which has a similar effect to entrained air voids. A study by Dhir et al (1989b) on non-air entrained concretes incorporating two British lightweight aggregates (Lytag and Aglite) and one natural dense aggregate showed that the lightweight concrete is potentially more durable than the normal weight concrete. Also of the two lightweight aggregates, the test results clearly showed that the Lytag is superior to Aglite. This is probably due to differences in internal structures where the higher cement demand of Aglite due to angularity of the aggregate with open surface as discussed in section 2.3.1. masks the effect of the aggregate structure.

The beneficial effect of sand replacement for fine lightweight aggregate on the freezing and thawing resistance of lightweight concretes were discussed by Pfeifer (1967a). The results showed that the use of natural sand in partial or complete replacement significantly improved the freezing and thawing resistance of low strength lightweight concretes e.g. $20N/mm^2$. However at higher strength levels such as $35N/mm^2$ the latter investigator showed that all lightweight concretes whether fully or semi lightweight concrete were highly durable, and the use of natural sand provided only minor improvement in this case.

2.3.2.5.2 Permeability

The permeability of a concrete may indicate the likelihood of moisture penetrating as far as the reinforcement and promoting serious corrosion, or of the danger of harmful solutions getting into the body of the concrete and setting up damaging chemical reactions.

The water absorption of lightweight concrete is generally higher than for normal weight concrete (ACI, 1979). However, high absorption does not necessarily mean poor durability or high permeability for concrete. This has been confirmed by Klieger and Hanson (1961) who have shown that there is little relationship between the water absorption properties of lightweight concretes and their frost resistance. Permeability of concrete depends primarily on the nature of the matrix and less on the porosity of the aggregate. A recent investigation by Dhir et al (1989b) on the permeability of lightweight concretes showed that lightweight concrete can achieve equal or lower permeability than normal weight concrete.

2.3.2.5.3 Thermal Insulation and Fire Resistance

One of the striking features of lightweight concrete is its high thermal insulation. This phenomenon is brought about solely by the cellular nature of the aggregate. As the density of the material also is dependent upon the cellular nature of the aggregates, it follows that there must be a relationship between density and thermal conductivity due to low conductivity of air. In the case of lightweight concrete the lower conductivity and the inherent fire stability resulting from the manufacture of lightweight aggregate, already heated to over 1000^{o}C, provides better fire resistance than normal weight concrete. Therefore, in a fire, lightweight concrete is less liable to spalling of the cover over the reinforcement.

2.3.2.5.4 Corrosion and Carbonation

Cellular lightweight aggregate has an adverse effect on the degree of protection against corrosion afforded to the steel reinforcement by the concrete cover. For the corrosion of steel to take place, the conditions necessary are access to the steel for oxygen and moisture. Also a third condition which has to be satisfied for corrosion is an environment which is not markedly alkaline, that is, with a pH value less than 11, means the environment must be acidic or mildly alkaline. Grimer (1967) carried out an investigation on the durability of steel-embedded in concrete incorporating five British lightweight aggregates, namely, sintered pulverized fuel ash (from two sources), foamed slag (from two sources) and expanded clay and one type of natural dense aggregate. He showed that carbonation is a key factor in corrosion of steel bars. When the alkaline environment around the bars is neutralized by the carbonation of the lime present in concrete, rusting can start.

Grimer's paper showed that a major factor affecting the depth of carbonation was the proportions of the mix. For example, the depth of carbonation for a given aggregate - cement ratio of 9 was about seven times that for the companion ratio of five. Two other factors, but of lesser importance were shown to be the sand replacement level and the type of aggregate. The inclusion of natural sand as fine aggregate reduced the depth of carbonation by about 25%. A similar effect has also been observed by Swamy and Bandyopadhyay (1975). The depth of carbonation for the semi sintered pulverized fuel ash aggregate concretes was smaller than for the other semi lightweight concretes, and was similar to the normal weight concrete. Other investigations by Dhir et al (1989b) on the depth of carbonation of Lytag, Algite showed that semi Lytag aggregate concrete exhibits greater depth of carbonation than semi Aglite aggregate concrete and they also claimed that at high strength level (e.g. $50N/mm^2$), these values are less than the depth of carbonation in normal weight concrete.

In investigations by Grimer (1967) and Swamy and Bandyopadhyay (1975), it was observed that in some cases the larger pieces of aggregate near the edge of the concrete specimen often provided a short circuit path for the advancement of the carbonation front. For this reason in lightweight concrete, the ratio of the concrete cover thickness to maximum aggregate particle size should be greater than 1. Swamy and Bandyopadhyay (1975) specified the depth of concrete cover to embedded steel reinforcement should be at least equal to the maximum aggregate size plus 5mm. Also BS 8110 : part 2 (1985) provides an additional 10mm cover when lightweight aggregate is used in reinforced concrete.

2.3.3 Economy of Lightweight Concrete

There is often economy in the use of lightweight concrete in place of normal weight concrete. However manufactured lightweight aggregate usually costs more than natural dense aggregates because of the manufacturing process and the energy used in production. It therefore follows that the cost per unit volume of lightweight concrete will be more than for normal weight concrete. In comparing costs, however it is unjust merely to compare the cost per unit volume of unplaced concrete. Instead, to obtain a true economic assessment, a comparison of final structural cost must be made which incorporates all the properties of lightweight concrete such as bulk density, thermal insulation and fire resistance.

The low density of concrete can provide an obvious saving in transportation. From the construction point of view, the low density of lightweight concrete will reduce the dead weight of the structure and this will provide some savings in the cost of foundations, reinforcement and overall size of some elements of the structures. In addition, for casting insitu lightweight concrete, the smaller dead weight of fresh concrete will reduce the formwork strength requirements. From the structural design point of view, bending moments are directly proportional to the total load and it seems that the use of lightweight concrete for slabs and beams can give a significant saving compared to use of normal weight concrete. It is however, of little benefit to use lightweight concrete in columns, because the weight saving is small and the lightweight concrete itself is more expensive.

The increased heat insulation and fire resistance of lightweight concrete generally requires less thickness of material, although this benefit may be reversed for other reasons (see section 2.3.2.5.4). However they often have a favourable indirect effect on buildings, particularly in relation to running and maintenance costs.

A study initiated by the Concrete Society (1983) compared the cost of an eight-storey office block in Central London designed in both lightweight and normal weight concretes. The results showed that in terms of the measurable direct cost of total construction, the use of lightweight concrete was marginally cheaper (0.2%). The same philosophy was also applied for a composite concrete beam and slab bridge with a span of 25m (Concrete Society, 1986). Taken on balance, the lightweight concrete solution showed a saving of about 1.5% over the normal weight concrete. The report also indicated that savings of about 4% are likely for a 60m span and 8% for a 200m span. A similar study, but on a long span bridge was taken in the U.S.A. (Bender, 1980) where a multi-span precast segmental post-tensioned box girder bridge showed a cost saving of 18% and 6% on the super-structures and sub-structures, respectively.

These investigations, clearly refute the view that lightweight concrete construction is more costly than normal weight. On the contrary they demonstrate that it represents value for money.

Table 2.1:Production of lightweight aggregate in U.K.-1970
(Horler,1980)

Aggregate	Raw Materials	No. of Plants	Production m^3x1000
Aglite	Blended colliery shale	1	200
Foamed Slag	Blast furnace slag	4	300
Pellite	Blast furnace slag	2	300
Leca	Clay	1	250
Lytag	Pulverized fuel ash(P.F.A.)	3	550
Sintag+	Colliery shale	1	200
Taclite+	P.F.A. and furnace clinker	1	150
Solite+	Slate	1	--

+;These are no longer manufactured in the U.K.

Figure 2.1: Typical samples of lightweight aggregates

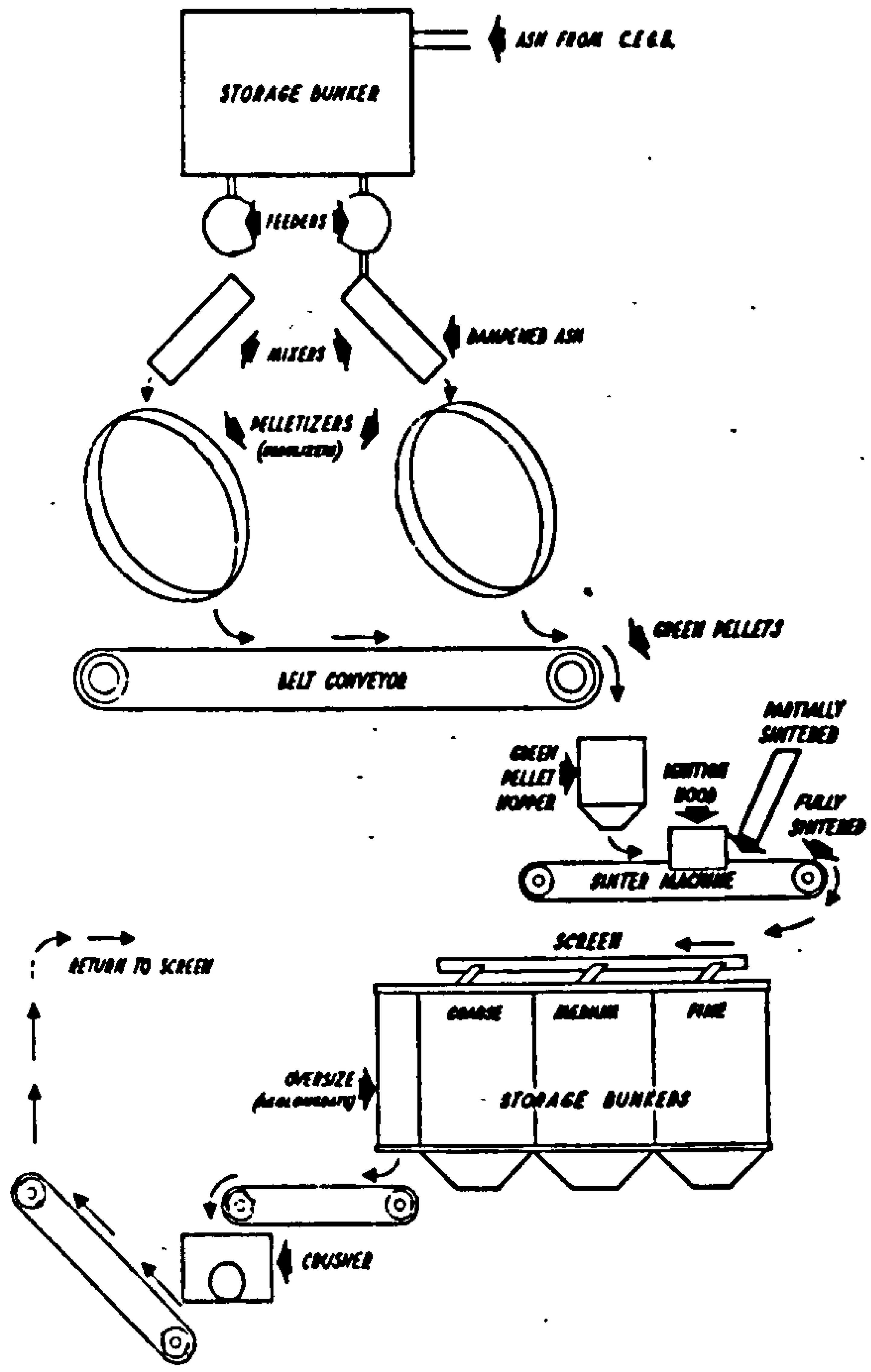


Figure 2.2: Manufacture of Lytag (Anon, 1967)

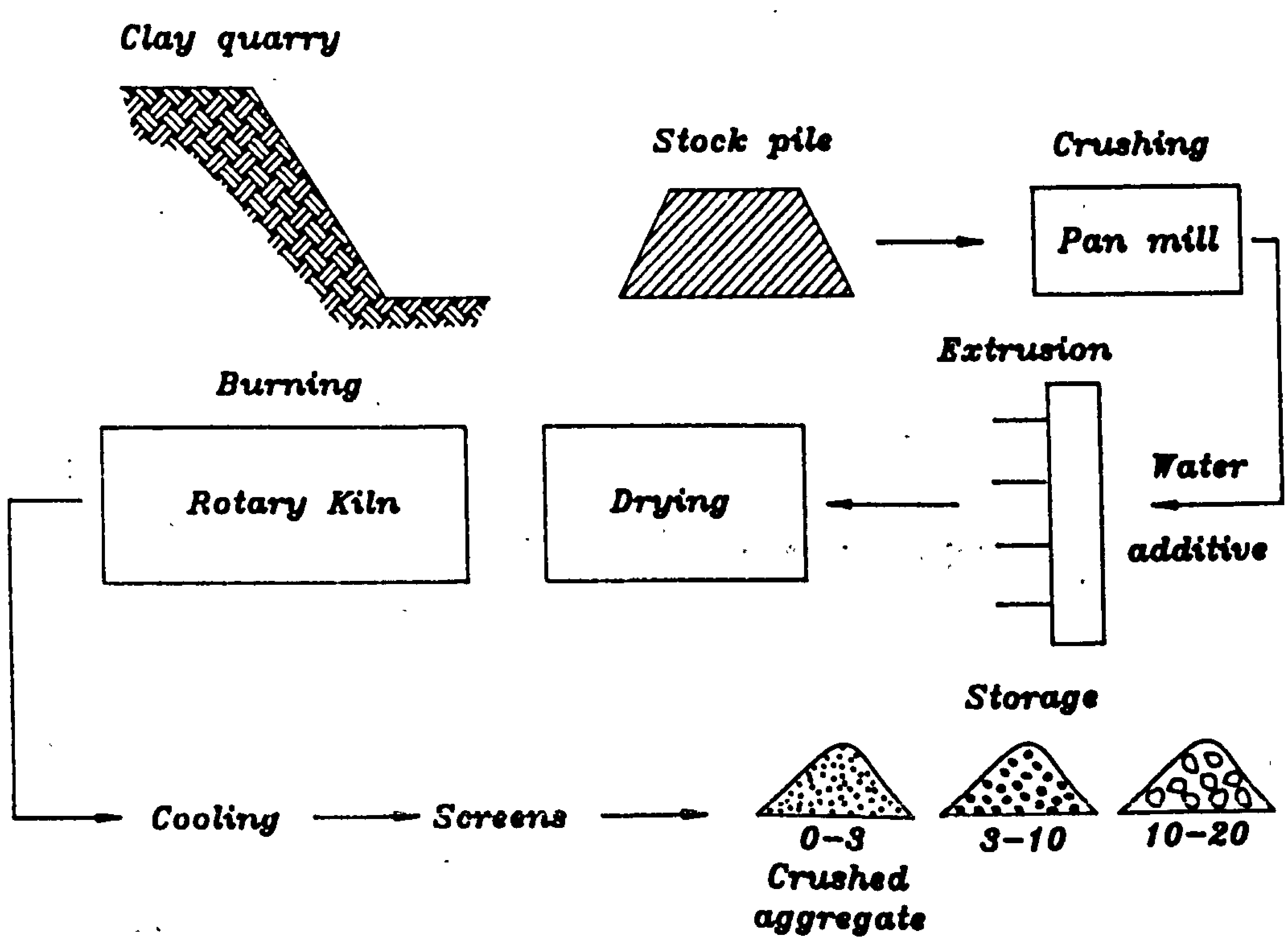
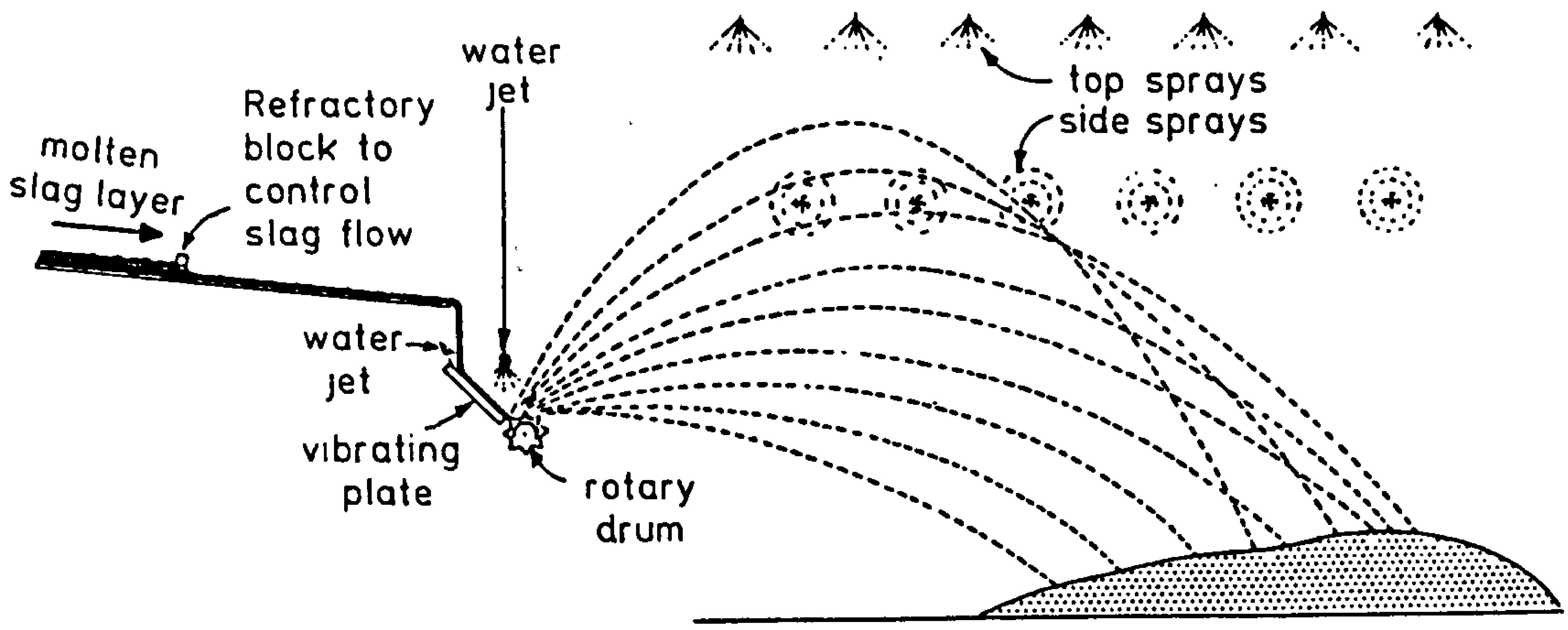


Figure 2.3: Manufacture of Leca (based on Venuat, 1980)



2.4: Manufacture of Pellite (based on Louati, 1988)

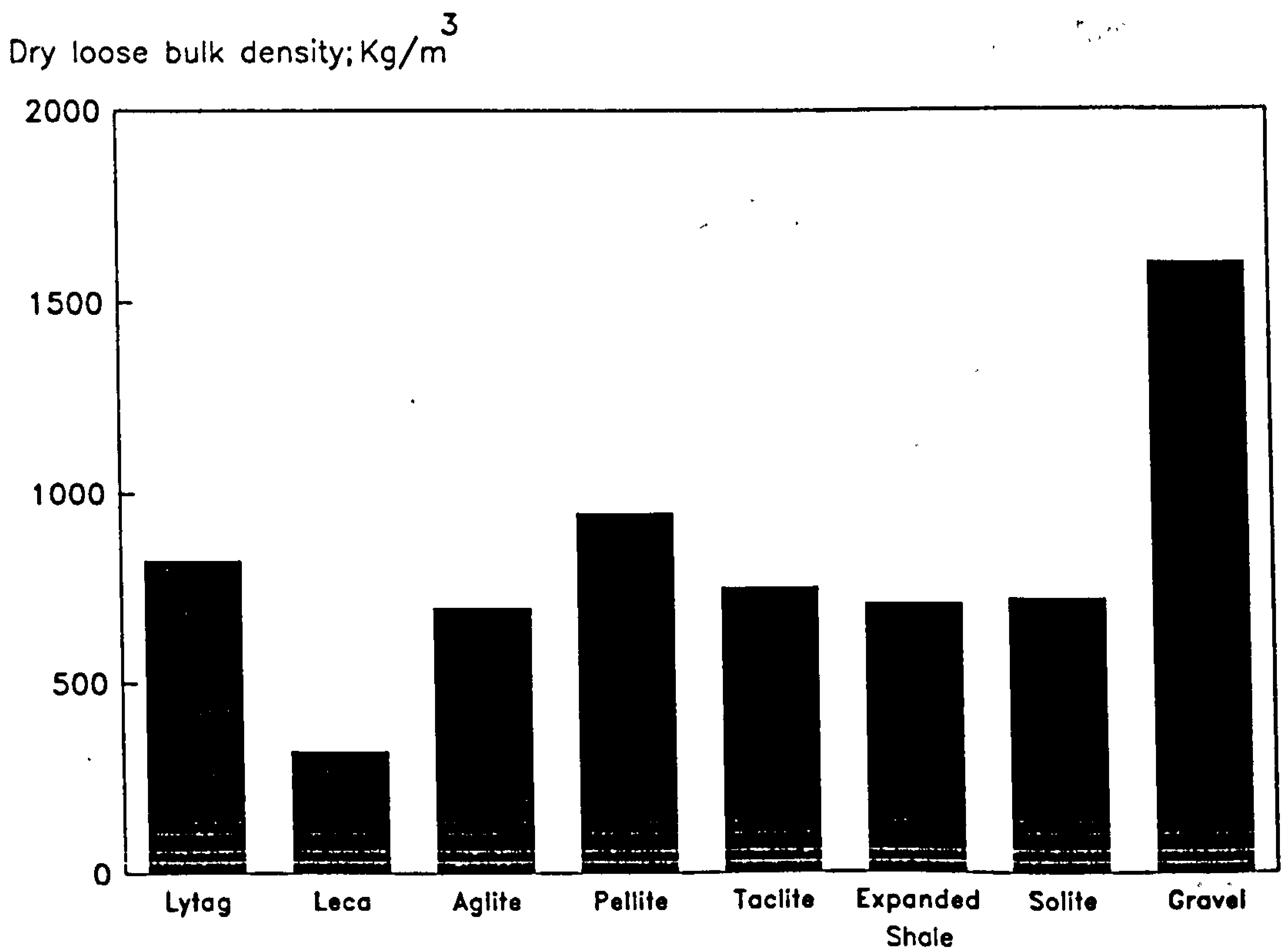


Figure 2.5:Typical dry loose bulk density of some aggregates

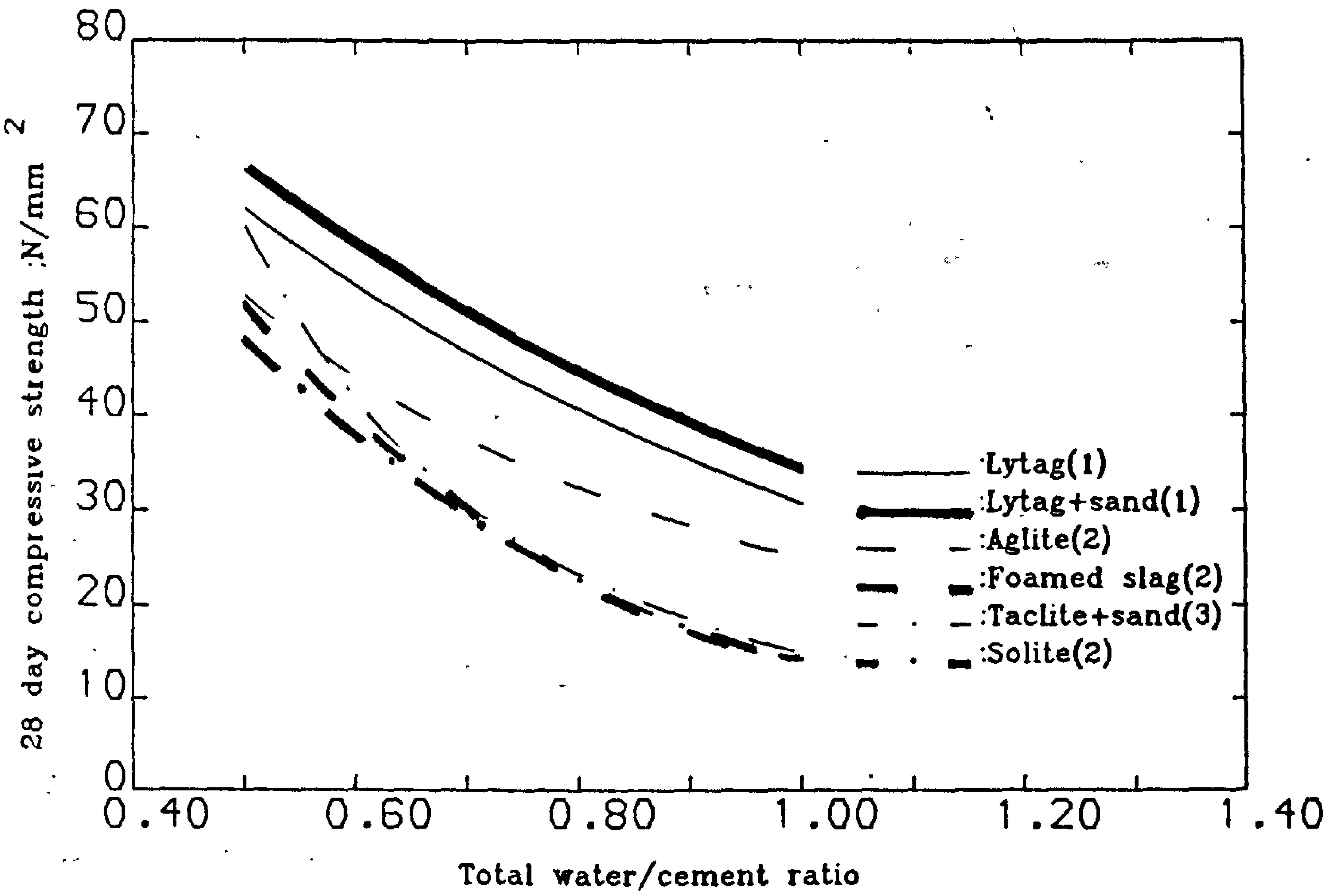


Figure 2.6:Typical relation between 28 day cube strength and total water/cement ratio

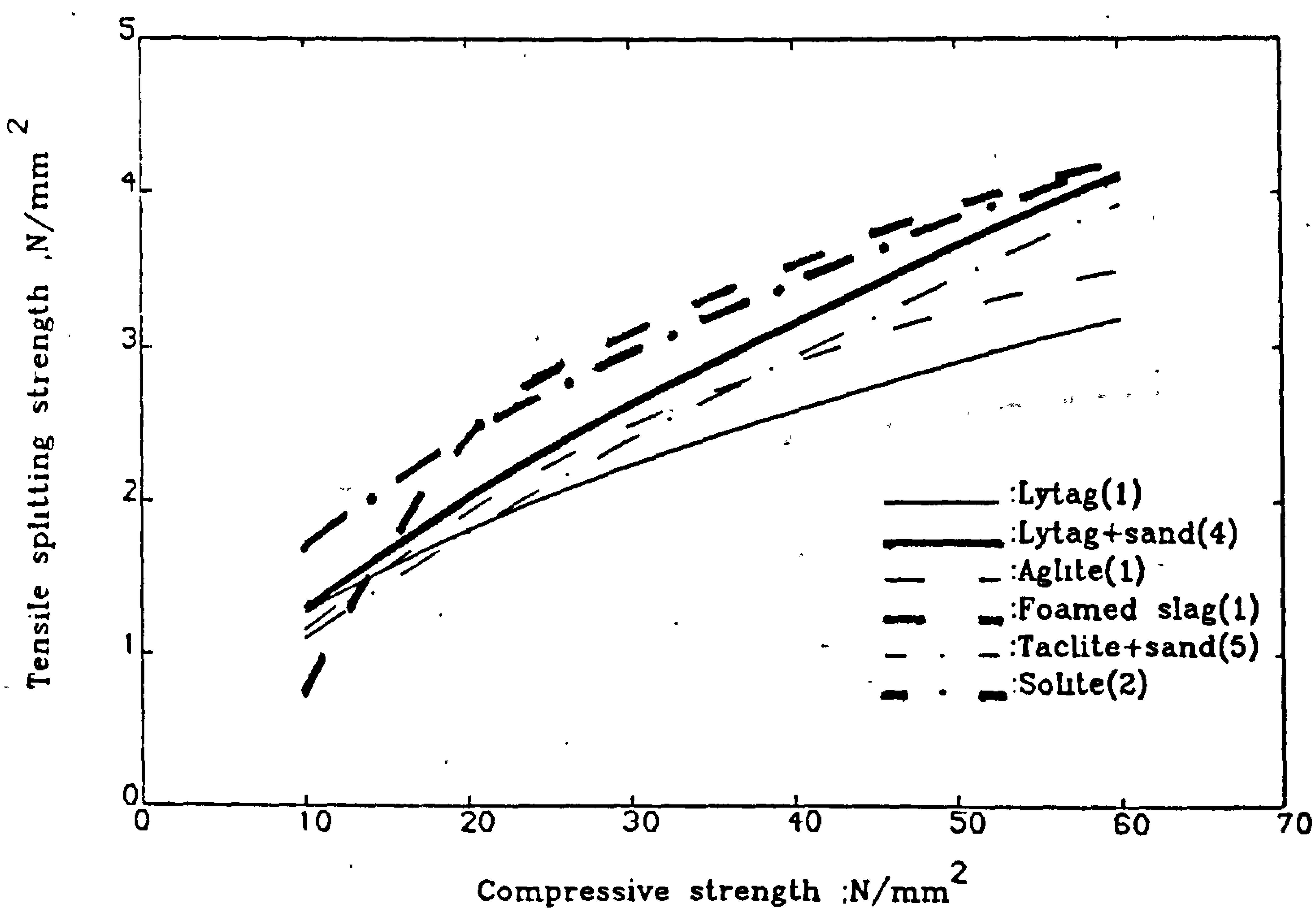


Figure 2.7:Relation between tensile splitting and compressive strengths in wet curing

(1):Teychenné(1967)   (2):Bandyopadhyay(1974)
(3):Owens(1971)   (4):Swamy and Lambert(1983)
(5):Balendran(1980)

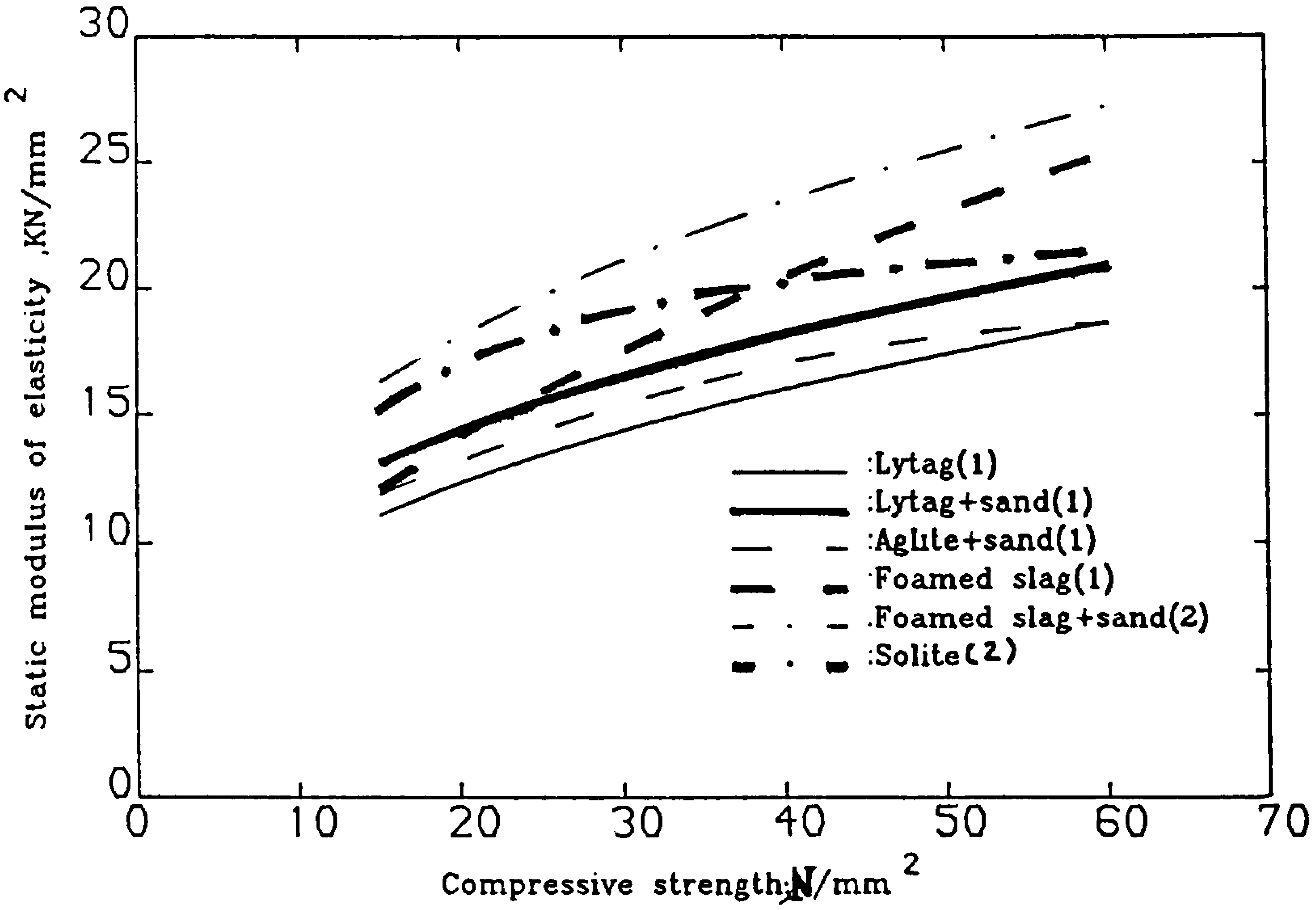


Figure 2.8:Relation between static modulus of elasticity and compressive strength

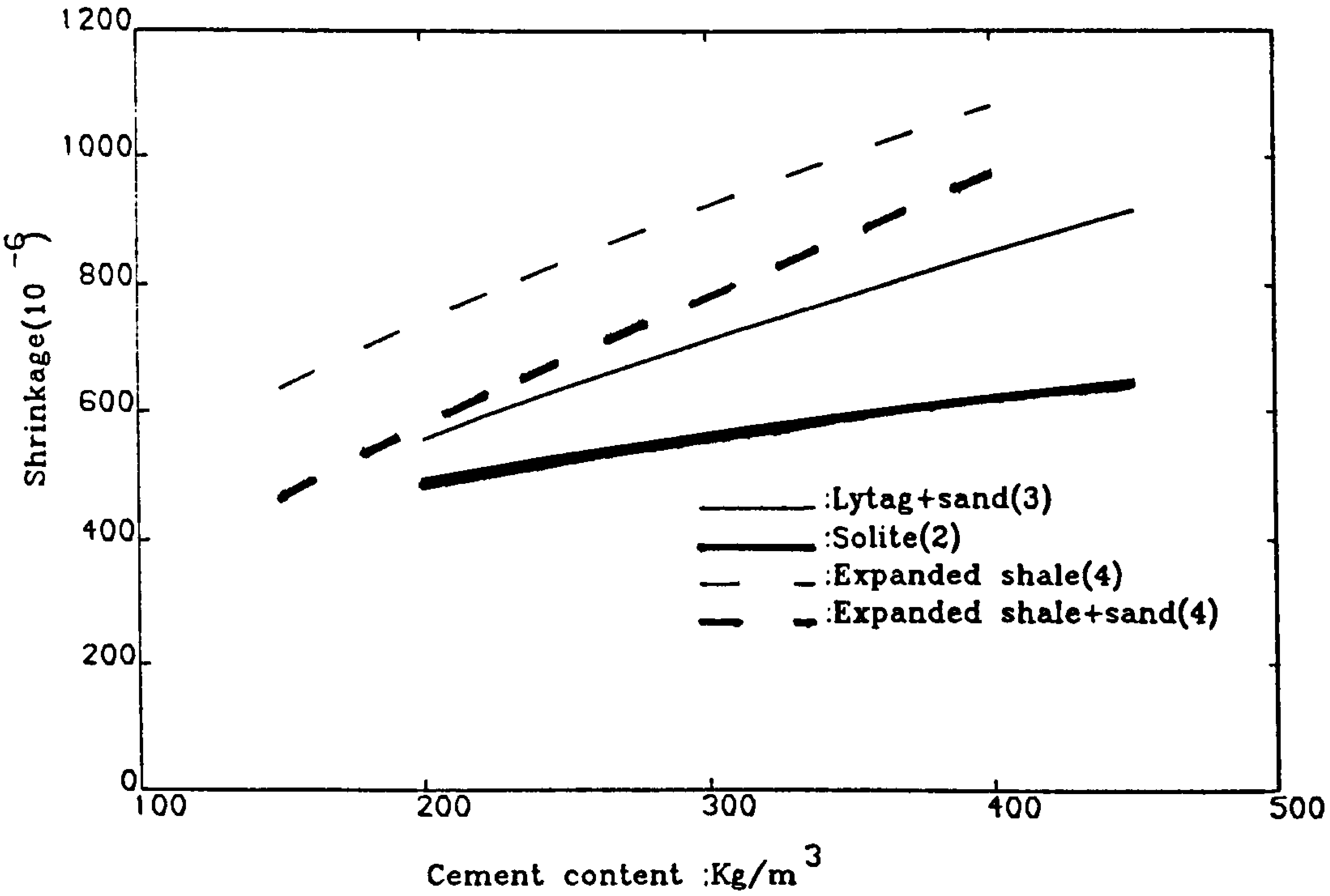


Figure 2.9:Variation of shrinkage with cement content

(1):Teychenné(1967) (2):Bandyopadhyay(1974)
(3):Lambert(1982) (4):Hanson(1964)

CHAPTER 3. INSITU CONCRETE STRENGTH TESTING

3.1 INTRODUCTION

The shortcomings and limitations of the conventional standard cube strength test as a measure of the quality of the concrete led to the development of methods for testing structural concrete insitu. Insitu testing of concrete was started in the early 1930's (Jones, 1970), but the greatest progress was achieved in the last two decades, (Malhotra, 1976, 1984), (Bungey, 1989).

The main objective, as applied to newly cast or old concrete, is to provide a reliable estimate of the quality of the concrete in a structure without relying solely on results from test specimens which are not necessarily representative as discussed in chapter one. The properties taken to define concrete quality are normally strength or durability. Attention is concentrated upon strength throughout this thesis.

When considering where to carry out the test in relation to the body of concrete element, it is well known that concrete at the bottom of a lift will be subjected to more compaction under the self weight of the concrete above it and suffer less bleeding than the concrete at the top surface (Bungey, 1989). Thus the location of a test in relation to the depth of a lift has a significant effect on its measured strength and suggestions have been made that whenever possible the test should be carried out from the mid-height of a lift (Munday and Dhir, 1984).

The insitu test measurement may be applied to determine the insitu cube strength or the potential strength. The insitu cube strength is the strength of the concrete as it exists in the element at the time of testing. The potential strength is defined as the strength of the concrete as it would have been at 28 days after being made into cubes, cured and tested in accordance with BS 1881 : part 116 (1983). A literature survey was carried out in Concrete Society Technical Report No. 11 "CSTR11" (1976) on the relation between the insitu cube strength and the potential strength. On average it has been recommended that the insitu cube strength, other than that near the top of a lift is about 70% of the potential strength.

This chapter deals with a number of insitu test methods, namely rebound hammer, ultrasonic pulse velocity, Windsor Probe, pull out, B.R.E. internal fracture, direct pull internal fracture, pull off and core test methods. These are categorized in three groups as non-destructive, partially destructive and destructive methods. A brief description of each test method has been given and major factors which influence the test results are outlined based on available data for normal weight concrete. A further discussion on lightweight concrete has been given in Chapter 6. Among these test methods, the non-destructive test methods are generally not recommended for absolute insitu strength estimation.

3.2 NON-DESTRUCTIVE METHODS

As the derivation of strength involves destructive stresses, it is clear that non-destructive tests cannot measure any strength parameter of a material. In general, therefore, these tests involve the measurement of some other material property, such as surface hardness which can be obtained by non-destructive means and may subsequently be used to estimate the strength through careful calibration. However, these empirical relationships between measured property and insitu strength are not universally applicable and must be qualified when other properties of the concrete such as mix proportions, type of aggregate, etc., are changed.

3.2.1 Rebound Hammer

The rebound hammer was applied to concrete in 1948 by a Swiss engineer named Ernst Schmidt (1950) although a similar method used for metals had been known since 1911 (Shore, 1911). The test gives a measure of the hardness of the concrete surface based on the rebound of a spring-loaded mass hitting a steel plunger in contact with the concrete surface. The extent of the rebound is measured and is designated as the rebound number. The plunger must always be normal to the surface of the concrete and the test may be made on concrete surfaces inclined at any angle, although the rebound number will be affected by the orientation of hammer on concrete. This is due to the action of gravity on the travel of the mass in the hammer. Thus the rebound number of a floor would be smaller than that of a soffit, and inclined and vertical surfaces would yield intermediate values.

Concrete under test should be sufficiently massive or should be secured in a heavy testing machine since jerking during the test would reduce the rebound number. A repeated test on or near the same spot will result in a lower reading owing, presumably, to local partial crushing of the concrete. Such results will naturally be unrepresentative of the tested concrete as a whole and should be avoided. The test position also should not be too near the edge as lower readings are likely to result. A minimum of 20mm from the edge should be chosen. BS 1881 : part 202 (1986) and ASTM C805 (1985) describe the method in detail.

In rebound hammer testing, only the concrete in the immediate vicinity of the plunger influences the rebound value. Hence the test is sensitive to local variations in the concrete. If the plunger is located over a hard piece of aggregate, an abnormally high rebound number will result. Conversely, the presence of a void in a similar position would lead to a very low result. To account for these possibilities, ASTM C805 (1985) requires that 10 readings be taken over the area to be tested. If one of the readings differs by more than 7 from the average, that reading should be discarded and a new average should be computed. If more than two readings differ by more than 7 from the average, the entire set of readings is to be discarded.

3.2.1.1 Correlation between Rebound Number and Concrete Strength

Numerous investigations have been undertaken to relate rebound numbers with concrete strength. As indicated earlier, because the test measures a surface hardness condition, it can not be directly related to any other property of the concrete. Hence, no theoretical relationship between rebound number and concrete strength has been established. The relationships which have been obtained are of an empirical nature founded on probability mathematics. Manufacturers of the instrument provide calibration diagrams and recommend their use for estimating strength properties of the concrete. This universal calibration does not however give an accurate assessment of strength and it is generally considered necessary to develop a calibration for each structure tested due to the significant influences of many factors as discussed in the following section.

3.2.1.2 Factors Influencing Test Results

It is recognized that the rebound number is affected by many factors. Within each type of concrete, differences resulting from different moisture content, rate of hardening, orientation of instrument on concrete, surface finishing, moulding materials, as well as degree of carbonation, age and local segregation are very significant and to enable an accurate estimate of the strength their quantative effect must be known. For instance, carbonated concrete can give a higher rebound number yielding an overestimate of strength which in extreme cases can be up to 50% (Bungey, 1989). Similarly, moisture conditions have a greater effect on the test results. The surface hardness of wet concrete is lower than when dry, and the correlation between the rebound number and the concrete strength will be influenced accordingly. The estimated strength on a wet surface will normally be about 20% lower than the equivalent dry surface (Willetts, 1958).

The influence of mix characteristics on rebound number has been shown to be significant, especially cement and aggregate types. Kolek (1970) reported that concretes made of high alumina cements can give strengths 100% higher than a calibration obtained on ordinary portland cement. Also concretes made of super-sulphated cements can give strengths 50% lower than ordinary portland cement calibration would indicate. The influence of aggregate type on rebound number can be considerable so that specific calibration should be determined for every concrete used on site.

3.2.1.3 Applications and Reliability of the Rebound Hammer Test

The applications of the rebound hammer are limited due to the large number of factors which influence the test results as discussed in the previous section. The rebound hammer is the least reliable of the available insitu test methods for assessing the insitu strength of concrete. The accuracy of the test method is not better than ± 15 to 20% for laboratory conditions and a carefully calibrated hammer. In structures the scatter is expected to provide 95% Confidence Limits greater than ± 25% (Malhotra, 1976). The expected accuracy of the test method under a number of conditions has been described by Facaoaru (1984) who indicated that for a case where the cores and concrete composition are available, the accuracy of strength prediction could be as high as ± 18%. However if only auxiliary data are available, the error in strength estimate could be well above 30%, provided that concrete is not older than 1 year. For any other intermediate conditions, the accuracy of strength may be within the above range.

The rebound hammer test is eminently suitable for the determination of uniformity of insitu concrete where it is not necessary to attempt to convert the test results to some other property of the concrete. However, it must be borne in mind that the rebound hammer examines only the near-surface layer of concrete and it does not detect poor internal compaction. Also the user must be aware of the many factors mentioned in section 3.2.1.2 when evaluating the test results.

3.2.2 Ultrasonic Pulse Velocity

The ultrasonic pulse velocity method consists of the propagation of an ultrasonic wave which passes through concrete at velocities ranging from about 3 to 5 km/sec. This method was developed, independently in Canada (Leslie and Cheesman, 1949) and U.K. (Jones, 1949) between 1945 and 1949. The most popular equipment used in the U.K. for this test is known as the PUNDIT (Portable Ultrasonic Non-destructive Digital Indicating Tester). Figure 3.1 shows the PUNDIT set up with two 54 kHz transducers and a cube sample under test.

The application of ultrasonic testing to concrete is generally confined to the use of relatively low frequency pulses. This differs from the technique for testing metals where much higher frequencies can be used giving better directivity and sensitivity of the ultrasonic beam. This is because of the heterogentic nature of concrete where the very high frequency pulses necessary to produce a narrow concentrated beam become severely attenuated and the multiple reflections at the aggregate boundaries cause the beam of vibrations to become scattered. Thus, an upper limit to the frequency that can be used has to be imposed and in practice, transducers having natural frequencies of 20 to 100 kHz are most suitable for testing concrete (Guha and Wedpathak, 1981).

The test operation is relatively simple but requires great care for reliable results to be obtained. One essential is a good acoustic coupling between the transducer face and the concrete which is achieved by applying grease, liquid soap or a similar couplant to the concrete surface.

For practical purposes when testing concrete, the direct transmission method in which transducers are placed on opposite concrete surfaces is most successful. This method is the most satisfactory as the transducers are highly directional and it provides a well defined path length which can be measured accurately.

3.2.2.1 Factors Affecting the Measurement of Pulse Velocity

Measurements of pulse velocity are affected by a number of factors regardless of the properties of concrete.

a) Moisture condition

One of the most important factors is moisture content when the air pores in concrete become impregnated with water the pulse velocity increases, whereas a reduction in pulse velocity would result as the concrete dries out. Bungey (1989) has reported that an increase in moisture condition of concrete from air-dry to saturated condition may increase pulse velocity by up to 5%. Thus, if the effects of moisture are not taken into account, erroneous conclusions may be drawn about insitu strength evaluation as will be discussed later in this chapter.

b) Path length

As concrete is inherently heterogenous, it is essential that path lengths be sufficiently long as to avoid any errors that may be caused by this non-homogeneity. In general, however, the influence of path length will be negligible provided it is not less than 100mm when 20mm aggregates are used or no less than 150mm for aggregate between 20mm and 40mm (BS 1881 : part 203 (1986)).

c) Shape of specimen

Pulse velocity will not be influenced by the shape of the specimen provided its lateral dimension, i.e. its dimension measured at right angles to the pulse path, is not less than the wavelength of the pulse vibrations. Thus for pulses of 50 kHz frequency, this corresponds to a least lateral dimension of about 80mm. Otherwise the pulse velocity may be reduced and the velocity measurements should be used with caution.

d) Presence of reinforcing steel

The presence of reinforcing steel in concrete considerably affects the pulse velocity measurements because pulse velocity in steel is up to about 1.9 times the velocity in plain concrete, thus pulse velocity measurements taken near the steel reinforcing bars may be high and may not represent the true pulse velocity in concrete. This apparent increase in pulse velocity depends upon the proximity of the measurements to the reinforcing bar, the diameter and number of the bars and their orientation with respect to propagation path.

Generally it is advisable to choose pulse paths which avoid the influence of the reinforcement whenever possible in selecting test location. However, when this is not possible, it is necessary to make a correction by applying a correction factor given in BS 1881 : part 203 (1986) to the measured value to give an estimate of the pulse velocity in the plain concrete.

The influence of reinforcing bars is usually insignificant if the reinforcing bars below 20mm diameter run in a direction at right angles to the path length (Bungey, 1984a). If reinforcing bars lie along or parallel to the pulse path, the effect of the reinforcements cannot be avoided and bars of this type as small as 6mm diameter may have a significant effect, particularly in concrete with low pulse velocity (Bungey, 1984a).

e) Effect of stress state of the concrete

In concrete cubes subjected to crushing loads it has been generally accepted that pulse velocity scarcely changes up to 50% of failure load. Similar results have also been shown by Bungey (1980) on a reinforced concrete beam subjected to flexural stress. But at higher stresses a decrease in pulse velocity is observed due to internal breakdown of the concrete and the onset of microcracks. However, under service condition in which stresses would not normally exceed about one third cube strength, the influence of compressive stress on pulse velocity is insignificant. Similarly, tensile stresses have been found to have no significant effect, but cracked zones should be treated with caution.

f) Temperature

Another influencing factor is the effect of temperature. At normal temperatures between 10° to 30°, the significancy of temperature on pulse velocity is negligible, but outside this range corrections to the pulse velocity measurement would be required and these are given in BS 1881 : part 203 (1986).

3.2.2.2 Correlation between Ultrasonic Pulse Velocity and Concrete Strength

The basis of the test method is to provide a measurement of the pulse velocity which depends essentially on the properties of the concrete. It has been shown that the longitudinal wave velocity depends upon the overall elastic properties and is a function of the elastic modulus related to mechanical strength (Long et al, 1945). This correlation, however, is not unique, because concrete is a composite material which consists of two separate constituents, matrix and aggregate, which both have separate elastic and strength properties (Neville, 1988). A number of other factors like mix proportions and moisture content also affect the correlation between pulse velocity and compressive strength thus it is necessary that a specific correlation is made for the particular type of concrete.

Various researchers have attempted to correlate pulse velocity and compressive strength. Notable among these are Bungey (1980), Samarin and Dhir (1984); Zivkovic and Kovacevic (1987). From these investigations it is generally accepted that for a given concrete, pulse velocity and compressive strength are related by an empirical equation of the form

$$f_c = ae^{bv} \tag{3.1}$$

where a and b are empirical constants and f_c and v are compressive strength and pulse velocity respectively.

3.2.2.2.1 Factors Affecting Strength-Pulse Velocity Relationship

a) Effect of changes in water/cement ratio

In fully compacted concrete, an increase in the water/cement ratio produces an increase in the percentage of water-voids which leads to a decrease in both strength and pulse velocity, provided all other factors remain constant. Jones and Gatfield (1955) and Elvery and Ibrahim (1976) reported that the correlation between pulse velocity and compressive strength is independent of the water/cement ratio. However, Zivkovic and Kovacevic (1987) showed the dependency of the correlation on the water/cement ratio such that for the same compressive strength an increase in the water/cement ratio produces a decrease in the pulse velocity. A similar observation has also been found by Kaplan (1959) for high strength concrete (above 30 N/mm^2).

b) Effect of change in aggregate type and maximum size

Aggregates generally occupy about 70% of the volume of concrete and it can therefore be expected that the pulse velocity of the aggregates is an important factor in the pulse velocity of the final concrete. The range in pulse velocity for different rock types was established by Deere and Miller (1966). These values range from 1.5 km/sec to 6 km/sec. These aggregate types are likely to have a less significant effect upon strength properties of concrete hence different relationships between pulse velocity and compressive strength are obtained.

The effect of maximum size of aggregate on the correlation for concretes containing similar aggregate materials is that for a certain pulse velocity the larger aggregate yields a lower compressive strength (Sturrup et al, 1984). Similar results were reported by Facaoaru (1970) as well as Bullock and Whitehurst (1959).

c) Effect of changes in aggregate/cement ratio

Coarse aggregate usually has an elastic modulus greater than that the cement paste matrix in which it is contained. Thus changes in the percentage of coarse aggregate will usually lead to changes in the pulse velocity. On the other hand, these changes are known to have a relatively small effect on the strength of the concrete at a given water/cement ratio. Therefore different relationships between pulse velocity and compressive strength will exist for different aggregate/cement ratios. This has been conclusively shown by Jones and Gatfield (1955), Elvery and Ibrahim (1976), Davis (1977), Chung (1978) and Zivkovic and Kovacevic (1987). They showed that the higher the aggregate/cement ratio, the lower the compressive strength was for a given pulse velocity. However Kaplan (1960) found that in concretes of the same material for a given workability, but for different aggregate/cement ratio and water/cement ratio, a single curve may be drawn between pulse velocity and compressive strength provided that the tests were made at the same age. This apparent independence of aggregate/cement ratio effect has been explained by the latter investigator that an increase of the aggregate/cement ratio would require an increase in the water/cement ratio in order to maintain the same workability. Hence, the effect of these two factors balance one another and a unique correlation would result.

d) Effect of age

When concrete hardens, both pulse velocity and strength increase rapidly during the first few days. At these ages, the pulse velocity is very sensitive to small changes in strength. However, at later ages, larger increases in strength are represented by little increase in the pulse velocity so that the curve tends to flatten out as the age and strength of the concrete increase. It is also noted that a pulse velocity value may represent a low strength at an early age but somewhat higher strength at older age (Facaoaru, 1970). The explanation for this behaviour has been given by the latter researcher due to a progressive drying of concrete (discussed in 3.2.2.1(a)) and on the progressive development of micro-cracking phenomena at the surface between coarse aggregate and mortar. This behaviour of the pulse velocity may lead to different correlations at different ages. Nevertheless, for practical purposes attempts have been made to correlate pulse velocity with compressive strength at different ages for specific concretes (Keiller, 1982) (Swamy and Al-Hamed, 1984).

e) Effect of curing condition

It is a well known fact that the temperature and humidity of curing have an important bearing on the strength of concrete. Similarly the pulse velocity is also dependent to a large extent on the curing regime. As a result, it can be expected that different curing conditions will give different relationships between the pulse velocity and strength during the ageing of concrete. Kaplan (1958) found that the relationship between pulse velocity and compressive strength was not the same for laboratory standard cured and site cured specimens. The site cured specimens had a lower pulse velocity than those cured in the laboratory for an equivalent strength. He also found a different relationship between pulse velocity and compressive strength with cores drilled from columns from the same concrete used in the above tests. Whitehurst (1959) also discovered that variations in curing conditions influenced the relationship between pulse velocity and compressive strength but the effect was considerably greater for flexural strength than for compressive strength.

Tomsett (1980) has proposed an empirical relationship incorporating the concept of a 'dessication' line which may be used to estimate the insitu strength of concrete from the difference in pulse velocity between the insitu concrete and a standard cube and the strength of the standard cube.

f) Effect of compaction and voids in concrete

The presence of voids has been shown to have a significant effect on the strength of concrete. The pulse velocity can also be expected to reduce through the overall reduction in Elastic Modulus and the lower density of the resulting concrete. The amount of reduction in strength and pulse velocity due to presence of voids has been investigated by Kaplan (1960). He concluded that voids due to incomplete compaction have much less effect on pulse velocity than on strength. Thus, 5% voids caused a reduction of approximately 2% in pulse velocity compared with reduction of 30% in compressive strength. It should be emphasized however, that the degree of loss in strength and pulse velocity is dependent to a certain extent on the size and nature of voids, e.g. intentionally entrained air or microscopic bubbles shows smaller effects than indicated above.

3.2.2.3 Applications and Reliability of Pulse Velocity Test

The pulse velocity test method has been found to be useful in assessing the state of concrete structures for strength, quality control, deterioration and other properties. Applications have been described by many investigators (Tomsett, 1980), (Bungey, 1984), (Hillger, 1987).

It has been shown that strength calibration is influenced by many factors and is by no means simple. Facaoaru (1984) has reported that the error in estimated strength could be well above ± 30% if only auxiliary data are available. The accuracy of strength estimates may be increased to within ± 20% if specific calibration is provided for particular mixes and conditions (Bungey, 1980). It must be noted that this assumes access to opposite faces of the member under test.

Repeated measurements of pulse velocity can be made on the same structure at different times. Thus the hardening of concrete can be monitored in the early stages to estimate the strength gain provided the above conditions have been satisfied. This approach may be suitable for early removal of the form-work as the correlation is comparatively better than at later ages (Bungey, 1989a).

Despite the fact that pulse velocity does not correlate well with compressive strength it is perhaps the best of the insitu test methods presently available for evaluating concrete quality. It provides a rapid evaluation of the average condition through the concrete rather than relating to some surface phenomenon. Generally, high pulse velocity readings in concrete are indicative of concrete of good quality.

Pulse velocity can also be used to detect defects in concrete, such as air voids or cracks, which are not water filled. This is because when such defects occur, they are usually associated with a reduction in pulse velocity. This reduction can be explained by the fact that the ultrasonic pulses cannot be transmitted through an air gap or void but have to be diffracted around the periphery of the defect thus increasing the path length and hence decreasing the pulse velocity. The minimum size of detectable void is generally taken to be that causing a 2% change in measured transit time.

3.3 PARTIALLY DESTRUCTIVE METHODS

The partially destructive methods that are available for assessing strength of surface zone concrete are based on the creation of a localized fracture of concrete under very high stress applied to an extremely small area. These test methods do not destroy the structural element, but do destroy the material that is analysed. The failure zone would require to repaired for the purposes of appearence and durability. The partially destructive tests require only one exposed test surface. Also, all have the important characteristic that they directly measure some type of strength property of concrete and strength calibrations are therefore not as sensitive to such a wide range of variables as totally non-destructive methods. Results of research already undertaken in the area of insitu strength assessment are encouraging, although they have some drawbacks in application and accuracy. This group of tests are to be covered by BS 1881 : part 207 (1991) which is likely to be published in 1991.

3.3.1 Windsor Probe Test

The Windsor Probe test was developed in the USA in the 1960's (Malhotra, 1976) for determining penetration resistance and it complies with ASTM C803 (1982). From a fundamental point of view, this method is somewhat similar to the rebound hammer being a form of hardness testing. However it is claimed that the Windsor Probe measures hardness at depths up to 50mm and is thus influenced to a lesser degree by surface moisture, texture and carbonation effects.

The equipment consists of a powder-activated gun or driver, steel probe, calibrated depth gauge and other related accessories which are shown in Figure 3.2. The probe tip is machined to punch through the aggregate as well as the matrix and to remain firmly embedded. Two types of probes are available; silver colored probe for normal weight concrete with 79.5mm overall length and 7.94mm diameter, with the penetrating end diameter reduced to 6.35mm for approximately 14.29mm in length , and gold colored probe with a constant 7.94mm in diameter and 79.5mm in length for lightweight concrete. The probes are threaded at the rear end and contain a plastic guide to locate the probe within the barrel of the driver. A driving head is screwed onto the probe which is inserted into the barrel of the driver. The driver is then loaded with a standardized powder cartridge which develops 790J of energy to the probe irrespective of firing orientation. This relates to a velocity of 183m/sec which does not vary by more than ± 1%. Although there is only one powder load, the driver can be operated at two power levels by an adjustment in the instrument. One is defined as standard power, and the other is low power which is achieved by pushing the probe/driving head assembly 63.5mm downstream into the driver barrel. Low power is used for concrete below about 26 N/mm^2 and standard power for concretes above this strength level. The driver as shown in Figure 3.3 is depressed against a probe locating template held on the concrete surface, followed by firing. Sometimes a three-probe locating template may be used instead of a single-probe locating template. In any case, a minimum of 3 probes has been recommended by the manufacturer to be fired at a test location. After firing, the probe locating template and the driving head are removed and the concrete surface is scraped or brushed to give a level surface. A rectangular base plate is then placed on this surface, and a measuring cap is screwed onto the probe. The exposed heights of the individual probes are measured by means of the calibrated depth gauge to the nearest 0.625mm, as shown in Figure 3.4. The manufacturer also supplies a mechanical averaging device for measuring the average height of the three probes if driven in a triangular pattern. After the test, the probes can be removed using a probe withdrawal kit and a hole remains in the concrete with possibly some spalling which may occur around the hole.

3.3.1.1 Windsor Probe Exposed Length versus Compressive Strength

The test compresses a section of the concrete by actually penetrating the materials. The amount of penetration of the probe into the concrete is the basis of measuring the compressive strength. From experimental investigation it is apparent that a linear relationship exists between exposed probe length and compressive strength. In addition, as indicated above, the probe tip penetrates through aggregate and matrix and penetration is thus affected by the properties of both components. The effect of aggregate properties on Windsor Probe test results is thus likely to be greater than would be expected upon compression tests.

The manufacturer provides a table which relates exposed probe length to compressive strength for a given universal aggregate, Mohs' hardness number and power level (standard or low power). A number of investigators such as Malhotra (1974), Bungey (1981) and Swamy and Al-Hamed (1984a) established calibrations and compared them to those recommended by the manufacturer. It was found that the manufacturer's calibration would have caused a considerable overestimation of the compressive strength. It has also been clarified by the above investigators that different aggregates with the same hardness may have different influences on penetration resistance. Similar observations have also been reported by other investigators (Law and Burt, 1969), (Keiller, 1985).

In addition to aggregate effects, it has been suggested that the calibration is dependent on curing conditions and age (Swamy and Al-Hamed, 1984a) and this further limits use of general calibration. Apart from the above restrictions, a reasonable degree of accuracy of strength estimations of the order of ± 20% may be obtained at 95% confidence limits for a given calibration based on aggregate type (Bungey 1981).

3.3.1.2 Applications of Windsor Probe Test

The method has proved useful as a check on the quality of concrete and its use on a number of different structures was reported by Klotz (1972). Malhotra (1974) has shown that for a given concrete, the exposed probe length increased with increasing age of concrete thus providing an excellent means of determining the relative strength of concrete in a structure. Another report by Carette and Malhotra (1984) showed that the test could be used at the early age which thus provide the facility of checking strength to estimate formwork stripping time. On the other hand, a report by Bungey (1989a) in monitoring concrete strength at early age showed the unreliability of the test at strengths below about $10N/mm^2$. Also reports by Law and Burt (1969) and Klotz (1972) indicated that slender members tend to crack during test and caution is thus required. An important advantage over other partially destructive tests for existing concrete insitu is that no power supply is required, and the method is suitable for use where access is difficult.

3.3.2 Cast-in Pull Out Tests

The concept of pull out testing is based on measuring the force required to pull a steel insert or similar device which was embedded during casting, out of the concrete. In most types of pull out test the insert is pulled out with a lump of concrete, approximately in the shape of a frustum of a cone.

Pull out testing is not a recent development. It has been in use in USSR as early as 1934 (Skramtajew, 1938), but the major development of this approach occurred in the 1960's, resulting in the Danish Lok-Test (Kierkegaard-Hansen, 1975) and a similar method developed by Richards (1972) and Malhotra (1972) in North America in the early 1970's. This test is covered by ASTM C900 (1987) which allows considerable flexibility in the details of the test assembly and loading method while specifying ranges of basic relative dimensions.

The Lok-Test as being the most popular pull out test has been developed at the Danish Technical University and satisfies the requirements of ASTM. The commercially available apparatus has been used in many countries and its popularity has been extended to North America (Bickley, 1982), (Nasser and Al-Manaseer, 1987). The pull out insert system is shown in Figure 3.5. It consists of a steel stem which is attached to a 25mm diameter, 8.5mm thick circular steel disk located at a depth 25mm below the concrete surface. The whole assembly is coated to prevent bonding to the concrete. To avoid rotation of the disk during testing, the disk is produced with a little cut-off as shown in Figure 3.5. The stem is normally screwed to the form before casting the concrete, but it can also be placed in unformed surfaces of concrete using a flotation cup. On the day of testing the stem is removed and a 7.5mm bolt of high tensile steel which is passed through a coupling and a centering plate is screwed into the steel disk. Next, a portable hand operated hydraulic jack is placed on the concrete surface and connected to the coupling on the steel bolt. The jack applies force through a reaction ring of 55mm diameter. Figure 3.6 shows a complete set of Lok-Test equipment.

The Lok-Test force is measured on an oil pressure gauge within the standard load range of 10 to 60kN. The accuracy of load measurement is within ± 2% over normal operating temperatures. A special precision valve system has been provided to ensure that the pull out force can be applied continuously at a speed of 30 ± 10kN/min as long as the turning speed of the jack handle is not less than half a turn of the handle per second.

The loading can be applied up to a required proof load and then released, in which case there is no failure of the concrete around the insert. Alternatively, load can be applied until failure (as indicated by a peak reading) just occurs, in which case little damage occurs to the surface of the concrete, and the cone of concrete fractured by the test does not come out of the mass of concrete. If this procedure is followed, all that shows on the surface of the concrete is a slightly raised ring, the size of the inside of the reaction ring. Finally, if required, loading can be continued past failure until the cone of concrete with an apex angle 'α' equal to 62° is extracted from the concrete. In this case, the small hole made by this procedure may subsequently have to be repaired.

3.3.2.1 Factors Affecting the Reliability of Pull Out

Extensive research has been undertaken at the National Bureau of Standards "NBS" (Stone and Giza, 1985) to study the effect of test geometry (apex angle and embedment depth) and, size and type of aggregate on the reliability of the pull out test. Throughout the investigation, inserts with diameter of 25mm were used. Investigation has been made on concrete with 19mm aggregate size as well as on mortar to study the influence of coarse aggregate in concrete. All tests were conducted at a single value of compressive strength of about $14N/mm^2$. The main findings from this investigation are as follows:

a) Effect of apex angle

Angles (α) from 30 through to 86 degrees with a fixed embedment depth of 25mm have been examined. Typical failure cones have been reported for the concrete and mortar. The failure surface geometry changes from the large apex angles to the smaller angles such that the pull out cones show a conic frustum geometry at the lower apex angles and a trumpet shaped geometry at the higher apex angles.

The effect of the apex angle on the magnitude of pull out force showed that the ultimate pull out force decreases with increasing apex angles. Similar trends were also detected for mortar, but mortar specimens failed at loads significantly below those of companion concrete specimens. The difference in ultimate force between mortar and concrete is presumed to be attributed to the mechanism of aggregate interlock, and becomes less for higher apex angles as might be expected.

For apex angles from 54 through to 86 degrees, there was no significant change in the variability of pull out force. For lower apex angles, variabilities were significantly different between each other and were much higher than for apex angles greater than 54 degrees. For mortar, it was found that the variability is not a function of apex angle.

b) Effect of embedment depth

The effect of embedment depth was based on depths from 12 through to 43mm with a fixed apex angle of 58 degrees. A similar approach to that comparing the effect of the apex angle was followed and it was found that for concrete and mortar, the ultimate pull out force increases with increasing embedment depth. Once again for the reason stated before, the ultimate pull out force in mortar was lower (by 20 to 30%) than in companion concrete.

The variability observed for concrete indicated that there was no significant change in variability of pull out force for embedment depths less than 25mm, possibly because the smaller embedment depth excludes larger size particles from interaction within the failure surface. For embedment depths larger than 25mm, there is a clear trend for increasing variability in ultimate pull out force with increasing embedment depth. For mortar, there was no significant change in variability at any embedment depth.

c) Effect of aggregate size

The effect of aggregate size was based on sizes from 6 through to 19mm with a fixed apex angle of 70 degrees and also a fixed embedment depth of 25mm. Within the range of nominal maximum aggregate sizes, there was no significant difference in mean pull out force. Similarly, there was no significant difference in variability except for the concrete with 19mm aggregate which had a significantly higher variation than other aggregate sizes tested. For an embedment depth of 25mm, it was suggested that variability will increase up to a 25mm maximum nominal aggregate size. Beyond this size, the largest aggregate will be mechanically excluded from the failure surface and the variability may be expected to be reduced. Mortar specimens always failed at loads below those of companion concrete, at about 80% of the pull out force in concrete. Similarly, a lower variation was also detected for mortar than for concrete.

d) Effect of aggregate type

Four different types of aggregate, namely river gravel, crushed limestone, crushed rock and expanded shale (lightweight aggregate) were used with fixed geometry conditions as apex angle of 70 degrees and embedment depth of 25mm. The results indicated no significant difference in mean pull out force for the four types of aggregate which were used. Later NBS experiments by Stone et al (1986) did not however show this similarity as discussed in the following section. For mortar, as expected, the pull out force remained 20% below that of companion concrete specimens.

Variation analysis showed that mortar specimens and those with expanded shale had significantly less variation in pull out force than the other aggregate types tested. In addition, there was no significant difference in variability of pull out force between mortar and expanded shale. Comparison of variability among the concretes made with the natural aggregate, showed significant variations between each other. Crushed rock displayed a larger variability than other types which has been explained by the possibility of damage induced in the aggregate during the crushing process.

3.3.2.2 Correlation between Pull Out Force and Compressive Strength

In an effort to accurately correlate the pull out force to compressive strength, a considerable amount of research has been undertaken all over the world. Experimentally, the pull out force has been found to be linearly proportional to the compressive strength (Kierkegaard and Bickley, 1978), (Bungey, 1983), (Petersen, 1984), (Krenchel and Bickley, 1987). For the Lok-Test, Krenchel and Bickley (1987) report that for concrete cylinder compressive strength from $15 N/mm^2$ to $85 N/mm^2$ the calibration graph is linear with relatively high correlation. Available test data on Lok-Test shows that the correlation is independent of water/cement ratio, type of cement, curing conditions, curing time and air content. The effect of aggregate size on correlations was studied by Krenchel (1970). There appeared to be a significant effect due to maximum aggregate size, but later experiments indicated that differential (Kierkegaard and Bickley, 1978) effect of the maximum aggregate is practically negligible. It is believed that the previously observed effect of aggregate size was due to variations in compressive strength because of variable compaction of concrete. Similar independency from aggregate size has also been reported by Stone and Giza (1985) as discussed in section 3.3.2.1. The significance of aggregate type on correlation was examined by Stone et al (1986) who indicated that at low strength level (below $14N/mm^2$) no significant difference in pull out forces were detected as similarly indicated in previous research by Stone and Giza (1985). However at higher strength levels it has been shown that for given compressive strength, the corresponding pull out force is a function of aggregate type. This significancy has also been stated by Bickley (1982) and Bungey (1983). However, for practical purposes, in most cases the effect of aggregate type has been neglected and a single calibration has been adopted. Danish authorities have recommended a regression formula which correlates the pull out force to the cylinder strength. Equation 3.2 presents this modified to the basis of cube strength using a specimen shape factor of 1.25 as reported by Bungey (1983).

$$L = (5 + 0.64\ f_c)\ \text{kN} \qquad (3.2)$$

Although it should be noted that for weak aggregate, like lightweight aggregate, it is necessary to use a specific calibration (Bungey, 1989).

3.3.2.3 Applications and Reliability of Pull Out Tests

The pull out test appears to be the most reliable partially destructive test, which measures a static strength property of concrete, 25mm under the surface of concrete (in Lok-Test). It is reported (Bungey, 1987) that the accuracy of strength estimation from a calibration may be within ± 20%. The within test variation has been declared to be low and is of the same order as the standard control specimen (Bickley, 1982). The test has been used on site in Europe and North America since the 1970s and the results have indicated no significant deviation from the laboratory findings. The ideal way to use pull out tests on site would be to incorporate pull out assemblies in the formwork for critical structural members and then test during the construction period, including early ages. Bungey (1989a) has confirmed the reliability of pull out tests for determination of form stripping times in cooling towers.

3.3.2.4 New Simple Pull Out Test

This recently proposed technique (Jaegermann, 1989) involves the use of wood-screw which is inserted in fresh concrete. The screw is supported by a light PVC ring to prevent it from sinking. At time of test, the pull out force is applied by means of a proving ring with dial gauge for measuring the applied force. At failure stage, no surface damage was detected except the hole left by extracting the screw.

The test was designed for monitoring the early strength of concrete in the range of 5 to 15 N/mm^2 for form stripping. Whilst data and experience are very limited it is claimed that results are promising and good correlation exists between pull out force and compressive strength.

3.3.3 Drilled-in Pull Out Tests

A number of proposals have been made for tests that can be applied to existing or finished structures where pre-planning is not possible. In UK, the internal fracture test was developed using expanding wedge anchor bolts (Chabowski and Bryden-Smith, 1977) whilst in Denmark, work on the Lok-Test was extended to develop the Capo-Test (Petersen, 1980). In Canada, pull out investigations (Mailhot et al, 1979) involved placing tapered bolts, epoxy grouted bolts, or split-sleeve assemblies in drilled holes. The tapered bolt approach posed some inherent difficulties and gave relatively poor results in terms of reproducibility and was discontinued. An alternative technique known as the Escot-Test (Domone and Castro, 1987) involves internal expansion of a metal sleeve inserted into a drilled hole, but this has not been developed commercially.

3.3.3.1 Internal Fracture Test

a) B.R.E. internal fracture test

Developed originally at the Building Research Establishment (B.R.E.) for testing high alumina cement concrete (Chabowski and Bryden-Smith, 1977) this method has subsequently been applied to portland cement concrete (Chabowski and Bryden-Smith, 1980). The test equipment is shown in Figure 3.7 and consists of a 6mm diameter bolt with an expanding sleeve, an 80mm diameter steel tripod ring acting as a reaction and a torque meter. Early tests also used a load cell, but for simplicity and practical use on site, this is no longer incorporated.

A hole is drilled 30-35mm deep using a roto hammer drill with bits of nominal 6mm diameter in the surface of the concrete. The hole is cleared of dust with an air blower and a 6mm diameter wedge anchor bolt with expanding sleeve is inserted into the hole to the depth of 20mm marked on the shaft as indicated in Figure 3.8. The reaction tripod is assembled over the bolt and a nut and washer screwed on the greased threads of the bolt. The torque meter is then applied and is turned slowly until it reaches the 1N-m reading on the dial. Then the torque meter is rotated a half turn at a time taking approximately 10 seconds to complete each half turn. After each half turn the torque meter is released. The idea of releasing load after each half turn has been proposed to avoid any timing requirement in steady continuous load application. The procedure of applying load is continued until torque meter readings reach a maximum and begins to fall. As the name implies, failure is thought to be initiated by internal cracking. If at this stage no additional load is applied, there will not be any visible damage on the surface of the concrete and the bolt can be sawn off. However, if the load continues, a cone of concrete, which is often intact, will be pulled from the surface.

b) Direct pull-internal fracture test

Whilst the B.R.E. test is simple to use on either horizontal or vertical surfaces it suffers from two apparent disadvantages. The first is that a twisting motion is applied to the bolt resulting from the torque involved in turning the nut, and this may reduce the failure load and increase the scatter of results. The second disadvantage is that the torque meter is relatively insensitive, and determination of the peak load is impeded by the procedure of releasing the load on each half turn of torque meter.

In 1981, Bungey (1981a) presented mechanical direct pull internal fracture equipment which is more sensitive. The latest version of this equipment is shown in Figure 3.9. The reaction ring tripod is identical to that used by the B.R.E. and a mechanical loading system is used together with load measurement by a proving ring. The initial procedure is identical to the B.R.E. method, but the bolt is then connected to the equipment by means of an adaptor nut. Load is applied at a steady rate, without pause, by rotating the loading handle at a rate of one revolution per 20 seconds.

3.3.3.1.1 Factors Affecting the Internal Fracture Force Measurements

As with cast-in pull out tests, the effect of test geometry such as embedment depth of wedge anchor, anchor bolt diameter, and reaction ring diameter may have significant effects on the failure force. Installing the wedge anchor deeper may increase the maximum load capacity of the pull out as shown by Paterson (1976). He also observed that an increase in the size of bolt diameter will increase the pull out force capacity. A similar influence may be created by the effects of a reaction ring of finite diameter. A smaller diameter reaction ring will result in a smaller apex angle, and the pull out force may be expected to increase. If the reaction ring is too small the anchor wedge may break before failure of the concrete. These geometric factors must be standardized, and the B.R.E. configurations have been generally adopted (BS 1881 : Part 207 (1991)).

Measurements on structural members subject to applied loads were examined by Chabowski and Bryden-Smith (1980) and Bungey (1981a). At B.R.E., the former investigators reported the effects of precompression as in column or prestressed member. They observed a large variability of the effects, although there appeared to be a trend for the maximum measured torques to increase with increasing compressive stress. However, for practical purposes the effect can be ignored provided that locations for the test are chosen where compressive stress are low. Bungey (1981a) reported tests on flexural beams with a similar conclusion, although there is an indication of increased test variability. He also specified that tests should not be made where visible cracks pass through the test area.

3.3.3.1.2 Correlation between Test Value and Compressive Strength

The available test data shows that a correlation can be obtained between either the torque value or the pull out force and compressive strength. The correlation has been found to be non-linear and independent of water/cement ratio, curing and cement type (Bungey, 1989). Other research by Swamy and Ali (1984) however suggested the dependency of correlation on curing using the direct pull method. It should be pointed out that the correlations obtained by the latter investigators for wet and dry curing were established in such a way that for both cases the direct pull internal fracture tests were carried out on a dry cured slab, whereas, the compression tests were carried out on cubes under wet or dry curing and this might explain the apparent dependency of correlation on curing condition. The type and size of aggregate have some effect on the torque value or pull out force, but the effect is relatively small (Chabowski and Bryden-Smith, 1980), (Bungey, 1981a).

The variability between individual tests has been reported to be high (Chabowski and Bryden-Smith, 1980), (Swamy and Ali, 1984), (Keiller, 1985), (Bungey, 1987) since only a small volume of concrete adjacent to the surface is stressed. For a reasonable estimate of cube strength, a minimum number of six tests is recommended. 95% confidence limits on estimated strength of ± 28% based on the mean of six test results were claimed for B.R.E. internal fracture test (Chabowski and Bryden-Smith, 1980), provided that any individual result causing a coefficient of variation of greater than 16% are discarded. Bungey (1989) has claimed the accuracy of strength estimation may improve by using the direct pull approach and has reported ± 20% at 95% confidence limit. Whatever loading method approach is adopted, the load application procedure will have a major influence on the calibration against compressive strength and thus any calibration must be specifically prepared for the procedure in use, and this must be carefully standardized.

3.3.3.2 Capo-Test

This was developed as a direct equivalent to the Lok-Test

(Petersen, 1980) with the name based on the expression "Cut and Pull out". The test procedure is that a hole 18mm diameter and about 45mm deep is drilled in the concrete perpendicularly to the surface. Afterwards a groove is cut out in the concrete with a Capo diamond miller (Figure 3.10). This groove has a diameter of 25mm and is 10mm high. It is formed 25mm below the concrete surface (Figure 3.11a). Water is used throughout the drilling and underreaming as a lubricant.

A special compressed expanding steel ring with an outside diameter of 18mm is then placed on the bolt of an expansion unit and the unit placed into the hole down to the level of the groove. By turning the nuts on the unit with two wrenches the steel ring is expanded until it reaches the inside diameter of the groove (Figure 3.11b). Then, the Lok-Test jack is connected to the unit and load applied as with the Lok-Test (Figure 3.11c). The Capo force is recorded as the maximum reading during pull out, which in this case is always continued past failure until the cone of concrete is removed and hence the cone hole may have to be repaired afterwards. This allows recovery of the expansion unit, and the expandable ring may be reused two or three times until cracking is showing up. The test requires considerable skill but it is claimed that operation takes only approximately 10 minutes.

Several investigations have been conducted in order to compare Capo force to Lok force. It has been concluded that the Capo force correlates with compressive strength in a similar manner to the Lok-Test, and that the accuracy of the Lok-Test and

Capo-Test are of the same order, thus identical calibrations could be used (Bellander, 1983), (Petersen, 1984).

3.3.4 Pull Off Test

Pull off testing as a means of predicting the strength of concrete was begun in the mid 1970's (Long, 1979). The development of a pull off test was undertaken at Queens' University; Belfast and the reliability assessed (Long and Murray, 1981) and it is now commercially available.

The method involves bonding a circular aluminium or steel disk (usually 50mm diameter) to the surface of concrete by means of a high strength epoxy resin adhesive (Figure 3.12). Before to this operation, the surface of the concrete is abraded using emery paper to provide a smooth surface and then degreased using a suitable degreasing agent. Bonding difficulties may be encountered on damp surfaces. After setting and hardening of the glue, a specially designed apparatus called "Limpet" is used to apply tensile load through a steel bar which is screwed into the centre of the disk (Figure 3.13). The "Limpet" rests on the concrete surface and applies force through a counter pressure ring. A slowly increasing tensile force is applied at a rate of one revolution of the handle every five seconds. Loading is continued until the concrete fails in tension just below the surface. A nominal pull off strength for the concrete specimen is then calculated as further details on the calculation will be given later in Section 6.3.4.

For old portland cement concrete which has been carbonated, or for high alumina cement concrete to avoid errors caused by hard surface shell effects, partial coring around the position of the circular disk (Figure 3.12b) was recommended (Long and Murray, 1981). However, partial coring will affect the variability of results adversely due to random weakening of both aggregate and aggregate-mortar bonds and lead also to a lower pull off strength compared to surface tests.

3.3.4.1 Correlation between Pull Off and Compressive Strengths

The results of a wide range of tests have indicated that a good correlation exists between pull off and compressive strengths (Long and Murray, 1984), (Murray, 1984), (Keiller, 1985). The accuracy and reliability of pull off tests were examined by Glass (1981) who indicated that the 95% confidence limit for a strength level of $30N/mm^2$ was found to be ± 15%. Murray (1984), Keiller (1985) and others have examined the effect of various factors on the reliability of correlation curves. It was found that factors such as the age of concrete, aggregate type and size, curing conditions and the influence of compressive stress had some effect as discussed in the following section. However it was mentioned that the influences were marginal and a single calibration curve can be used with reasonable confidence for natural aggregate concrete.

a) Effect of age

A very limited number of investigations to study the effect of age on the correlation have been reported. Murray (1984) produced calibration graphs for specimens at 7 and 28 days. From the results of this work, it was shown that the ratios of compressive strength to pull off strength for 7 days differed by less than 3% from those of the corresponding 28 day graphs. It was thus considered appropriate that the 7 and 28 day results be combined. Also Dini (1980) tried to study the effect of age on the correlation at 7, 28 and 91 days. From the results of his work, it was clear that although separate curves did exist they were fairly close together. It was concluded that, for most practical purposes, an average graph would be quite acceptable.

b) Effect of aggregate type and size

It is known that the type and the maximum size of coarse aggregate used in concrete can have an influence on its strength. The aggregate stength itself is of less importance, since natural aggregates are all generally much stronger than that cement paste, except in a very high strength concrete. However, the aggregate texture which depends on whether the aggregates are smooth or rough and angular, affects both the bond and the stress level at which microcracking begins. Erntroy and Shacklock (1954) showed that the relation between compressive strength and splitting tensile strength depends on the type of coarse aggregate used since (except in high strength concrete) the surface texture affects the compressive strength very much less than the strength in tension. Keiller (1985) has reported tests using pull off tests which demonstrate a similar conclusion. He assessed the effect of aggregate type on the correlation between pull off and compressive strengths using gravel and granite aggregates. He stated that the gravel aggregate gave a lower pull off strength for a given compressive strength. Also he showed that the results for granite aggregate concrete gave a better correlation with compressive strength than results for gravel aggregate concrete.

The use of a larger maximum size of aggregate affects the strength in several ways. Because the larger particles reduce the specific surface area of the aggregate, the total bond strength is also less, and this tends to reduce the strength. Also larger aggregate particles provide more restraint on volume changes in the paste, which thus may induce additional stresses in the paste, which tend to weaken the concrete. Murray (1984) has attempted to establish relations between pull off and compressive strengths for two types of concrete with maximum aggregate sizes of 10mm and 20mm. It was observed that the pull off strength/ compressive strength ratio for 10mm aggregate concrete was 7% greater than the corresponding ratio for 20mm aggregate concrete coupled with a lower variation in test results for 10mm aggregate concrete.

c) Effect of curing condition

Murray (1984) carried out pull off tests on both wet and dry cured cubes at ages of 7 days and 28 days. The results showed that the ratio of pull off strength/compressive strength is lower for dry conditions which is similar to that reported by Neville (1988) in that, compared to wet curing, air curing reduces the splitting tensile strength more than it does the compressive strength.

d) Effect of compressive stress

It is known that when compressive stress is applied to a concrete specimen, its tensile strength in the orthogonal direction will be reduced. Experimental work was performed by Cochrane (1978) to study the effect of compressive stress on the pull off strength. On this investigation, concrete specimens were stressed to 0.25 and 0.5 times their compressive strength. It was found that a compressive stress of 0.25 times the compressive strength reduced the pull off strength by 12% while a stress of 0.5 times the compressive strength reduced it by 24%.

3.3.4.2 Applications of Pull Off Test

The principal applications of pull off tests, like other partially destructive tests, are insitu strength assessment and to control the quality of concrete members. It has been claimed (Long and Murray, 1984) that the pull off test is particularly suitable for testing high alumina cement concrete or carbonated portland cement concrete using the partial core technique as indicated previously. However, the use of partially cored pull off tests leaves a damage zone which is larger than for surface tests which must be made good. Also, prior to applying load in either case (surface or partial cored pull off test), a sufficient time is required for adequate curing of the adhesive to reach a bond strength in excess of the tensile strength of the concrete. Any tests giving a partial or complete bond failure must be discounted. The method is also being standardized in BS 1881 : Part 207 (1991).

3.4 DESTRUCTIVE METHODS

These methods involve the testing to failure of either a sample of concrete removed from a structural unit or the complete structure. The fact that a strength parameter is actually measured, is an obvious advantage of such tests. Although the results obtained are generally more reliable than those from non-destructive or partially destructive methods, their usefulness is limited by the relatively small number of tests which can be performed on a single structure. This is largely due to the resulting damage and weakening of the members when portions are removed but the cost of the tests is another limiting factor.

Destructive tests are, therefore, normally only used when non-destructive or partially destructive methods are considered insufficient to estimate the strength, or yield inconclusive results. This section includes details of core testing which is commonly used to estimate the insitu strength of concrete, and the only destructive method considered in this programme of work.

3.4.1 Core Test

The testing of cores, taken from a structural unit is the most direct method for determining the insitu strength of concrete. The methods of testing cores in compression are fully described in BS 1881 : part 120 (1983) and ASTM C42 (1984). Usually, a core is cut by means of a rotary cutting tool with diamond bits and should be operated by skilled operators. Each core must be cut and trimmed by means of masonry or diamond saw to the required length followed by end preparation to provide plane end surfaces. One way to achieve planeness is by grinding the ends; this is satisfactory but is expensive and time-consuming. The most common way of achieving this planeness requirement is to cap the ends of the cores with a suitable material. Suitable materials are high alumina cement mortar and sulphur-sand mixture as recommended by BS 1881 : part 120 (1983). After end surface preparation, BS 1881 : part 120 (1983) or ASTM C42 (1984) requires that the core shall be soaked in water for about 2 days prior to test. Some investigators such as Petersons (1971) and Yip and Tam (1988) do not agree with this opinion and require that the core at the time of test should be dry or have a moisture condition similar to that of the parent concrete in the structure. After the 2 day pre-test soaking, dry cured concrete will register a lower strength than if it was tested in its insitu moisture state. A value of about 16% strength reduction due to soaking dry cured concrete has been reported (Bloem, 1968), (Sangha and Dhir, 1976). Testing dry cores is also specified in German, Swiss and Australian Standards and ACI 318-83 (1983).

Cores of 150mm or 100mm diameter are recommended by most national codes and specifications. Though these sizes of core will ensure more consistent and reliable results, not infrequently it is impracticable to obtain such cores. This is due to either small size of the member or to congestion of reinforcement. Consequently, smaller cores may have to be resorted to. There are also other advantages in taking small diameter cores. A smaller coring machine is required and there is less damage to the structure. It may be possible to take a larger number of small cores which give a better overall evaluation of a structure than a small number of larger cores. For example, if twice as many 50mm cores are cut from the structure as 100mm diameter cores only a quarter of the volume of concrete will need to be removed from the structure provided the same length/diameter ratio is adopted. Cores of 50mm (or 44mm) and also 75mm diameter have been investigated (Bungey, 1979), (Munday and Dhir, 1984), (Yip and Tam, 1988) and Addendum to "CSTR11" (1976) permits the use of such cores. BS 1881 : part 120 (1983) specifies that the core diameter should be at least three times the nominal maximum size of the coarse aggregate in the concrete. However, it tends to be over cautious in additionally requiring the core diameter to be 100mm or 150mm and moreover stating 150mm as the preferred diameter size, even though BS 6089 (1981) does make provision in exceptional cases for core diameter less than 100mm. In some other countries like Australia a 75mm diameter core is considered acceptable and cores as small as 50mm are permitted in Swiss and German standards.

3.4.1.1 Factors Affecting Measured Compressive Strength of Drilled Cores

Core testing in compression, as with other standard specimen testing, is simple and straightforward. However, the procedure used has to be carefully established and well understood as numerous factors can effect the measured value and hence the judgement on the quality of concrete. Some important factors are outlined below.

a) Diameter of core

The available data on strength measurements of cores or cast specimens having different diameters are somewhat contradictory. For cast specimens, it is generally known that the strength tends to increase with decreasing size of specimen (Neville, 1988). Similarly, Malhotra (1976a) reported higher strength for 102 × 203mm cylinders than those of 152 × 305mm cylinder, except that at low strength, the indications were reversed. On the other hand, Nasser and Kenyon (1984) reported that on average a lower strength resulted for 75 × 150mm cylinders compared to 150 × 300mm cylinders for a strength range up to $35N/mm^2$. Further contradictory data has been seen for core specimens. For example, Keiller (1984) showed that 50mm cores are stronger than 100mm cores as also reported by Lewandowski (1971). Other researchers such as Meininger (1968) and Munday and Dhir (1984) observed that core diameter did not effect the average strength level, whilst Campbell and Tobin (1967) reported higher strength for 150mm cores than for 100mm cores. Recent research by Yip and Tam (1988) showed a similar observation that 100mm cores are stronger than 50mm cores.

In spite of these disagreements however, variation analyses by all these investigators have shown that the variability of test results on cores or moulded specimens generally increases with decreasing core or moulded specimen size.

b) Length/diameter ratio

For a given diameter core, it is known that the measured strength of a core increases as its length/diameter (ℓ/d) ratio decreases. The influence of ℓ/d ratio on core strength is due to the effect of shape of the specimen on the stress distributions whilst under test. Standard cylinder specimens are of length equal to twice the diameter, but for cores the length of specimen depends on the thickness of the structural member. In general the ℓ/d ratio in core testing is limited to between 1 and 2, and the measured strength is expressed as the equivalent strength value for a specified ℓ/d which is normally equal to 2. CSTR11 (1976) provides a formula to calculate the equivalent core strength at $\ell/d = 2$.

$$f_{\lambda=2} = \frac{2\ f_{\lambda}}{1.5 + 1/\lambda} \tag{3.3}$$

$$\lambda = \ell/d$$

This equation also provides the basis for the current BS 1881 : part 120 (1983). ASTM C42 (1984) and several investigators (Kesler, 1959), (Sangha and Dhir, 1972), (Bungey, 1979), (Yip and Tam, 1988) also specified correction factors to express the measured strength as equivalent strength value for $\ell/d = 2$. Correction factors for small cores reported by Bungey (1979) showed that the effect of ℓ/d ratio is considerably higher than that indicated by ASTM C42 (1984), but it is only marginally higher than BS 1881 : part 120 (1983). On the other hand the results obtained by Yip and Tam (1988) did not indicate any significancy of small cores on ℓ/d ratio.

c) Direction of drilling

Normally cores are drilled either parallel to the direction of casting (vertical drilling) or perpendicular to direction of casting (horizontal drilling). The choice of coring direction depends on the feasibility of access on structural members. A number of investigators have examined the influence of direction of drilling on core strength and results are again contradictory. Petersons (1964), Meininger (1968), Munday and Dhir (1984) and Yip and Tam (1988) have reported that cores drilled horizontally developed lower strength than similarly located cores drilled vertically. CSTR11 (1976) recommends that the core strength taken in the horizontal direction to be 8% less than that of vertical cores and this figure has been adopted by current BS 1881 : part 120 (1983). However, other researchers like Bloem (1965), Meynink and Samarin (1979) and Keiller (1984) found no difference in the strength of cores whether these were drilled vertically or horizontally.

d) Reinforcement

The existence of reinforcement in cores may result in a reduction in core strength of up to 10% (Bungey, 1989). Therefore drilling through reinforcement should be avoided whenever possible, otherwise the core strength measurements should be corrected by factors given in BS 1881 : part 120 (1983). The Addendum to CSTR11 (1976) further specifies that cores less than 100mm diameter containing reinforcement should be rejected.

3.4.1.2 Estimation of Cube Strength and its Reliability

The estimation of cube strength may be obtained from core strength measurements by applying appropriate factors. The estimation of insitu cube strength involves an allowance for the basic differences in shape between the core and a cube, and direction of drilling. CSTR11 (1976) recommends the following formula:

$$\text{Estimated insitu cube strength} = \frac{K}{1.5 + 1/\lambda} \times \text{measured core strength} \qquad (3.4)$$

where λ = length/diameter ratio

K = 2.5 for cores drilled horizontally

or K = 2.3 for cores drilled vertically.

This formula also has been adopted by BS 1881 : part 120 (1983) and BS 6089 (1981). It is generally accepted that the insitu strength estimated from a single 100 or 150mm core can be considered to lie (with 95% confidence) within ± 12% of the true strength of the concrete at that location, and if n cores are taken the accuracy will be increased to ± 12%/$\sqrt{n}$. For small diameter cores, Bungey (1979) has suggested that the accuracy of estimated insitu cube strength may have 95% confidence limits as high as ± 36%/$\sqrt{n}$.

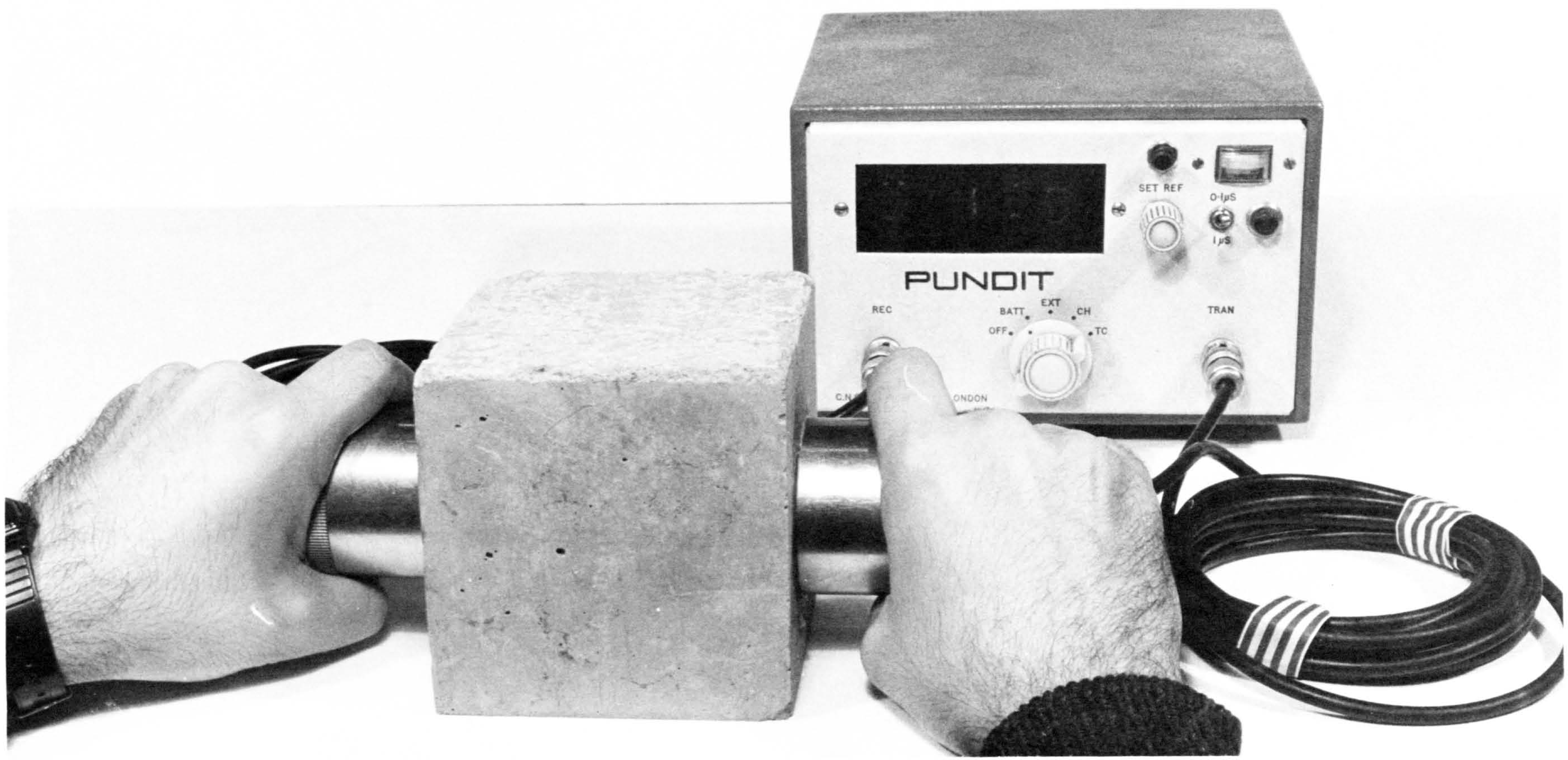
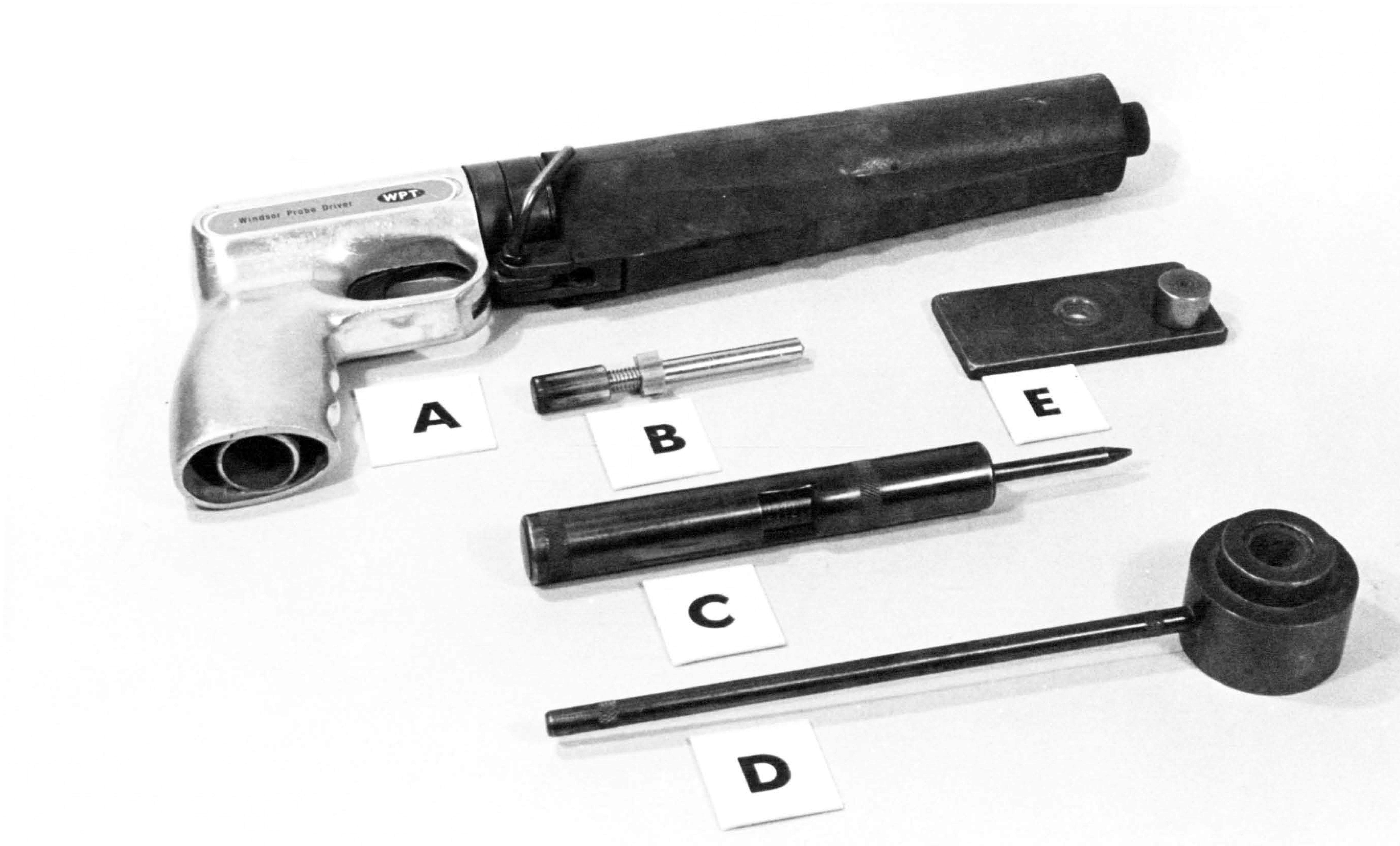


Figure 3.1: Apparatus for pulse velocity test



A: driver unit; B: probe for lightweight concrete; C: calibrated depth gauge; D: single probe template;
E: base gauge plate with probe measuring cap

Figure 3.2: Apparatus for Windsor Probe test

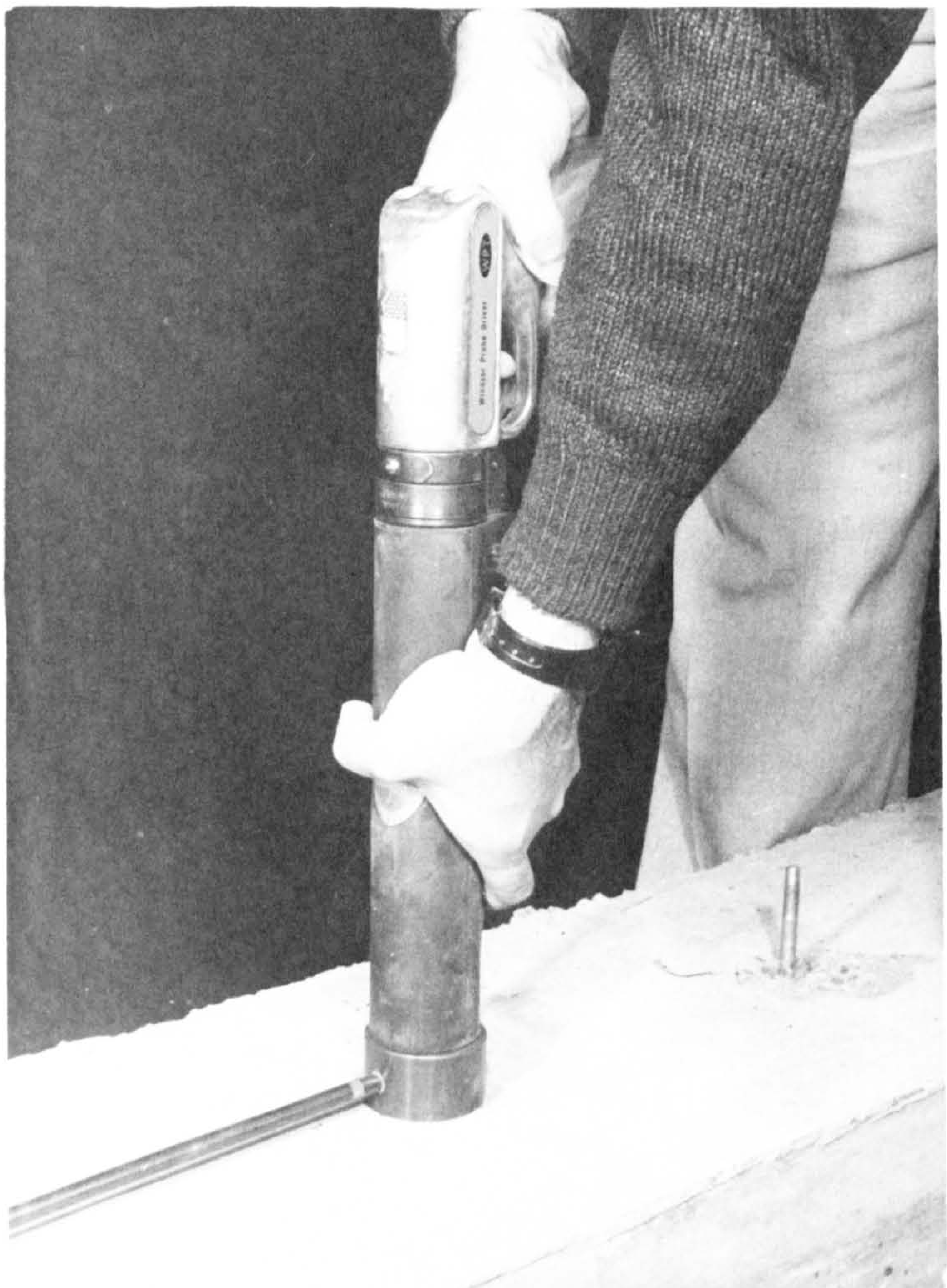


Figure 3.3: Driver in use

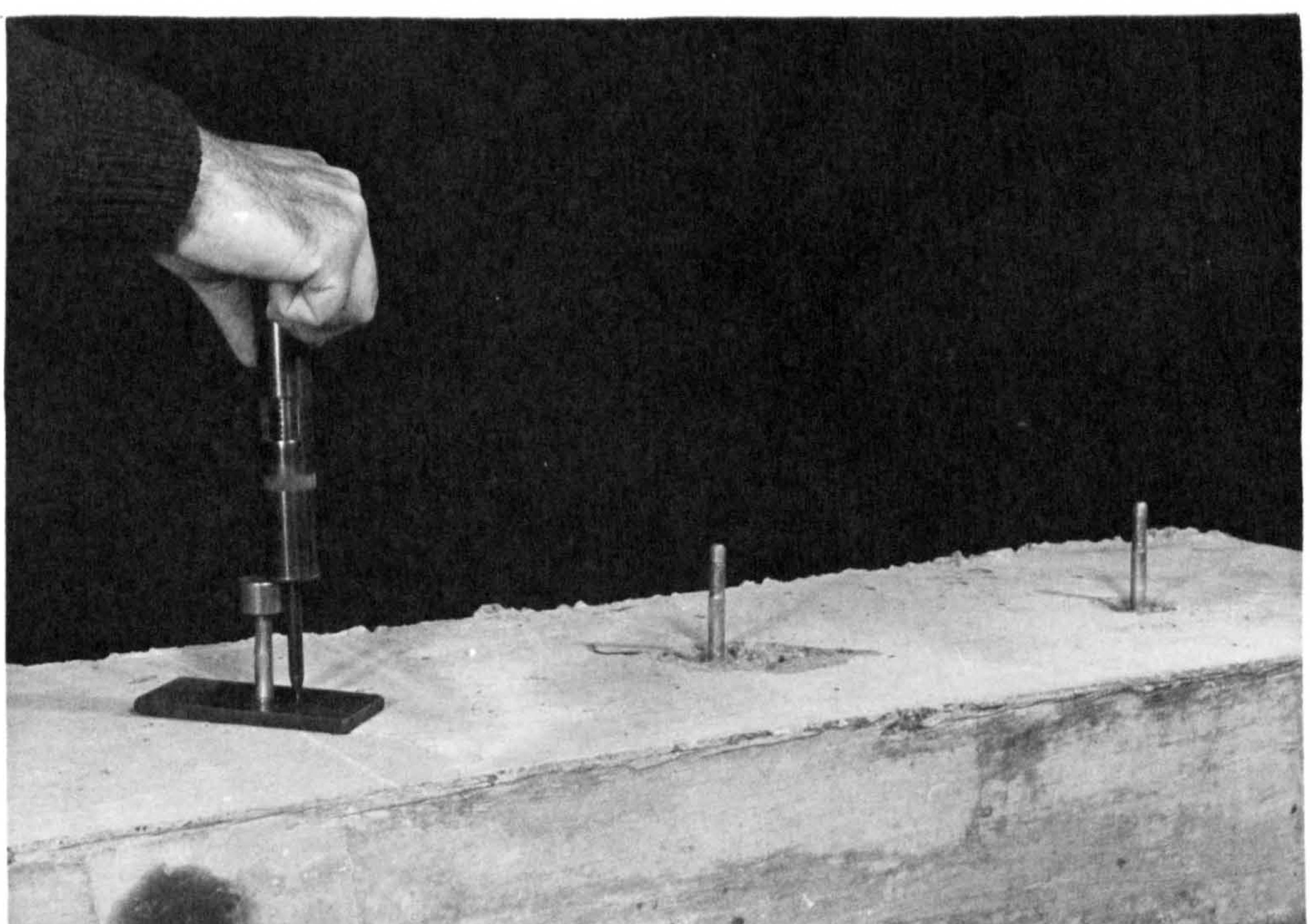
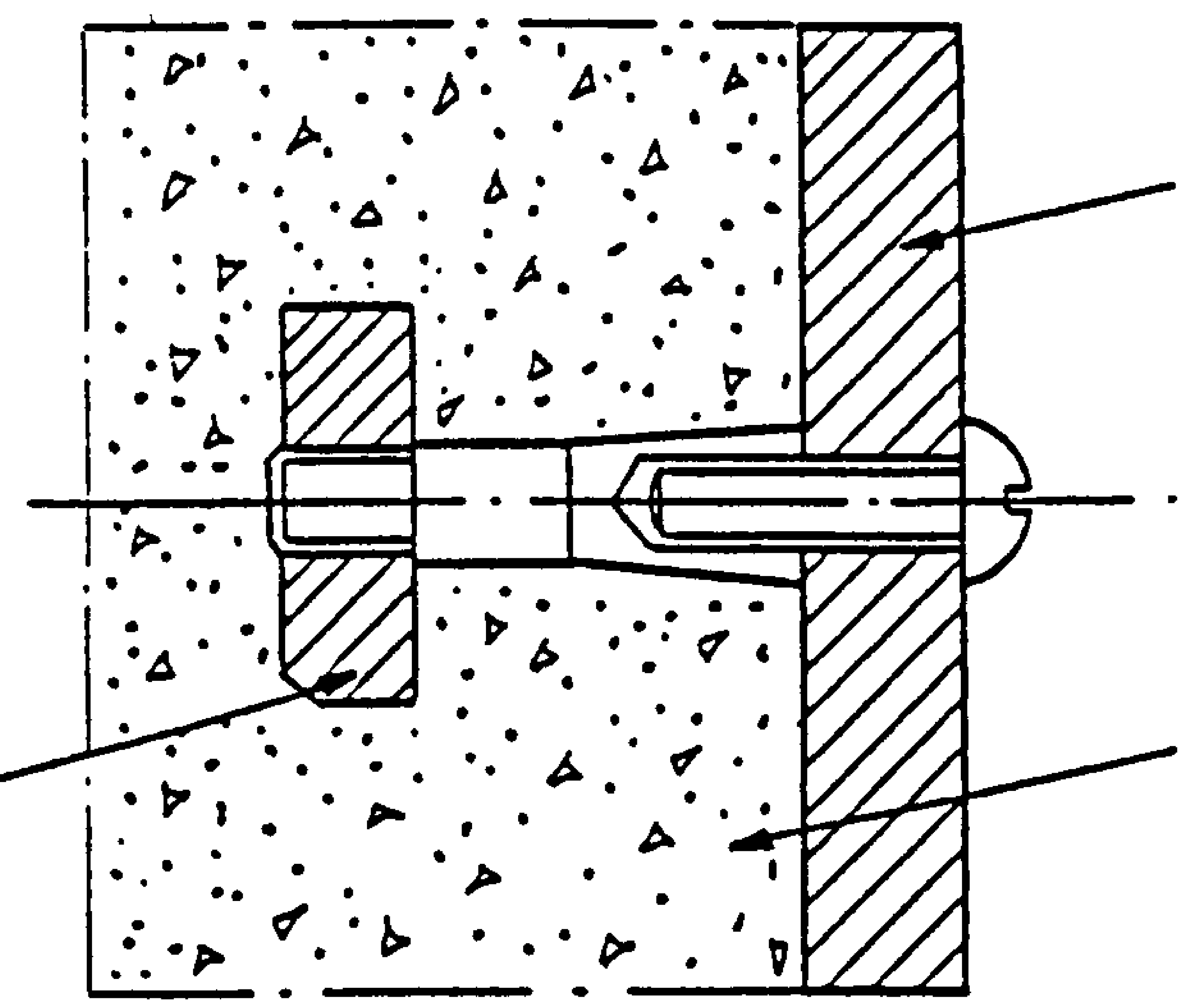
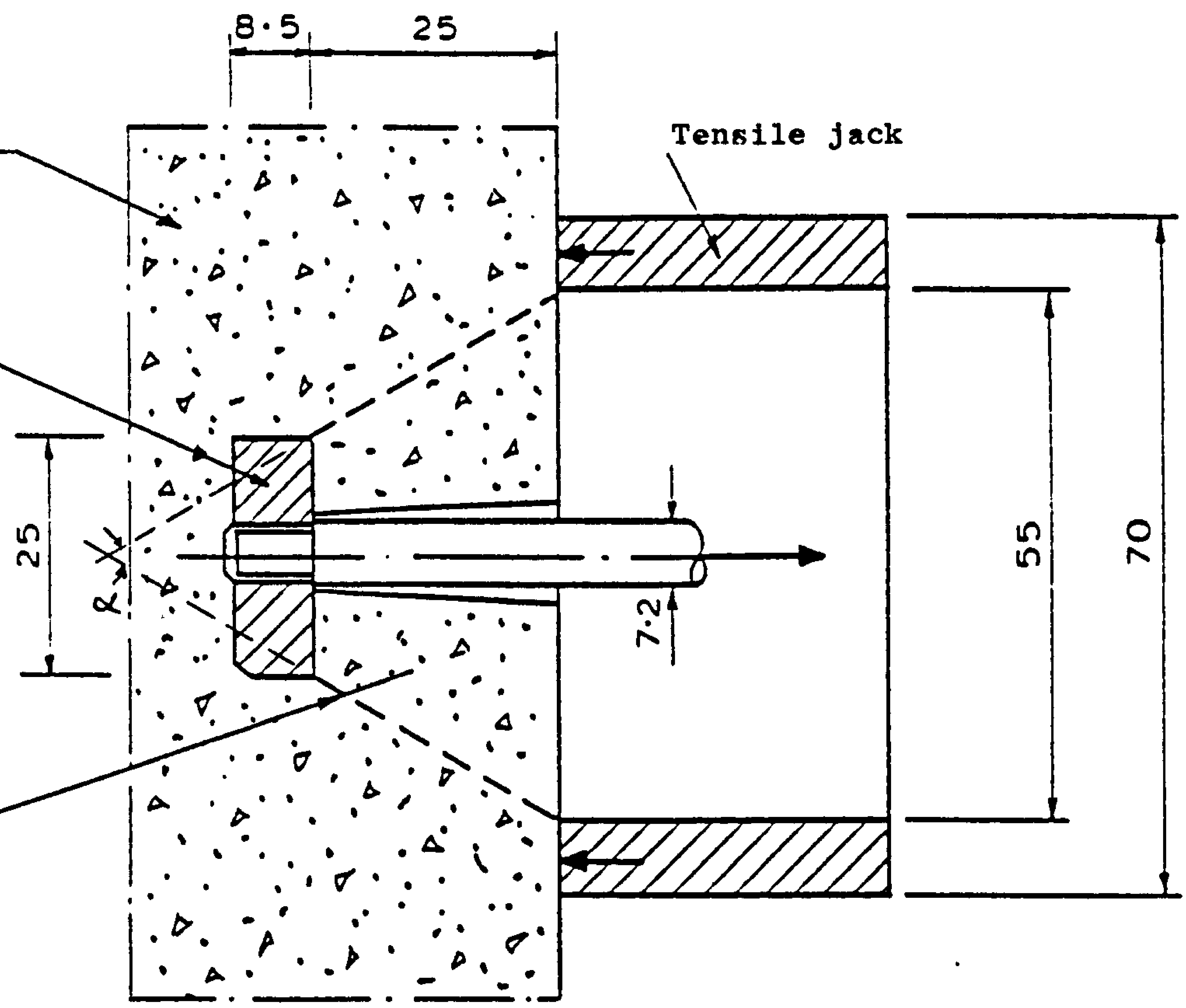


Figure 3.4: Exposed probe length measurement



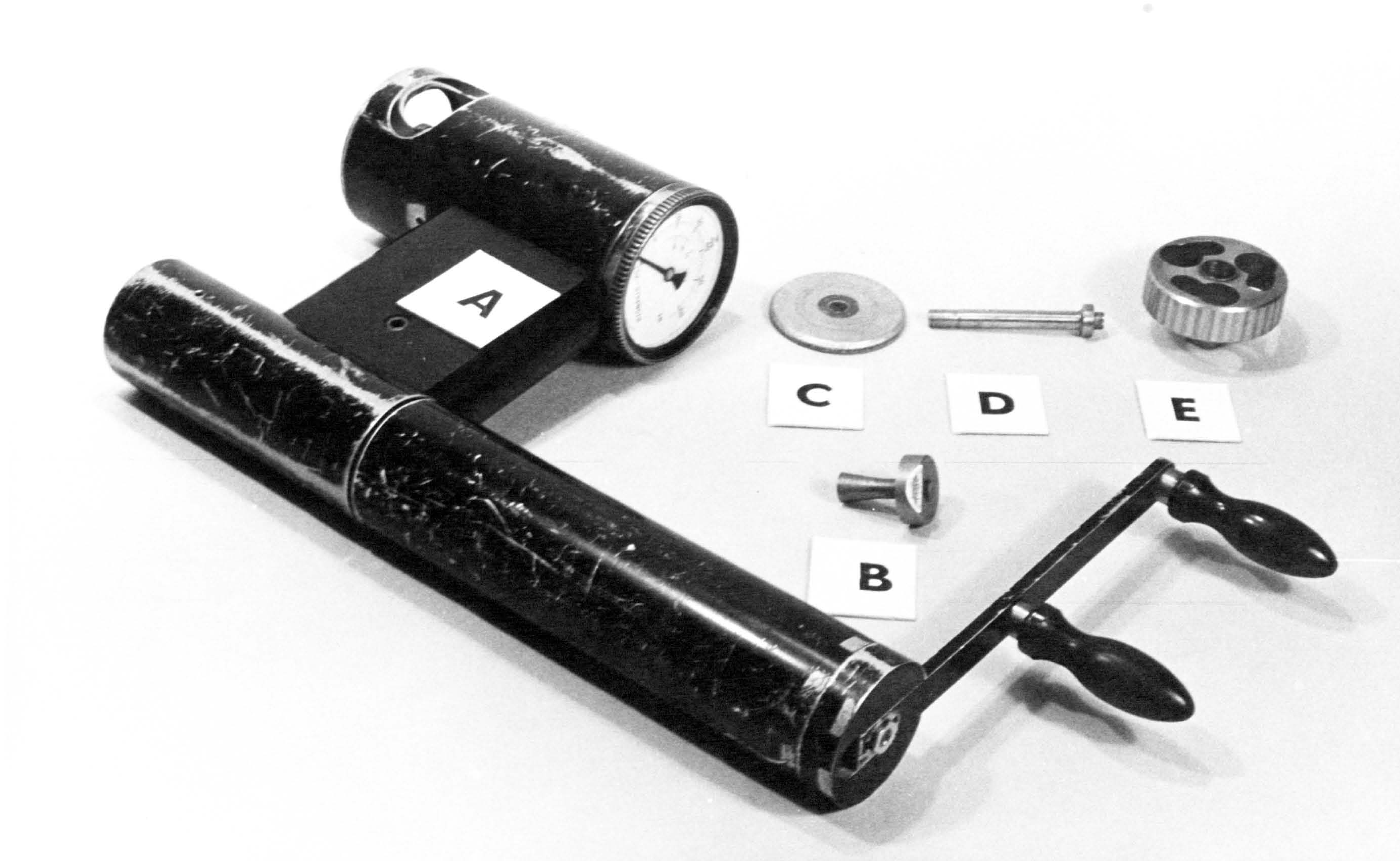
a) Pull out disk



b)The tensile jack connected to the pull out disk

Figure 3.5: Test set-up for Lok-Test (dimensions in mm)

SCALE 1:1



A: Lok-Test instrument; B: Lok insert; C: centering plate; D: pull bolt; E: coupling

Figure 3.6: Apparatus for Lok-Test

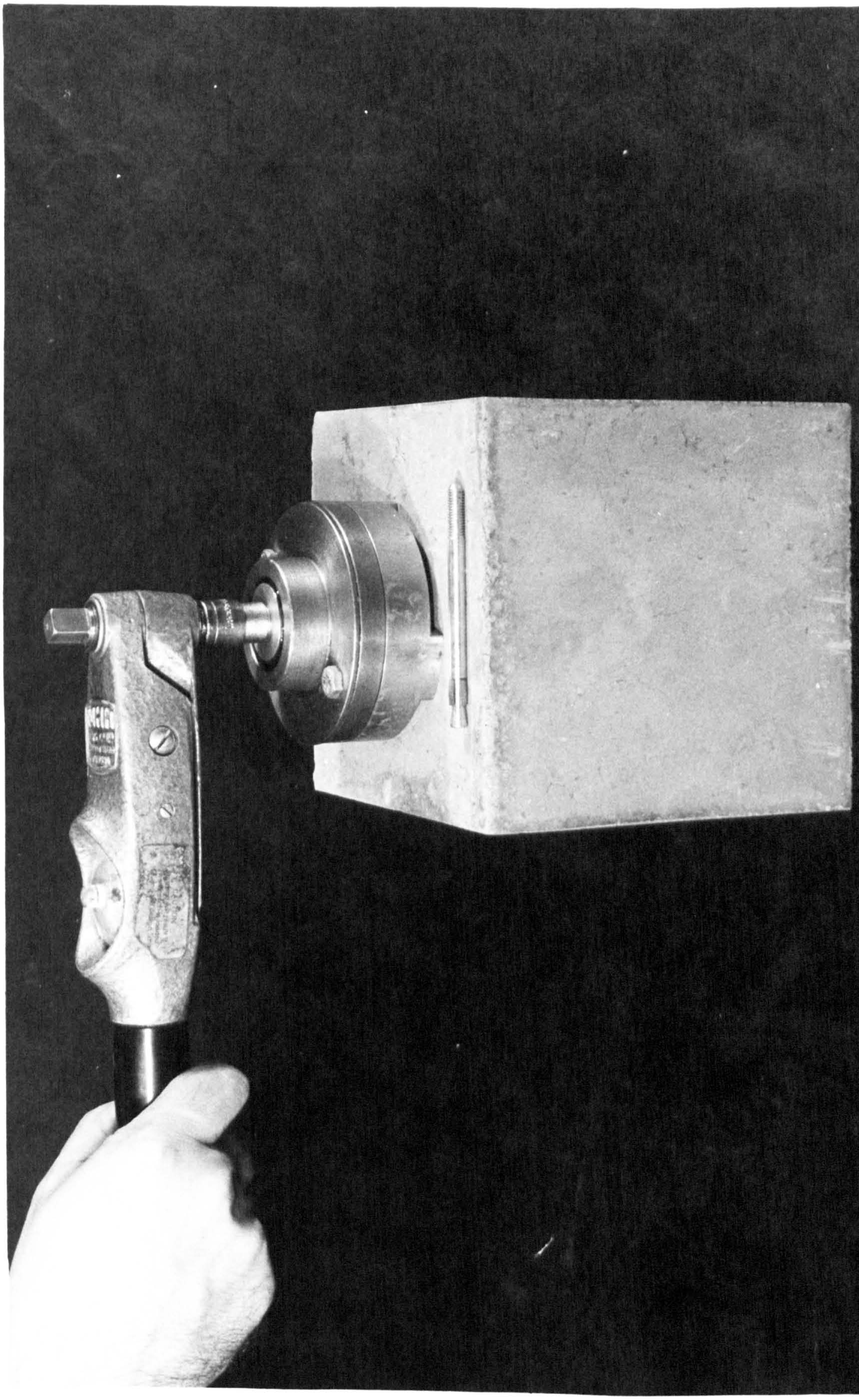


Figure 3.7: Apparatus for B.R.E. internal fracture test

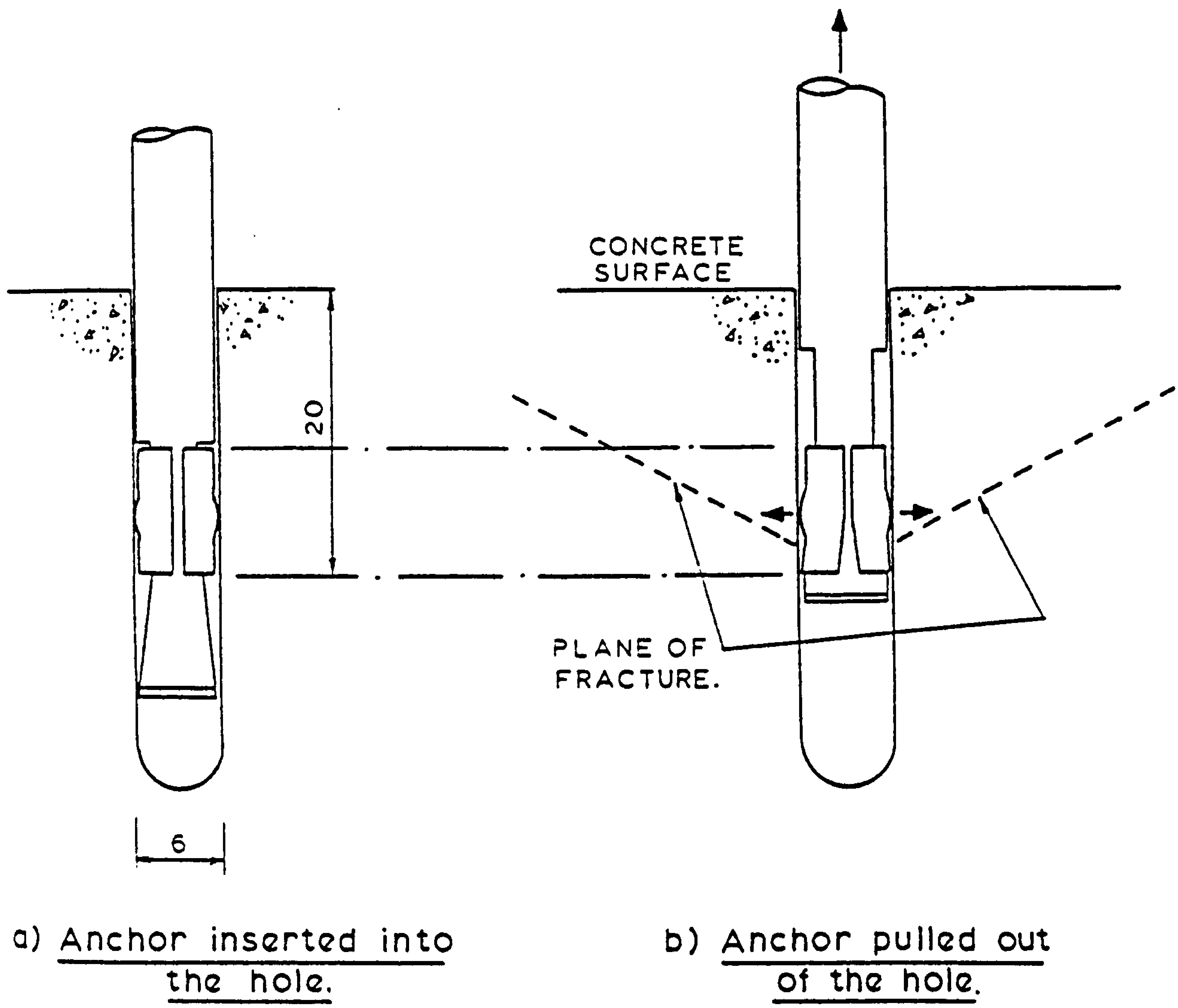


Figure 3.8: Internal fracture test configuration

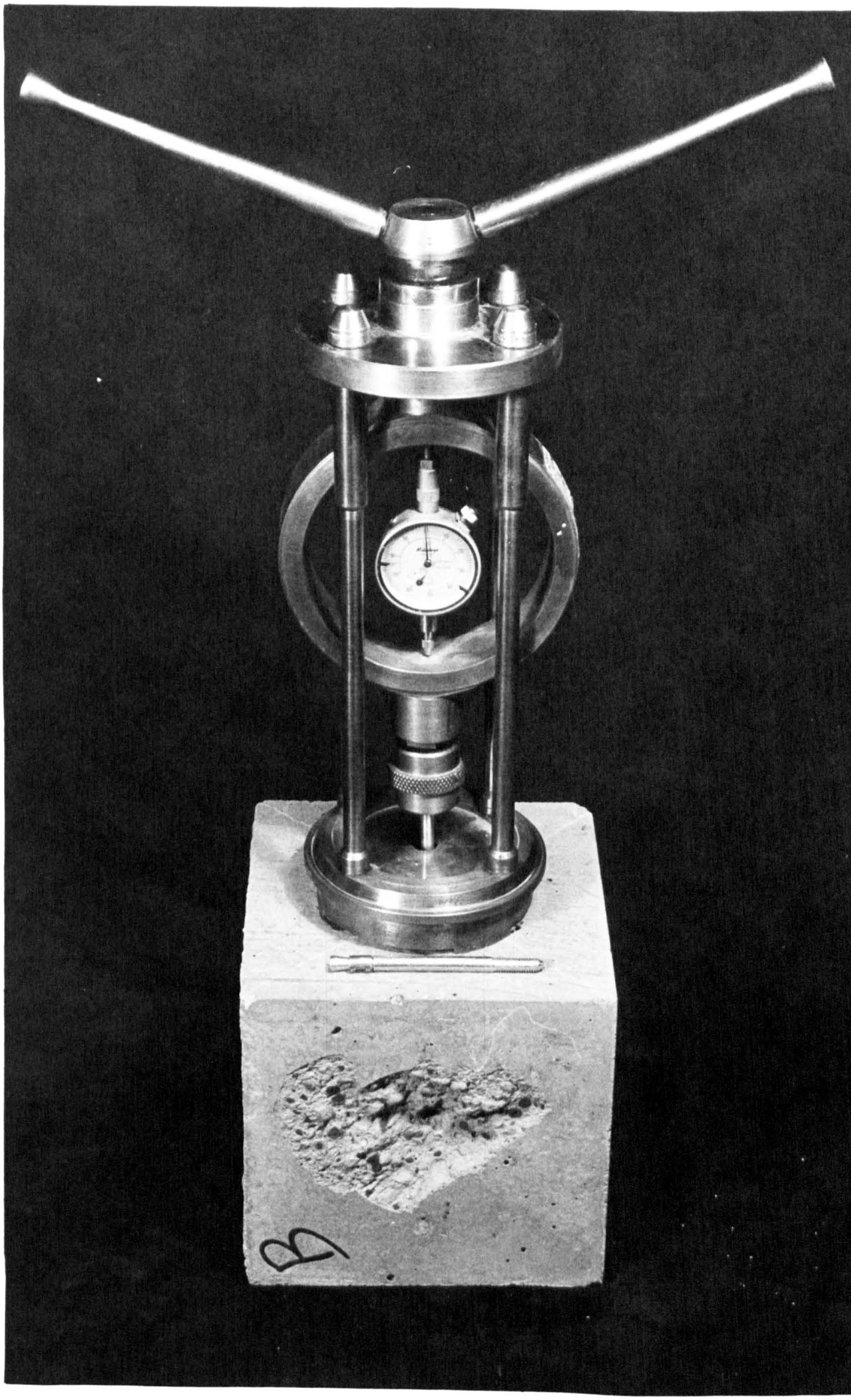
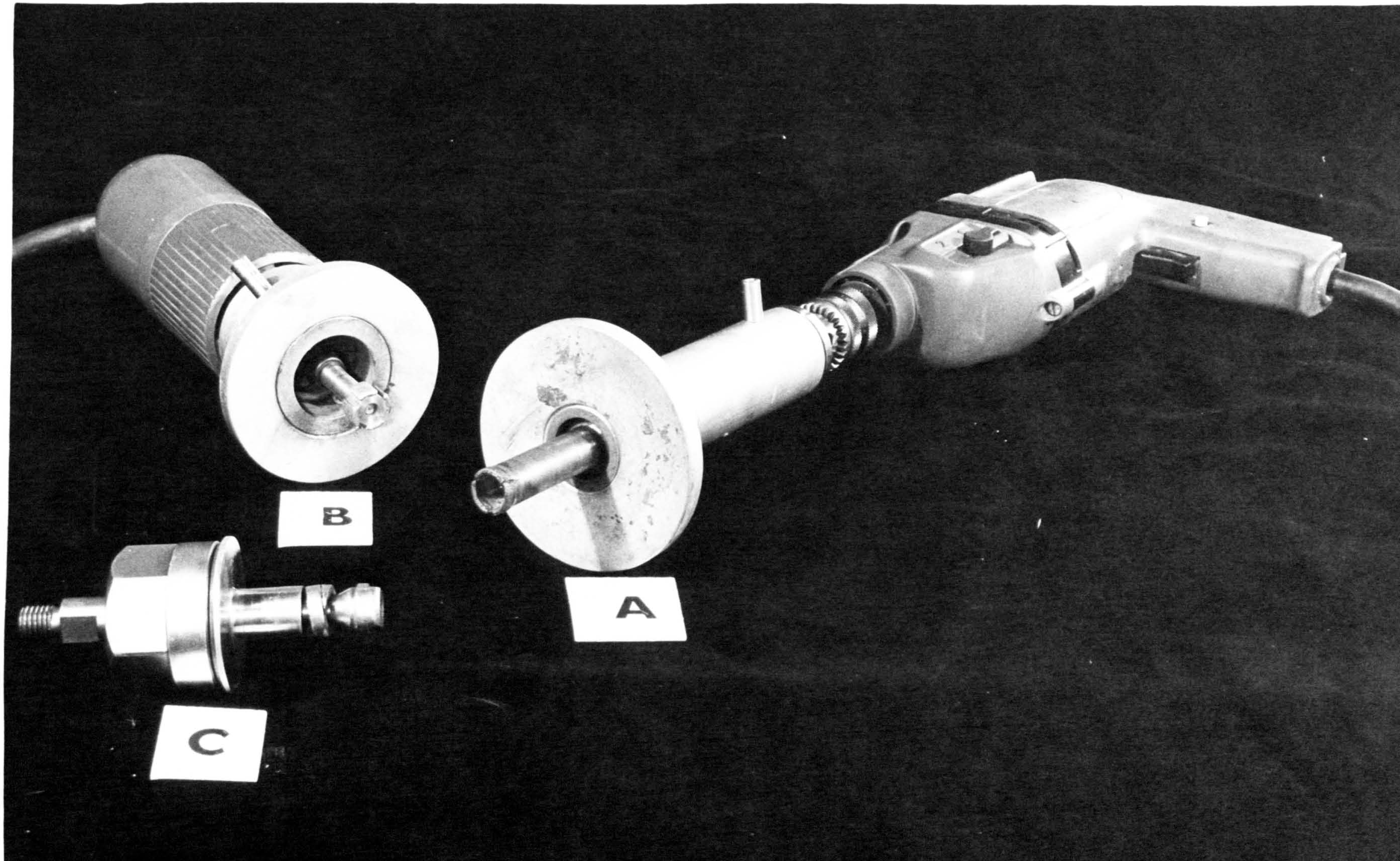


Figure 3.9: Apparatus for direct pull internal fracture test



A: drilling unit; B: milling cutter unit; C: expansion unit

Figure 3.10: Apparatus for Capo-Test

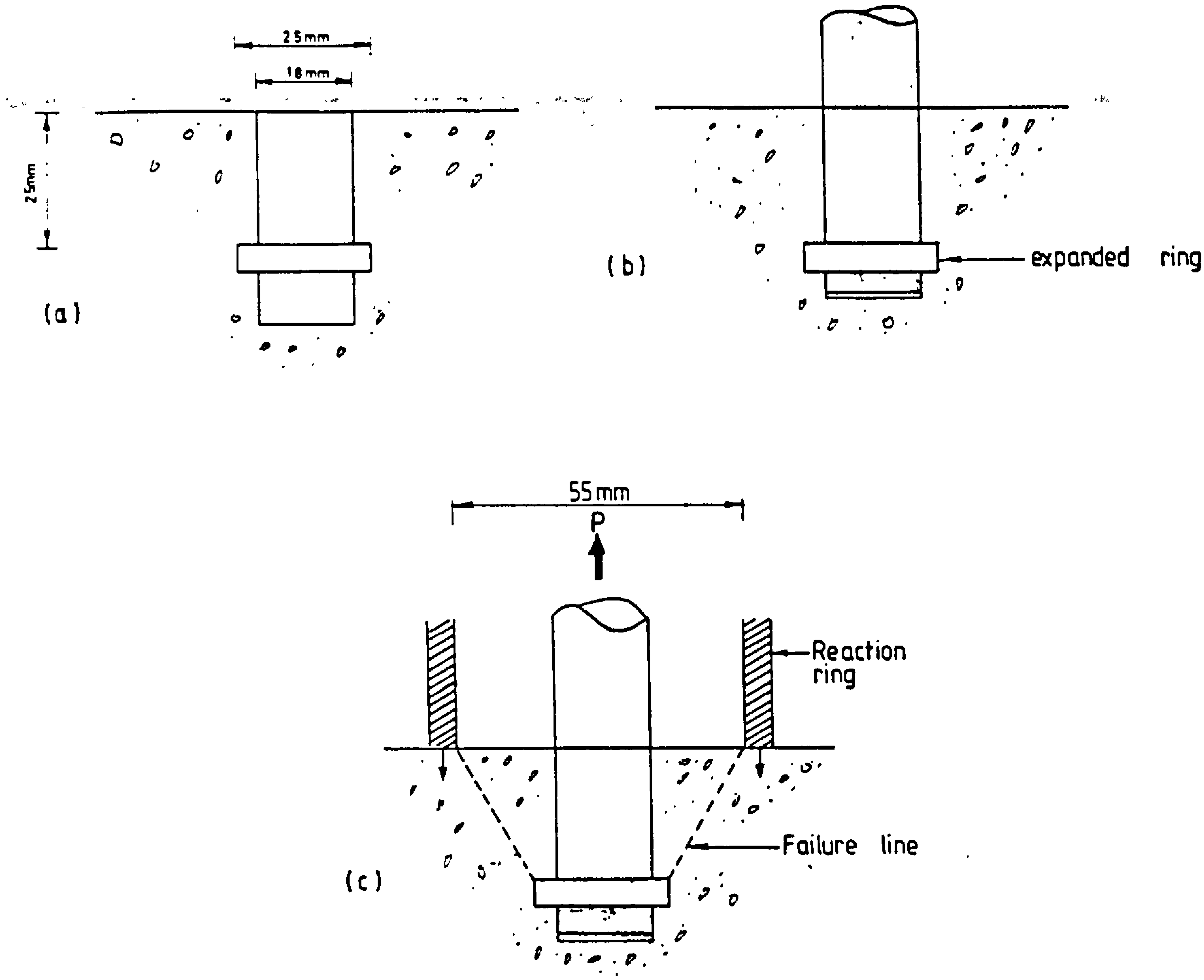


Figure 3.11: Capo-Test sequence

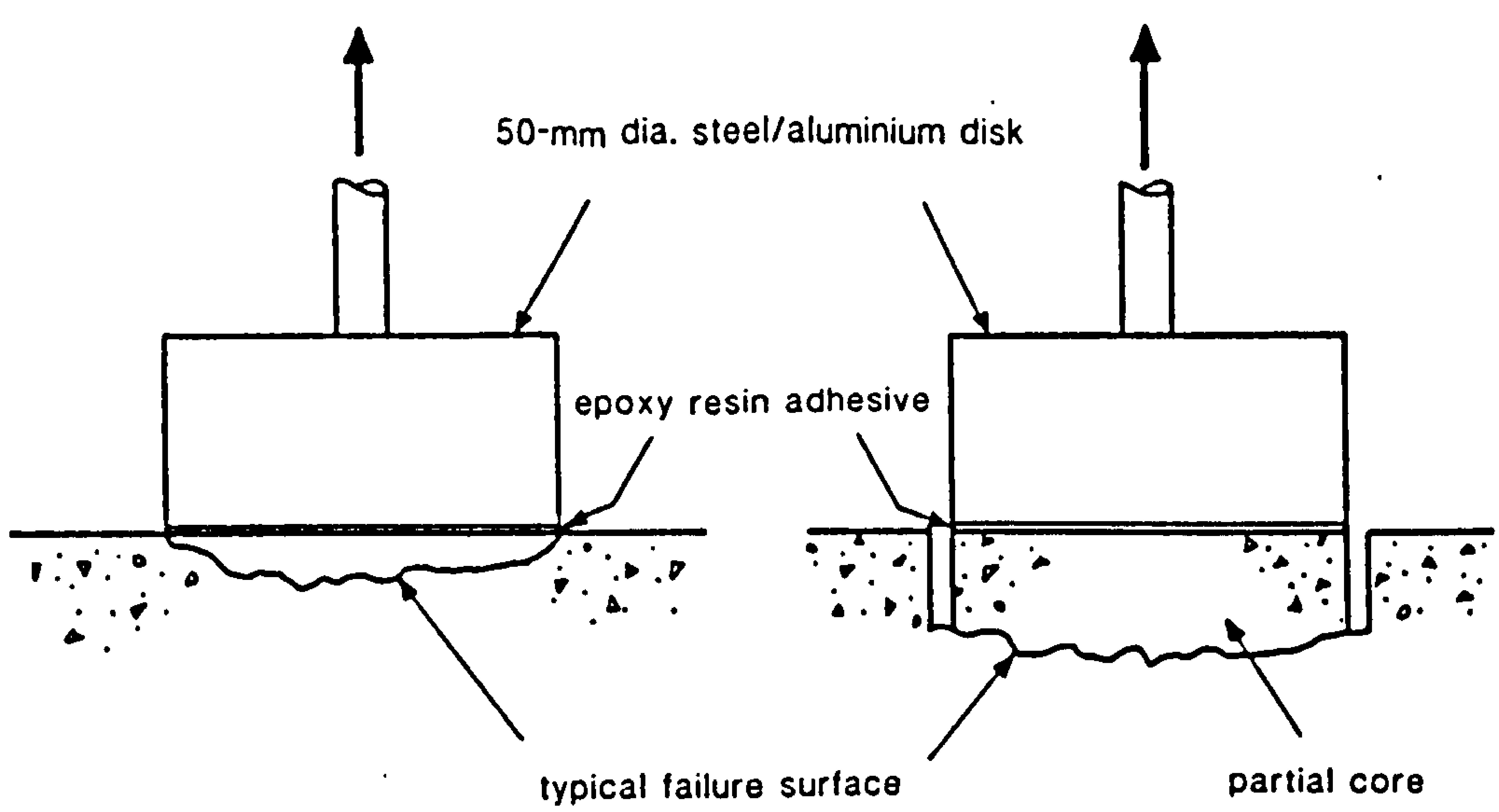


Figure 3.12: Test arrangement for surface and partial cored pull off tests

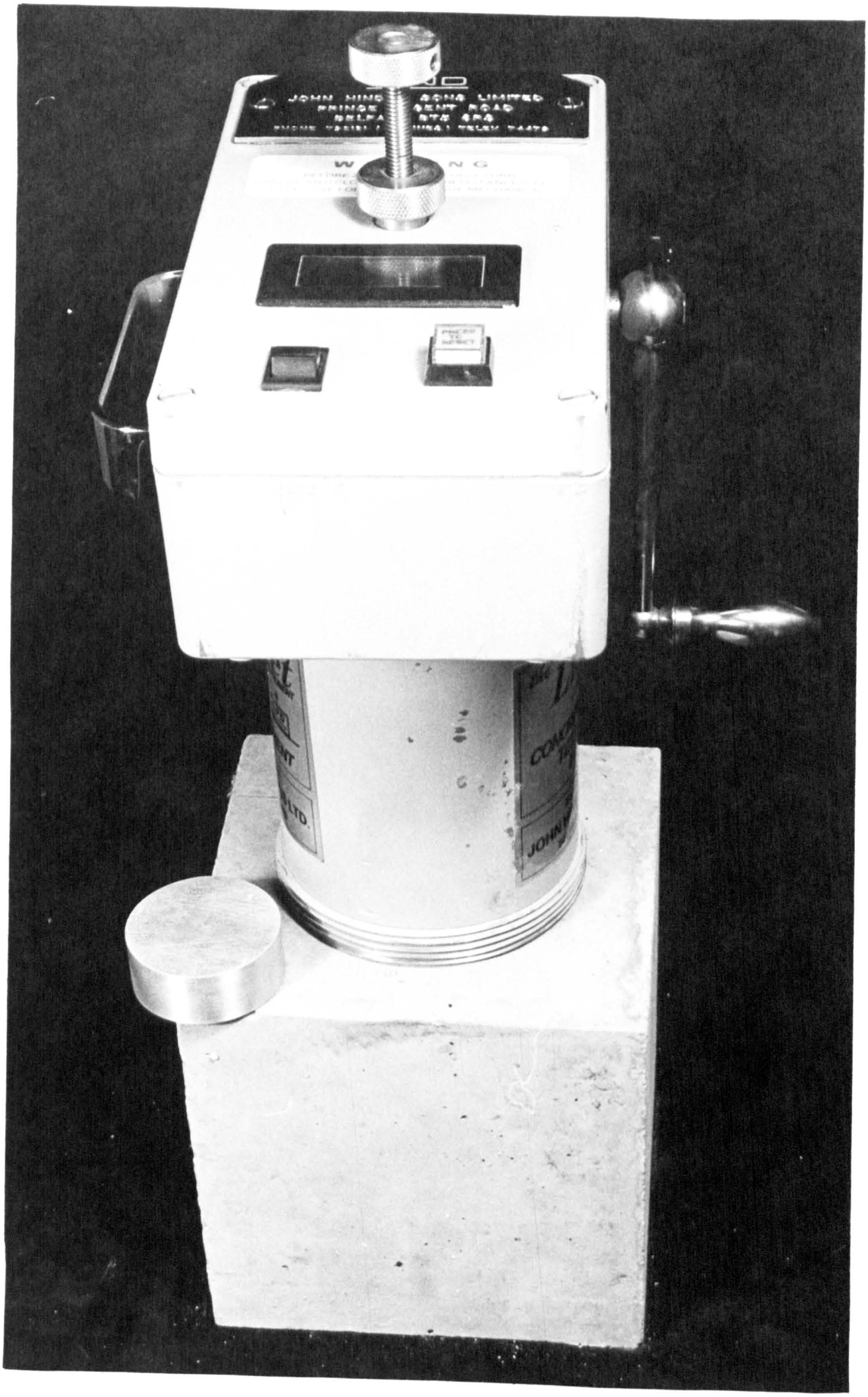


Figure 3.13: Apparatus for pull off test

CHAPTER 4. EXPERIMENTAL PROGRAMME

4.1 INTRODUCTION

This chapter deals with the laboratory tests which have been carried out by the Author. The experimental programme was designed to cover two parts. The first part involved the assessement of reliability of a number of insitu test methods. The tests were carried out on small concrete elements. Four different types of lightweight concrete have been examined incorporating a range of aggregate types. In addition, in some instances normal weight concrete has also been used for comparative study. Three basic parameters (strength levels, curing conditions and age of concrete) were considered throughout this part of the investigation. The second part was concerned with the dispersion and variation in the strength of concrete in large scale elements such as beams. A series of 2.2m long beams were used for this purpose.

4.2 MATERIALS

Three categories of concrete identified as fully lightweight, semi lightweight and normal weight were used for the entire study. Fine and coarse (12mm) particle sizes of Lytag were used for fully lightweight concrete, whereas coarse Lytag, Leca with 12mm particle sizes, or Pellite with 10mm particle sizes were used with North-Notts crushed sand to provide three alternative semi lightweight materials. Fully lightweight concretes made with Leca and Pellite were not used since fully lightweight Leca concrete is known to produce a very poor strength quality, whilst fully lightweight Pellite concrete gives a harsh mix with low cohesion due to the absence of small sized particles within the fine aggregates (Louati, 1988). Both are thus of limited value for structural concrete.

Preliminary trials with Pellite have confirmed the difficulty with fully lightweight concrete. Figure 4.1 compares the interior appearance of fully and semi lightweight Pellite concretes. For fully lightweight the absence of fine particles is clearly shown to increase the number of voids and this may affect strength properties as discussed in section 3.2.2.2(f).

For simplicity, shortened descriptions of the different types of lightweight concrete will be adopted. Fully and semi lightweight concretes made with Lytag will be referred to as fully and semi Lytag concretes, whereas the semi lightweight concretes made with Leca and Pellite will simply be called Leca and Pellite concretes.

For the normal weight concrete North-Notts crushed gravel and sand were used throughout the investigation. A combination of 20 and 10mm particle sizes of gravel were used as coarse aggregate.

Coarse aggregates (lightweight and dense) were always used in air-dried conditions. Also the sand was used in an air-dried condition, but the fine lightweight aggregate (Lytag), because of its high moisture content, as delivered, was used in a damp condition and the moisture content measured by Speedy Moisture

Tester. Later on in the research, it was found that sometimes the variation of compressive strength of fully lightweight concrete from batch to batch was quite high, so it was decided to use the fine aggregate in an oven dry condition also so that its moisture content could always be regarded as zero.

A grading analysis of all aggregates was carried out in accordance to BS 3797 : part 2 (1976) and BS 882 (1983). Typical grading curves are shown in Figures 4.2 through to 4.9. Concerning the fine lightweight aggregates, Lytag and Pellite could be best described by zones L-2 and L-1 respectively. As stated before, Figure 4.8 shows that for seive sizes below about 2mm, the fine Pellite is found to be coarser than zone L-1 by nearly 50%. This is considerably higher than the allowable 5% permitted by BS 3797 : part 2 (1976). In contrast, fine Lytag in Figure 4.7 seems to be finer than the specified zone L-2 by about 13% for sieve sizes below roughly 300μm.

The physical properties of aggregates such as specific gravity, water absorption and aggregate crushing value as 10% fines value described in section 2.2.2.3, are given in Table 4.1.

'Castle' Ordinary Portland cement was used throughout the investigation and supplied in bags. The main physical and chemical properties of the cement, as provided by the manufacturers, are given in Table 4.2.

The large scale beams were reinforced by 6mm and 12mm main bars and 6mm links to prevent cracking during demoulding and handling. The mild reinforcing steel had yield and ultimate strengths of $398N/mm^2$ and $509N/mm^2$ respectively.

4.3 MIX DESIGN

Four or six different mixes were designed for each type of concrete to give 28 day wet cube strengths of about $20\text{-}50N/mm^2$ except for concrete made with Leca where the strength range was between $14\text{-}25N/mm^2$. In the mix design of concretes, the main aim is to produce a concrete of minimum density with adequate workability and minimum cement content to achieve the required strength. At the present there is no standard method available for lightweight concrete, thus the mix designs are largely based on information provided by the aggregate manufacturers and previous research, consequently, to achieve a desired mix for this investigation, a larger number of trial mixes were made and suitable mixes were selected with no admixtures added to the concrete. Normal concrete mixes were proportioned using standard procedures based on 'The Design of Normal Concrete Mixes' by Teychenné et al (1988). The mix details for all types of concrete are given in Table 4.3.

4.4 MIXING PROCEDURE AND MANUFACTURE OF TEST SPECIMENS

The mixing of the concrete was done in a horizontal pan type batch mixer of capacity $0.1m^3$. For lightweight concrete, due to high water absorption of lightweight aggregates, the constituents were combined in such a way that would be applicable for site to produce a homogenous mix. The coarse lightweight aggregates (and fine lightweight aggregates if used) were placed in the mixer with approximately half of the water required for mixing. The aggregates and water were mixed for approximately one minute. For concrete made with Leca, due to weakness of aggregate, this step of mixing only was done by hand to prevent crushing the aggregates. Cement and sand (or cement only for fully lightweight) was then added and mixed for approximately 30 seconds. The remaining water was added and mixing continued for a further 1.5 minutes. In order to ensure uniformity of the concrete mix, it was hand mixed at the end of the machine-mixing period. The consistency of the fresh concrete was measured by the compacting factor test and results are given in Table 4.3 for all mixes.

For normal weight concrete, the mix procedure involved placing the coarse and fine aggregates and the cement in the pan, in the order listed. After rough dry mixing, water was added slowly and then it was mixed for 3 minutes. Similarly to lightweight concrete, the compacting factor test was adopted to measure the consistency of fresh concrete as given in Table 4.3.

All elements were cast in steel or wooden moulds which had first been lightly oiled. Elements used for the first part of the investigation included standard laboratory specimens and unreinforced beams. These are summarized in Table 4.4. For the large scale test beams, a wooden mould was designed with dimensions 2.2 × 0.3 × 0.5m.

The Large Beams, along with 100mm cubes as control specimens, were cast from five batches of concrete. In order to limit batching error, the constituent materials were weighed and bagged the day before casting. On the day of mixing, first the pan and mixer blade were dampened with grout before adding the batch components. This may help provide uniformity of mix between the first and the remaining batches. Details of the beam mould and the arrangement of reinforcement are shown in Figure 4.10.

The small elements and specimens were compacted on a vibrating table. The 100mm cubes, along with prisms, were compacted in two layers whereas other elements were compacted in three layers. The large beams were vibrated by internal vibrator in five layers. In general, each layer was vibrated until full compaction was obtained as indicated by removal of the majority of entrapped air, with minimum segregation and bleeding. The top of all specimens were levelled and finished with a steel trowel after compaction.

4.5 CURING

All elements were covered with wet hessian and polythene sheets 2 hours after casting. The small elements were demoulded after 24 hours. Two curing regimes were then adopted, wet and dry. The elements for wet curing were stored up until the time of testing in the moist curing room at a temperature of 18°C to 20°C, according to the BS 1881 : part 111 (1983), whilst the dry cured elements were cured in the laboratory under polythene for 3 days and then under uncontrolled lab conditions. The curing conditions were monitored by means of a thermohydrograph and the average conditions found to be about 21°C with a relative humidity of 75%. For the large scale beam, the sides of the mould and control cube specimens were demoulded after 24 hours and kept covered under wet hessian and polythene for the subsequent 6 days. At the age of 7 days, the base of the beam mould was removed and then the beam and control cube specimens exposed to the uncontrolled laboratory condition.

4.6 DETAILS OF TESTS

a) On small elements

The insitu tests along with standard laboratory tests were performed on specified concrete elements made as described in section 4.4. A summary of test details on small elements are given in Table 4.5.

The procedure for use of each type of insitu equipment was fully described in Chapter 3. Test positioning on elements and preliminary test preparation were as follows.

The rebound hammer tests were performed on two opposite side faces and on the bottom face of 100mm cubes with five tests on each face. In pulse velocity measurements, the pulse path was between two opposite side faces of 100 mm cubes and the readings taken from the digital readout which showed the time in microseconds. Usually, the last digit (the 0.1 microsecond) of the display fluctuated and in this circumstance the lower reading was recorded. The readings recorded were subsequently converted to pulse velocity.

The Windsor Probe tests were carried out by firing the golden probes on side faces of 1000 × 150 × 250mm beams of fully Lytag concrete. The hardness of Lytag was determined by using the scratch mineral kit provided by the manufacturer and it was found to be Moh's No.3. The probes were fired singly at the mid-height 170mm apart from each other when the beam was supported on the other side face. Each beam was designed for Windsor Probe tests at two consecutive ages, 7 and 28 days, and each side was used for one particular age. The positions of probes were distributed in a manner that the lines of probing did not coincide from the two side faces. In pull out tests, the Lok-Test assemblies were installed in the side and bottom faces of the large cube moulds before concreting. For top surfaces the Lok-Test assemblies were installed by means of a plate and plastic buoyancy cup to ensure that assemblies floated and a good testing surface maintained. These inserts were loaded using Lok-Test model L12.3 equipment. For internal fracture tests (B.R.E. and direct pull), tests were located on two opposite side faces and one on the bottom face of 150mm cube. Hole drilling operations as part of test preparation were carried out by rotary hammer drill with a nominal 6mm bit. The drill was connected to a special frame to improve the drilling accuracy. The drill bit diameter was normally examined to ensure the proper hole size with 'Go-NoGo' plate supplied by B.R.E. internal fracture manufacturer. For the wet cured condition, the cubes were removed from the moist curing room two hours before testing. Hole drilling was undertaken and the specimen left in the laboratory to dry for easy removal of dust. Parabolts, as commercially available in U.K., were used as expanding wedge anchor bolts with nominal 6mm diameter and 85mm length, and threaded at the rear. For direct pull internal fracture, the calibration of the proving ring was done in a 100kN Denison testing machine. Tensile force was applied up to 10kN. Dial gauge readings were recorded at 1kN intervals. The calibration chart is shown in Figure 4.11. In pull off tests, two tests were performed on each cube where opposite side faces were considered for testing. Aluminium disks were used, generally with 50mm diameter, although in some instances 40mm diameter was necessary due to the limited loading capacity of the Limpet apparatus which was used throughout. The adhesive used for bonding the aluminium disks to the surface of the concrete was commercially available Devcon "5 minute" Epoxy Resin.

The 50mm nominal diameter cores were cut either vertically or horizontally at the age of 28 days, and in some instances at age 6 and 12 months, using a diamond-tipped core cutter as shown in Figure 4.12. A special clamp was used to hold the elements during cutting, to limit relative movement between the element and rig. The core positioning, as drilled vertically, is shown in Figure 4.13 in which the cores are 50mm apart from each other. For horizontal drilling, the cores were drilled at mid-height along the length of beam. After drilling, the cores were trimmed with a diamond impregnated saw and were then left to dry under laboratory conditions. On the following day, the cores were capped with sulphur and sand to given an overall length/diameter (ℓ/d) ratio between 1.0 and 2.0. For capping a special device was designed to ensure that they came out smooth, plane-parallel and at right angles to the axis of the specimen. The reason for capping when the core was dry that a wet core would cause adhesion problems between the cap and the core face. Because of the heat involved in capping, the water present on the core face would evaporate when capped, causing small pockets of voids on the capped surface. After completion of core capping, the cores for dry curing were left in the laboratory for 24 hours prior to compression testing whereas the compression tests were applied on wet cured cores after 48 hours storage under water as described in BS 1881 : part 120 (1983). At the day of testing, prior to loading, the length and the diameter of each core was measured at different places and then the average considered. Compression testing of the cores was carried out at the rate of $0.3N/mm^2.sec$ (in the range given by BS 1881 : part 120 (1983)). Those cores with an anticipated measured strength of $40N/mm^2$ or less were tested in a 100kN capacity Avery machines, whilst a 3000kN capacity Dension machine was used for stronger cores.

The cube compressive strength corresponding to each insitu test was obtained from 100 or 150mm cubes according to BS 1881 : part 116 (1983) with the loading applied at the rate of $0.3N/mm^2.sec$ (in the range given by the standard) using the Dension 3000kN compressive machine. Compressive strengths related to Windsor Probe, pull out, and core tests were obtained from groups of three companion 100mm cubes under identical conditions and tested at the same ages. For the other methods the actual tested cubes were crushed and in some cases predetermined corrections applied to obtain compressive strength values.

Tensile splitting tests on cylinders were carried out according to BS 1881 : part 117 (1983) in diametral compression with a rate of loading of $0.03.N/mm^2.sec$ (in the range given by the standard) using Dension 3000kN compressive machine.

Static and dynamic modulus of elasticity were determined to BS 1881 : part 121 (1983) and BS 1881 : part 5 (1970) respectively. In the static modulus of elasticity, Demec points at 100mm centres were fixed in the central zone on two sides of each cylinder in the longitudinal direction, and the strain readings were taken with a mechanical Demec strain guage having a sensitivity of 1.62×10^{-5}m/m per division. The dynamic modulus of elasticity was obtained by the Electrodynamic Material Tester SCT4, type 1821A and made by DAWE instruments Ltd. The apparatus consisted of an electro-magnetic exciter unit, an electro-magnetic pick up unit and a digital counter unit which measures the natural frequency of the fundamental mode of longitudinal vibration of the element.

b) On large scale beams

The non-destructive testing techniques of ultrasonic pulse velocity along with rebound hammer and the partially destructive pull-out testing technique were used on a total of five beams cast from different types of concrete (Table 4.6) and tested initially at age 28 days. On each beam, fifteen pull out inserts were placed prior to concreting by fastening to the inside of each side face of the mould. The inserts were positioned at three levels along the length with five inserts on each row as shown in Figures 4.10, 4.14 and 4.15. The positioning of the inserts as shown in Figure 4.15 was selected to eliminate edge and reinforcement effects as specified by Lok-Test manufacturer (minimum distance from edge and reinforcement were recommended as 100 and 20mm respectively) and satisfied the proposed requirement of BS 1881 : part 207 (1991). To identify test locations for ultrasonic measurements, a layout of test points were marked on both side faces of each beam as shown in Figures 4.14 and 4.16. Locations of the counterpoints were established through measurements along and across the layout to ensure that each pair of related points was located on a perpendicular to the side faces. Attempts were made to ensure that there were no reinforcing bars along or perpendicular to the path length. Measurements of the pulse path were taken by using a specially built rigid caliper with the facility of a mechanical gauge of accuracy of 0.01mm as shown in Figure 4.14.

The caliper had two parallel and equal branches of which one end branch was always placed tightly against the test point. A 300mm bar, was calibrated and the gauge reading was recorded as a reference number. The desired path lengths were obtained by subtracting the gauge reading from the reference number and subsequently multiplying by 0.01 followed by adding or subtracting from 300mm depending upon whether the gauge reading was greater or smaller than the reference number. Rebound hammer measurements were obtained on one side face, in three levels along the length with ten readings at each location within an area of approximately 120 x 150mm as shown in Figure 4.17.

At the age of six months further tests were conducted on fully Lytag concrete only using the non-destructive technique of ultrasonic pulse velocity, the partially destructive testing technique of Capo-Test, and the destructive testing technique of cores. The layout of test points for these tests are shown in Figure 4.18.

Table 4.1 :Physical properties of aggregates

Type of Aggregate	Specific gravity			Water Absorption		Crushing Value
	Oven Dried	Saturated Surface Dried	Apparent	24 hr %	1/2 hr %	KN
Coarse Lytag	1.60	1.82	2.01	13	11	70(8.2% fine)
Coarse Leca	0.63	0.84	0.80	32	9.6	28(9.0% fine)
Coarse - Pellite	1.77	1.85	1.93	4.7	3.6	40(11.5% fine)
North Notts Gravel	2.62	2.63	2.64	0.35	0.2	200-300 See sec 2.2.2.3
Fine Lytag	1.74	2.00	2.35	15	11.6	-
North Notts Sand	2.62	2.64	2.67	0.78	0.35	-

Table 4.2 :Physical properties and chemical analysis of cement

Description of Test		
Setting times: Initial		165 mins
Final		240 mins
Specific surface		360 m^2/Kg
Physical tests-mortar strength(w/c=0.6)		
Compressive strength of 100 mm cubes		N/mm^2
	3 day	27
	7 day	37
	28 day	47
Chemical analysis	(% by weight)	
CaO	64.21	
SiO_2	20.88	
Al_2O_3	5.00	
Fe_2O_3	3.17	
MgO	2.42	
K_2O	0.78	
Na_2O	0.32	
SO_3	3.50	
Free CaO	1.30	
Loss on ignition	1.02	
Insoluable residue	0.46	

Table 4.3 :Details of concrete mixes

Type of concrete	Mix No.	28 day Wet cube Strength (N/mm^2)	Material per cubic metre: Cement (Kg)	Water (Kg)	Fine Aggregate (Kg)	Coarse Aggregate (Kg or m^3)	Compacting Factor
Fully Lytag concrete	L-A	23	200	300	750	554 Kg	0.93
	L-B	32	226	287	633	628 Kg	0.91
	L-C	35	259	287	601	642 Kg	0.92
	L-D	42	329	289	526	657 Kg	0.93
Semi Lytag concrete	L-1	19	201	258	790	717 Kg	0.95
	L-2	30	240	257	753	715 Kg	0.95
	L-3	39	301	258	695	717 Kg	0.94
	L-4	48	345	260	658	722 Kg	0.96
Leca concrete	Le-1	14	230	210	840	0.75 m^3	0.88
	Le-2	19	320	204	770	0.75 m^3	0.94
	Le-3	23	380	204	720	0.75 m^3	0.95
	Le-4	25	508	230	662	0.74 m^3	0.95
Pellite concrete	P-1	20	245	225	836	705 Kg	0.86
	P-2	34	316	227	790	710 Kg	0.95
	P-3	36	350	225	740	740 Kg	0.94
	P-4	41	370	225	730	740 Kg	0.96
	P-5	45	405	227	710	740 kg	0.92
	P-6	52	490	225	680	740 Kg	0.93
Normal concrete	N-1	20	217	195	878	749 Kg(20mm) + 321 Kg(10mm)	0.94
	N-2	24	244	195	836	760 Kg(20mm) + 326 Kg(10mm)	0.95
	N-3	32	279	195	830	739 Kg(20mm) + 317 Kg(10mm)	0.94
	N-4	43	361	195	825	685 Kg(20mm) + 294 Kg(10mm)	0.92

Table 4.4 :Details of elements used for different tests

Type of Element	Dimension mm	Operation
Beam	650x225x120	50 mm core
	1000x150x250	Windsor Probe
Cubes	100	Rebound hammer,pulse velocity, pull off,compression test
	150	B.R.E. internal fracture, direct pull internal fracture, pull off,pulse velocity, compression test
	200	Lok-Test
Cylinder	150x300	static modulus of elasticity, splitting tensile test
Prism	500x100x100	dynamic modulus of elasticity, pulse velocity
Large Scale Beam	2200x300x500	Lok-test,Capo-Test,50 mm core, Pulse velocity,Rebound hammer

Table 4.5 :Details of tests

Type of Test	No. of Tests: No. of Specimens	No. of Tests: No. of Tests per specimen	Type of Concrete	Age at Test Days	Curing Regime	Remarks
Rebound hammer	3	15	All types of concrete	1,2,3,5,7, 14,21,28	Dry+	On mixes L-C,L-3, Le-4,P-5,N-4
Pulse velocity	3	1-2	All types of lightweight concrete excluding semi light-weight Lytag concrete	1-360 at various ages	Wet,dry	Mix P-4 not used
			Semi Lytag concrete	1,2,3,5,7, 14,21,28	Dry+	
				1-28 at various ages	Wet,dry	
			Normal weight concrete	1,2,3,5,7, 14,21,28	Dry+	
				28	Wet,dry	
				180	Wet	
Windsor Probe	1	3	Fully Lytag concrete	7,28	Wet,dry	Low power
Pull out	1	6	Fully Lytag concrete	7,28,180,360	Wet,dry	
			Semi Lytag concrete	7,28	Dry	
			Leca concrete	7,28	Wet	
			Pellite concrete	7,28	Dry	Mixes P-4,P-6 not used
			Normal weight concrete	360	Wet	
B.R.E. internal fracture	2	3	Fully Lytag concrete	7,28,180,360	Wet,dry	
			Semi Lytag concrete	7,28	Wet,dry	Mixes L-2,L-3 tested at wet curing only
			Leca concrete	7,28	Wet	
			Pellite concrete	7-28 at various ages	Wet	Mixes P-4,P-6 not used

Continued....................

Table 4.5 (Contd.)

Type of Test	No. of Tests: No of Specimens	No. of Tests: No. of Tests Per specimen	Type of Concrete	Age at Test Days	Curing Regime	Remarks
Direct pull internal fracture	2	3	Fully Lytag concrete	7,28,180,360	Wet,dry	
			Semi lytag concrete	7,28	Wet,dry	Mixes L-2,L-3 tested at wet curing only
			Leca concrete	7,28	Wet	
			Pellite concrete	7-28 at various ages	Wet,dry	Mixes P-4,P-6 not used
Pull off	3	2	Fully Lytag concrete	7,28,180, 360	Dry	
			Semi Lytag concrete	7,28	Dry	
			Leca concrete	14,28,360	Dry	
			Pellite concrete	7-28 at various ages	Dry	Mix P-6 not used
			Normal weight concrete	7,28,180,360	Dry	
Core^{x}	3	1	Fully Lytag concrete	31,180,360	Wet,dry	180 and 360 days tests were used only for mixes L-A and L-D
			Semi Lytag concrete	31	Wet	
			Leca concrete	31	Wet	
			Pellite concrete	31	Wet,dry	Mixes P-4,P-6 not used
Static modulus of elasticity	3	1	Fully Lytag concrete	28	Dry	
			Leca concrete	28	Dry	Mixes Le-1,Le-4
			Pellite concrete	28	Dry	Mixes P-1,P-5
Dynamic modulus of elasticity	3	1	Fully Lytag concrete	28,360	Dry	
				1-360 at various ages	Wet,dry	Only on mix L-D
			Semi Lytag concrete	1-28 at various ages	Wet	
			Leca concrete	1-360 at various ages	Wet,dry	Mixes Le-1,Le-4
			Pellite concrete	1-360 at various ages	Wet,dry	mixes P-1,P-5
			Normal weight concrete	28,180	Wet	

Continued..................

Table 4.5 (Contd.)

Type of Test	No. of Tests: No. of Specimens	No. of Tests: No. of Tests per Specimen	Type of Concrete	Age at Test Days	Curing Regime	Remarks
Tensile splitting strength	3	1	Fully Lytag concrete	28,360	Wet,dry	
			semi Lytag concrete	28	Wet,dry	
			Leca concrete	28	Wet,dry	
			Pellite concrete	28	Wet,dry	Mixes P-3,P-4 not used
			Normal weight concrete	28,360	Dry	
Compressive strength	3	1	Fully Lytag concrete	1-360 At various ages	Wet,dry	
			Semi Lytag concrete	1-28 At various ages	Wet,dry	
			Leca concrete	1-28 At various ages	Wet,dry	
				360	Dry, Wet+dry	
			Pellite concrete	1-28 at various ages	Wet,dry	
			Normal weight concrete	1-360 at various ages	Wet,dry	

+; Same curing as for large scale beam

x; At different L/D ratios from 1.0 to 2.0

Table 4.6:Mix details used in large scale beams

Type of Concrete	Mix No.
Fully Lytag Concrete	L-C
Semi Lytag Concrete	L-3
Leca Concrete	Le-4
Pellite Concrete	P-5
Normal Weight Concrete	N-4

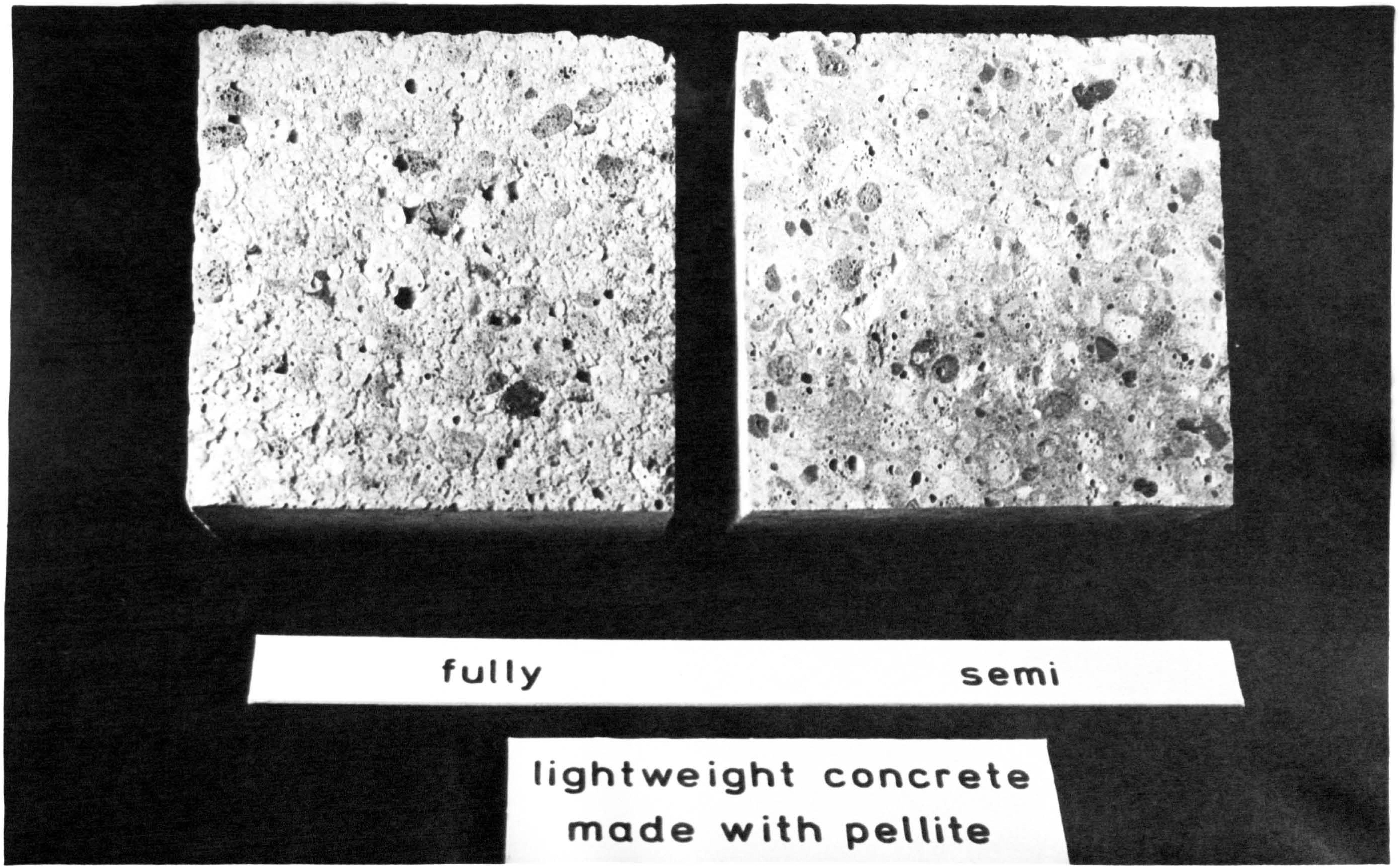


Figure 4.1: Interior appearance of fully and semi lightweight Pellite concrete

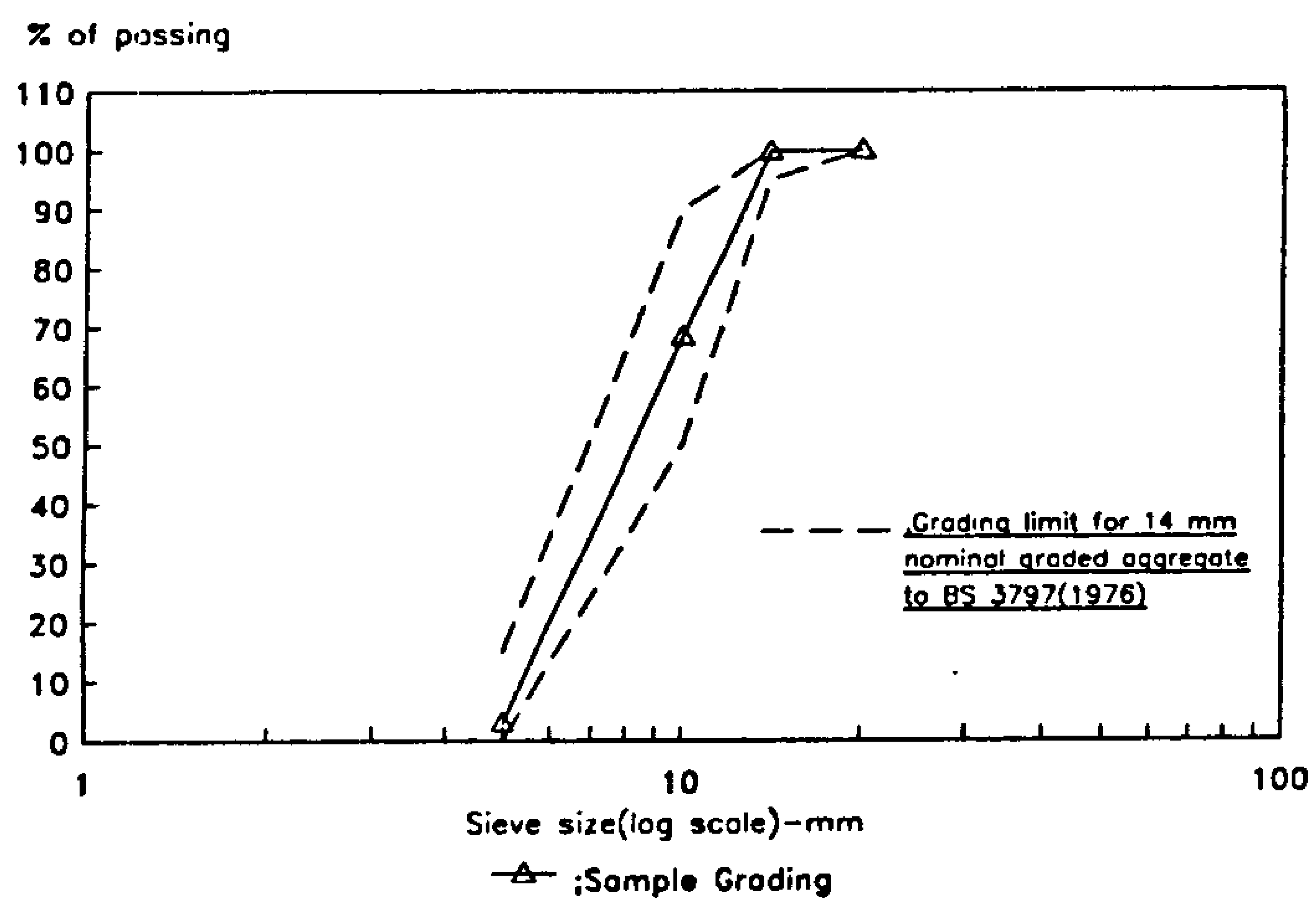


Figure 4.2.Typical grading curve for coarse Lytag

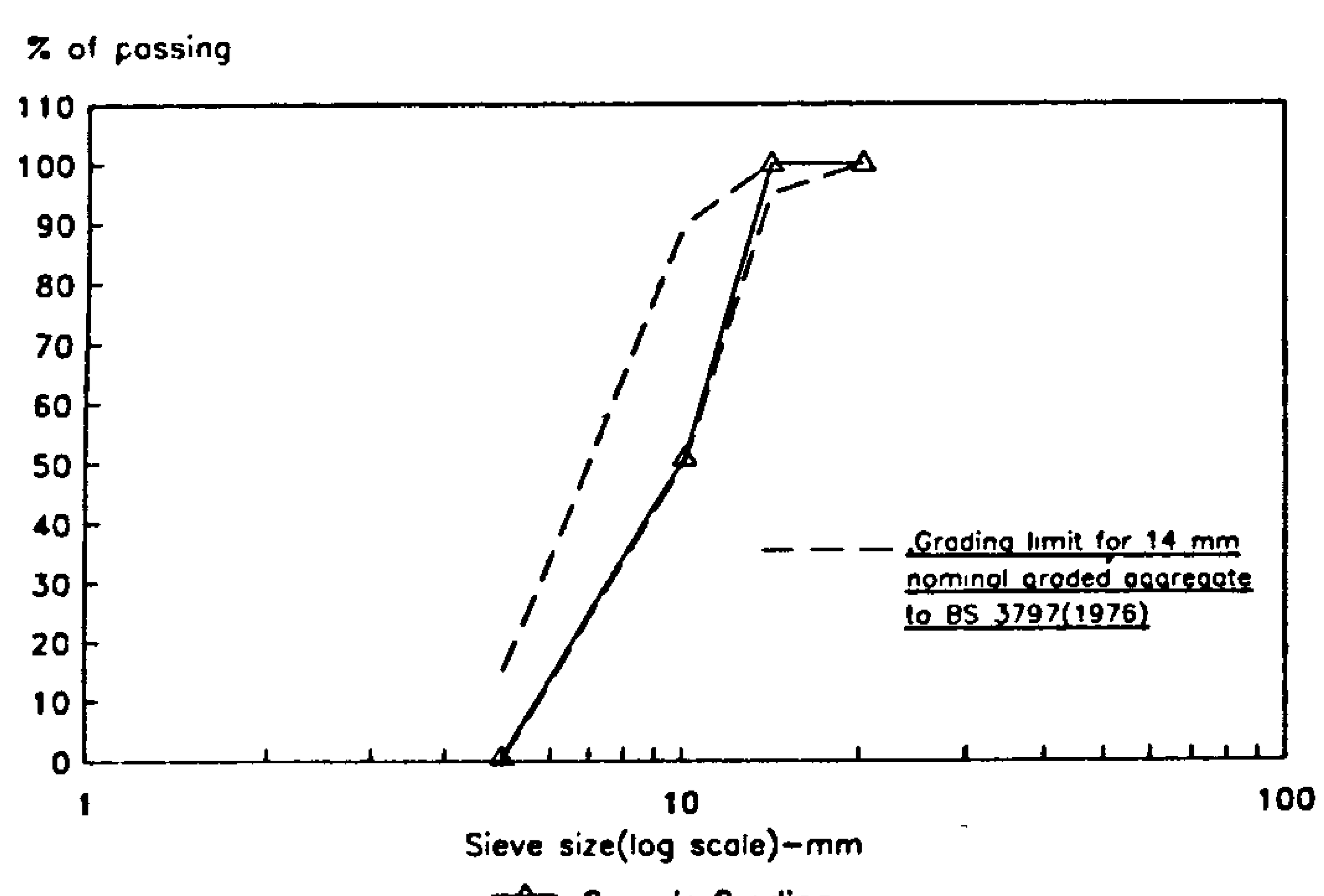


Figure 4.3 Typical grading curve for coarse Leca

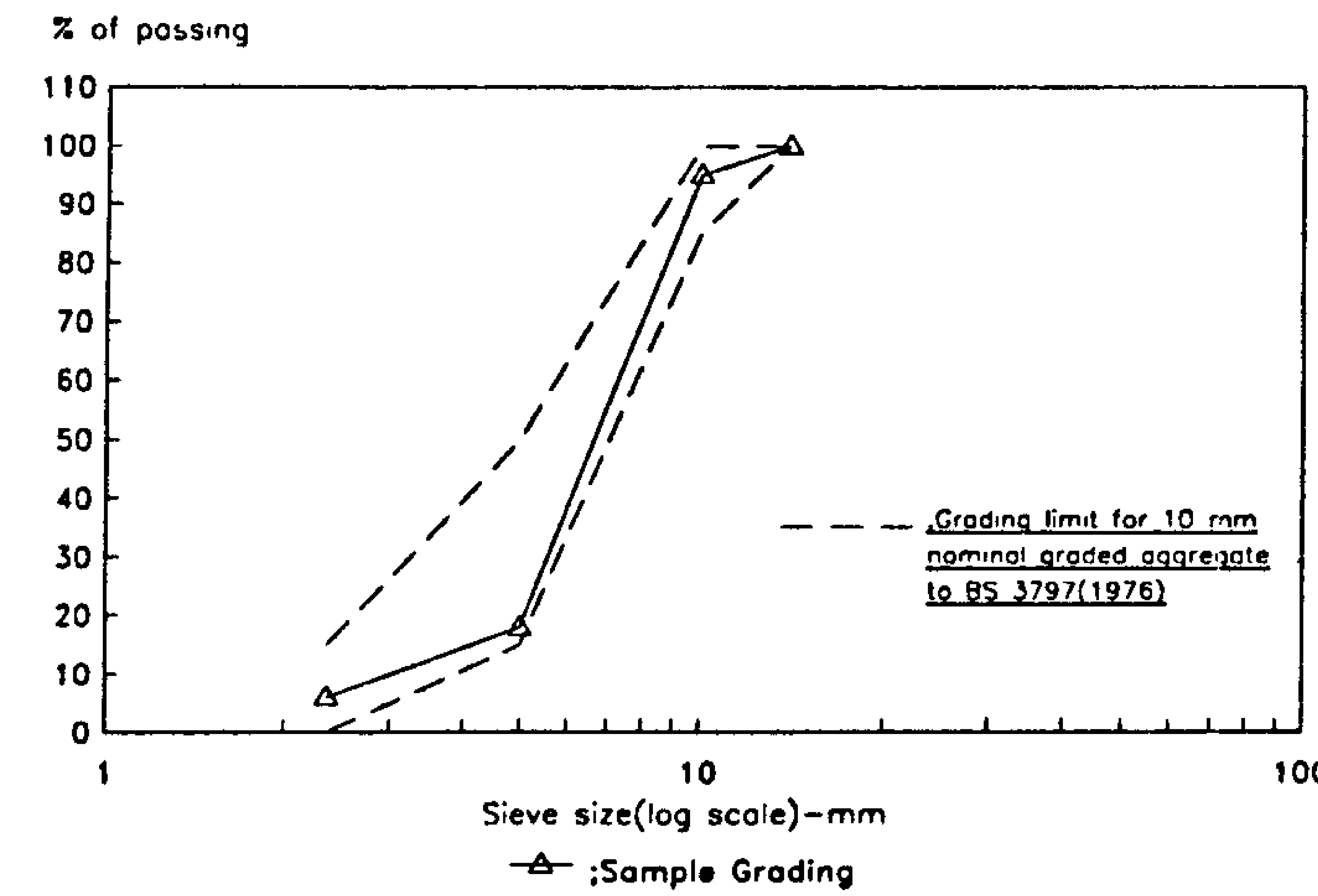


Figure 4.4.Typical grading curve for coarse Pellite

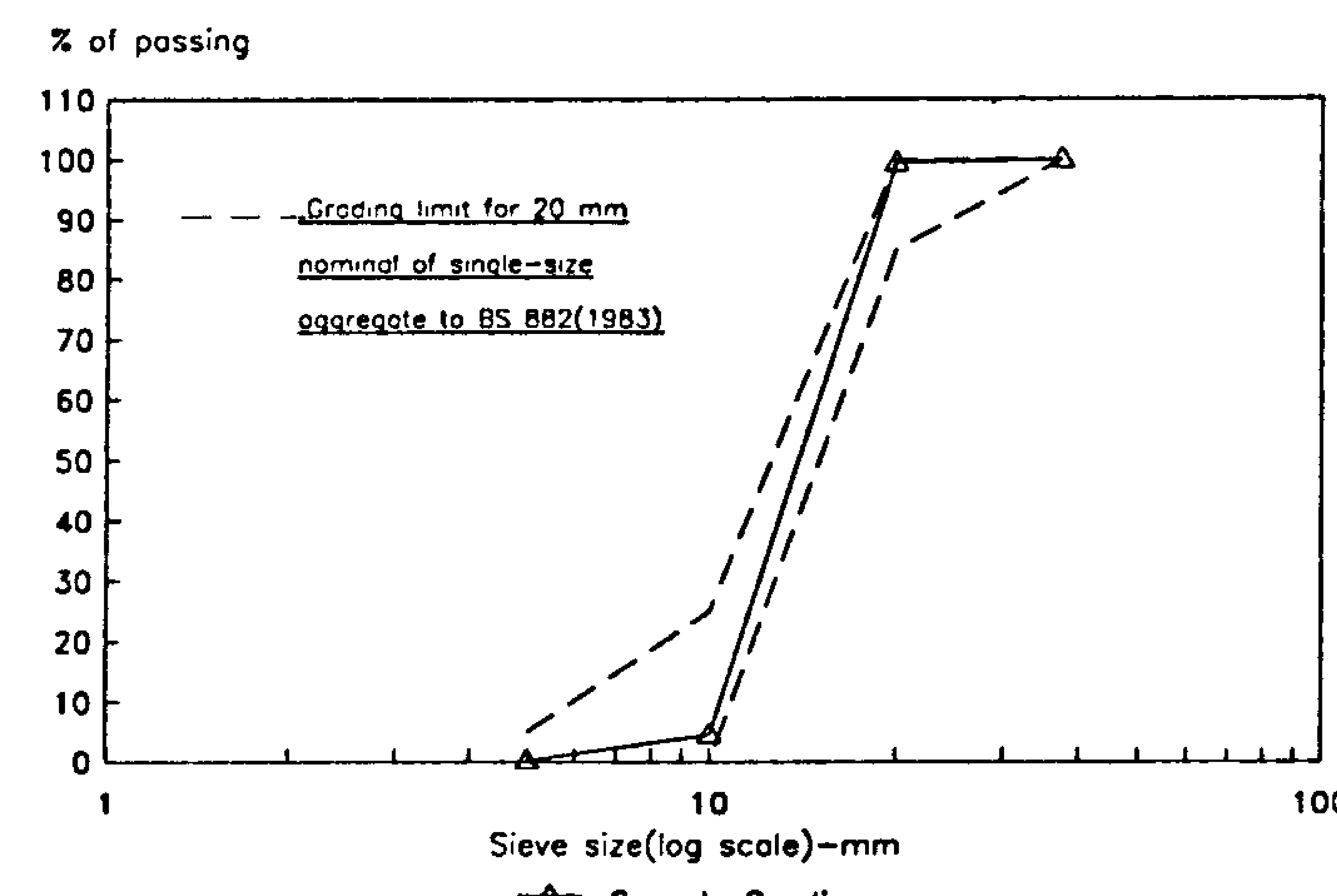


Figure 4.5 Typical grading curve for coarse 20 mm North-Notts gravel

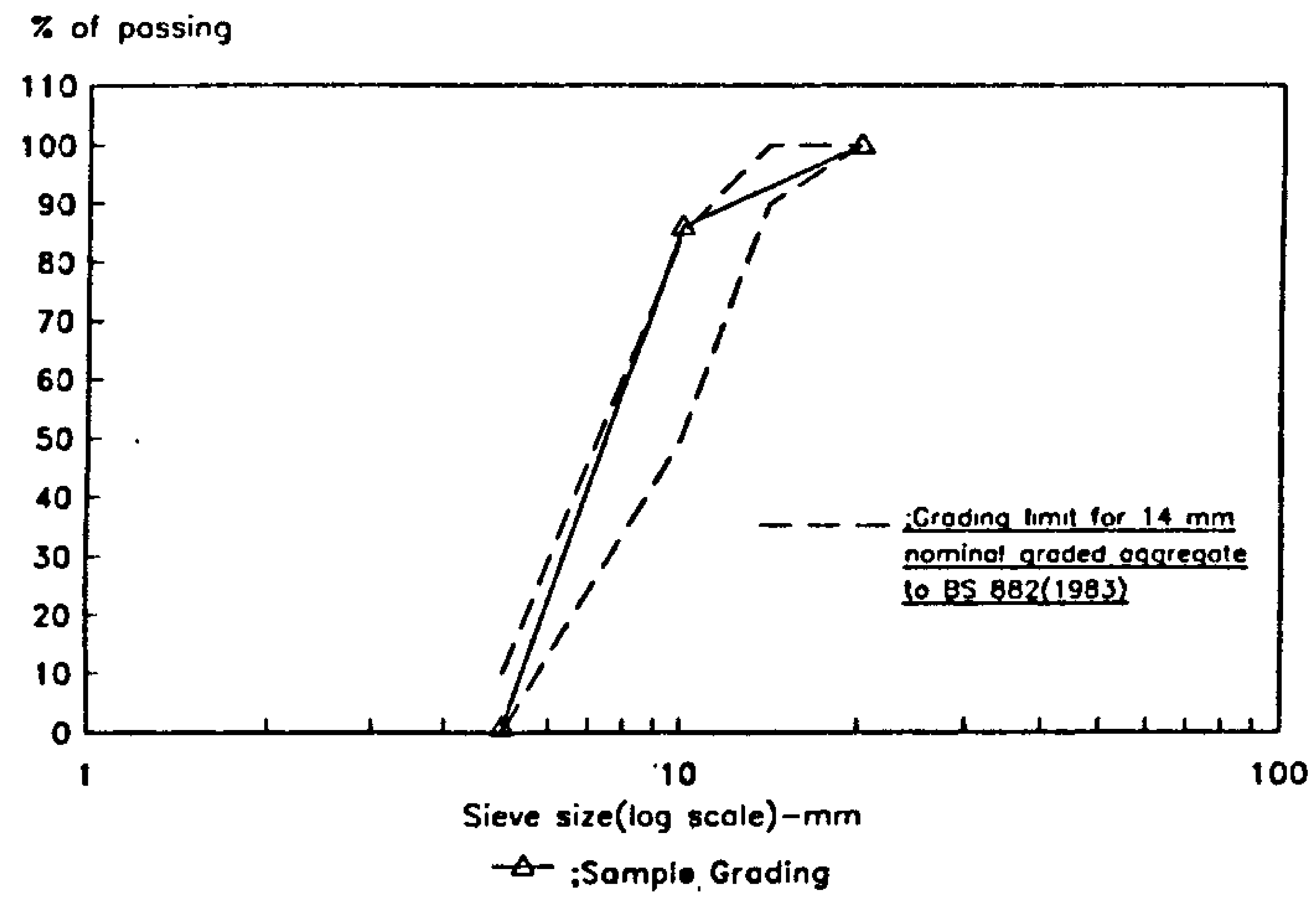


Figure 4 6:Typical grading curve for coarse 10 mm North-Notts gravel

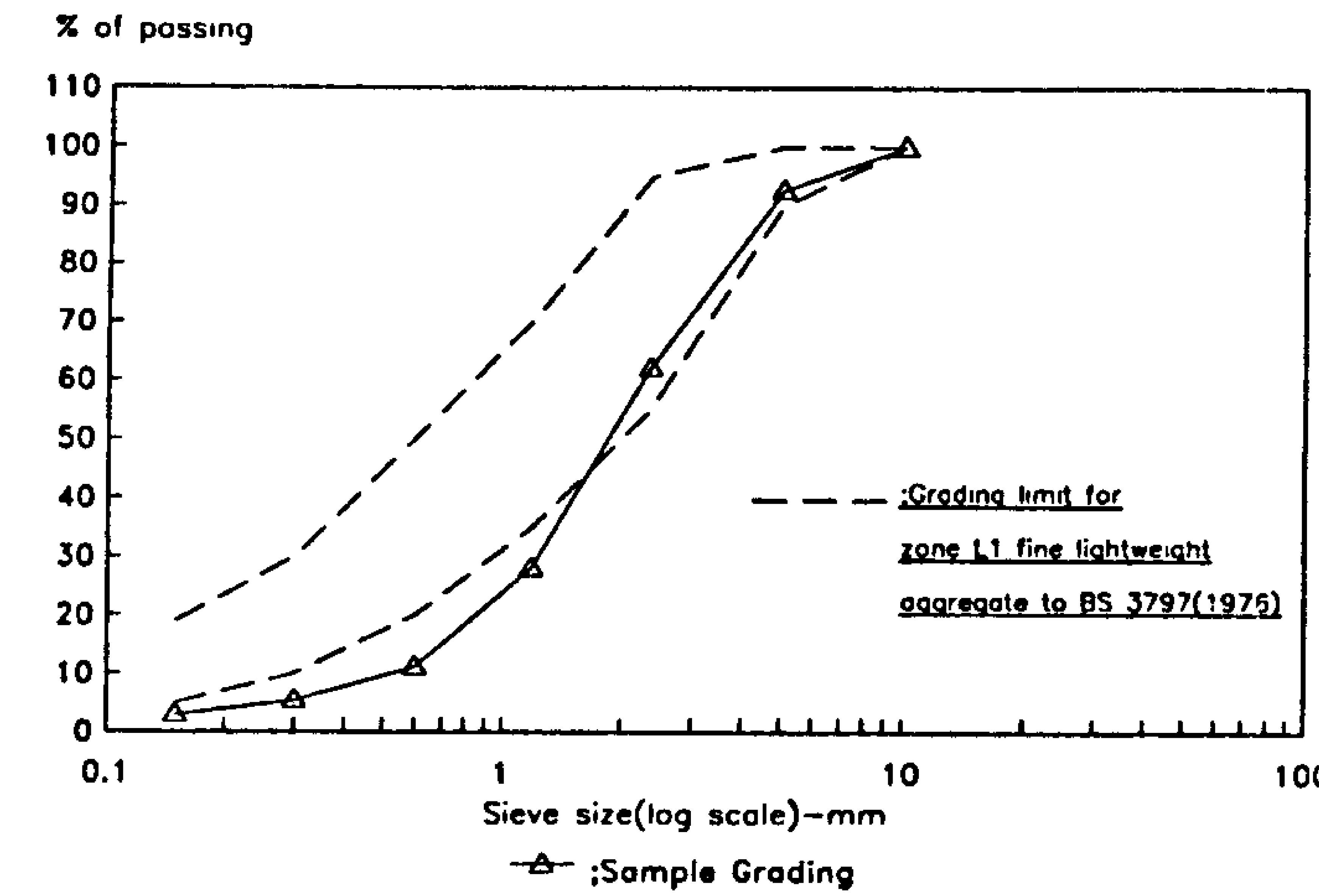


Figure 4.8:Typical grading curve for fine Pellite

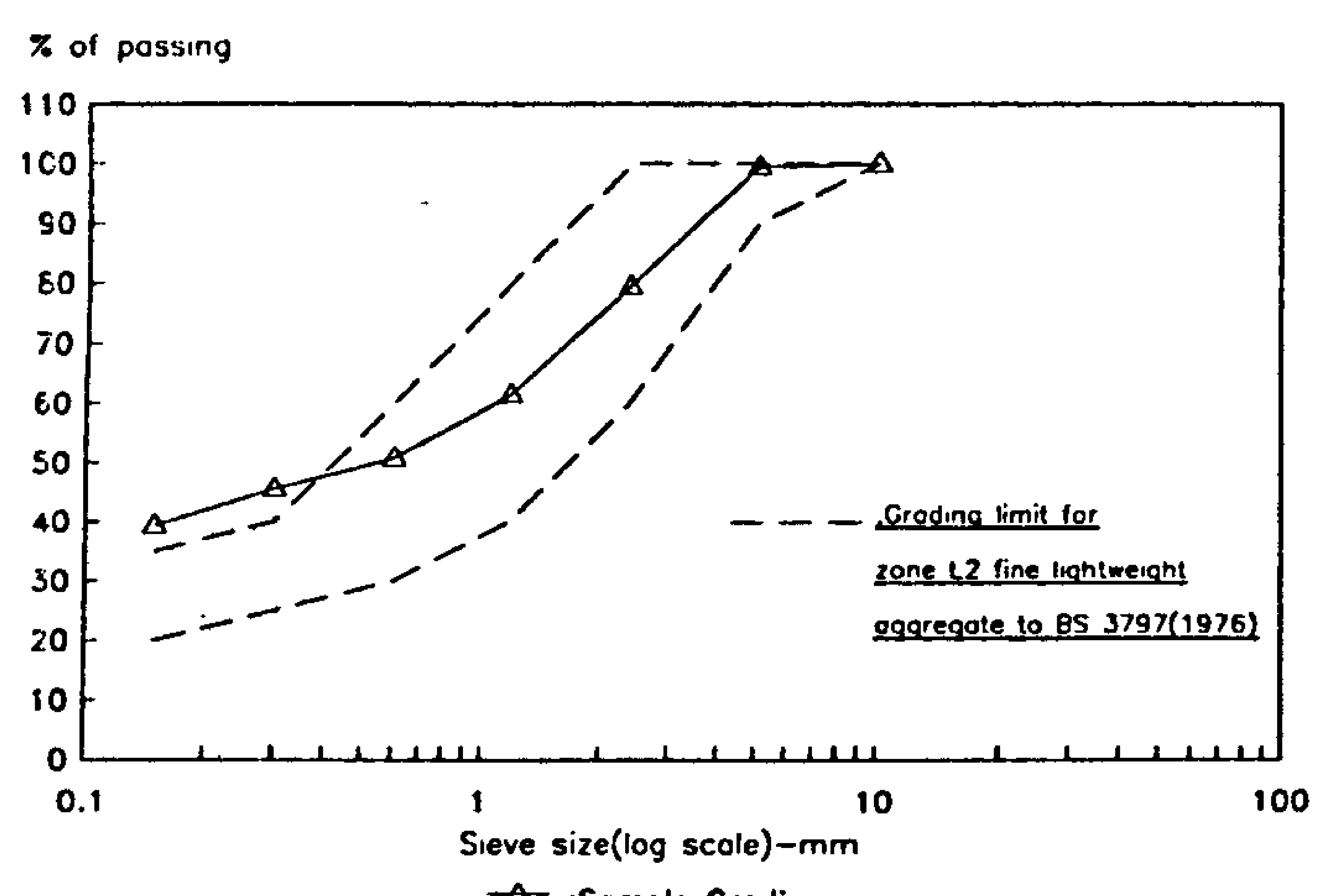


Figure 4.7:Typical grading curve for fine Lytag

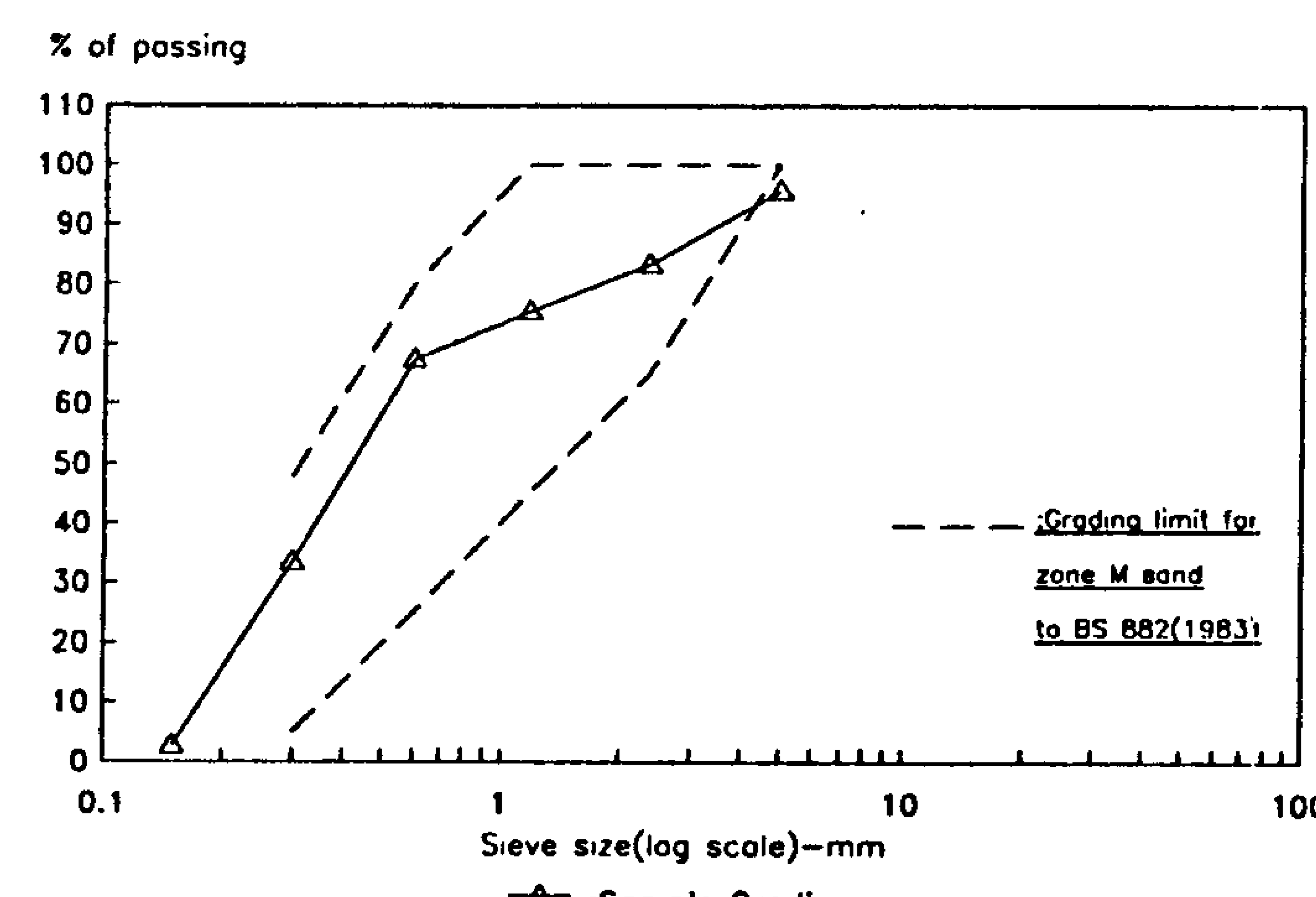


Figure 4.9:Typical grading curve for North-Notts sand

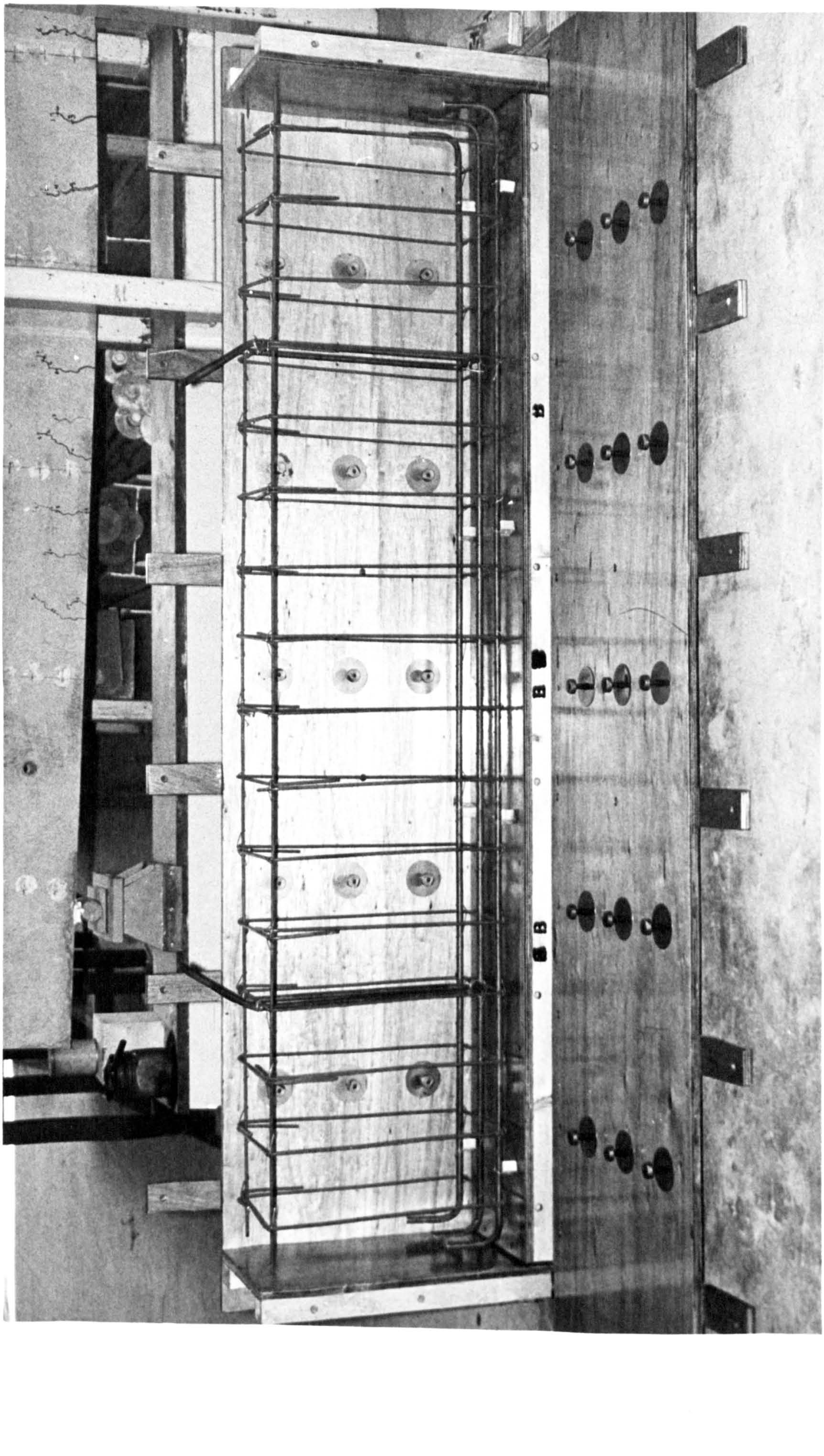


Figure 4.10: Beam mould with the arrangement of reinforcement and Lok inserts

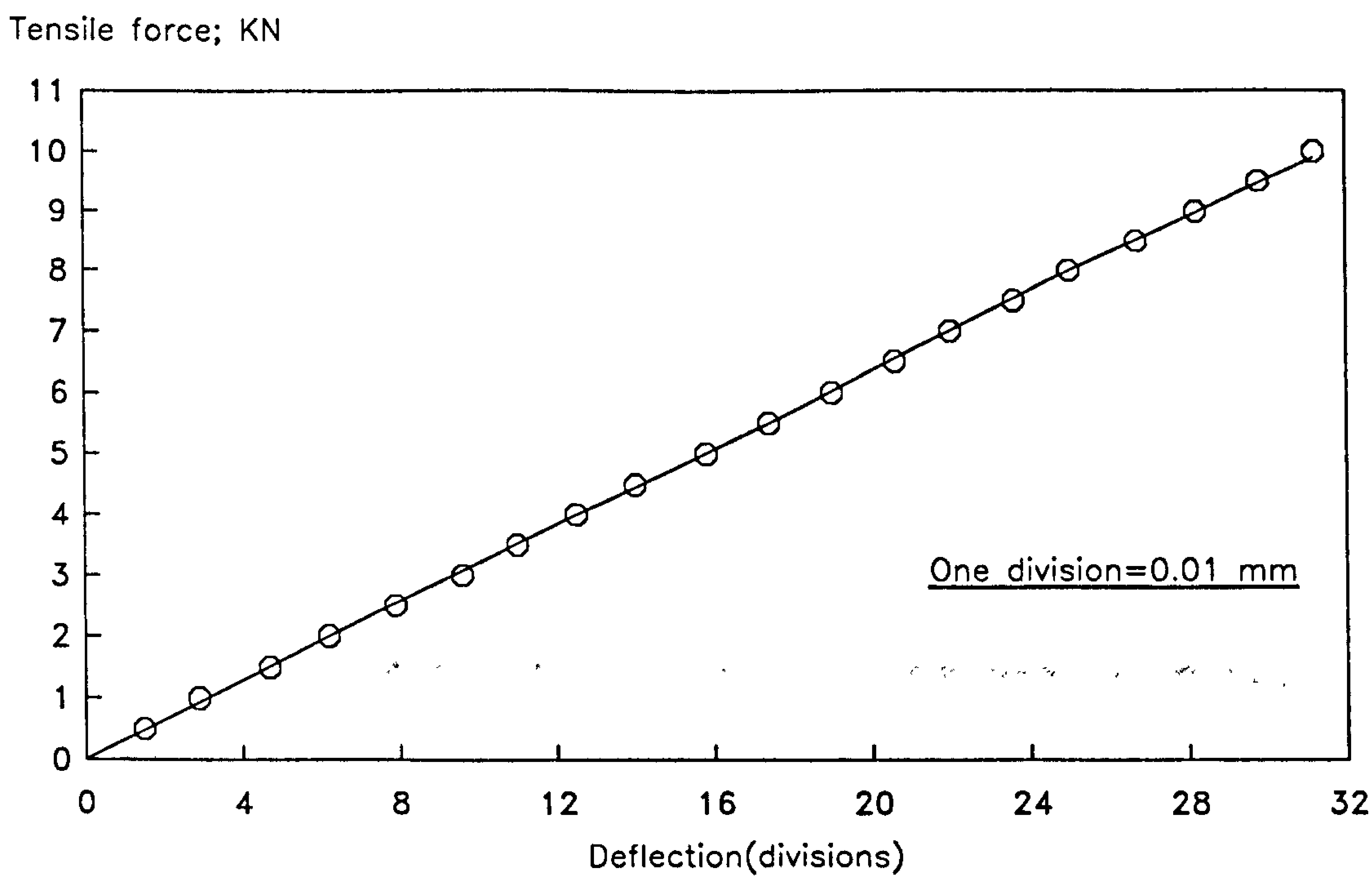


Figure 4.11:Calibration of proving ring.

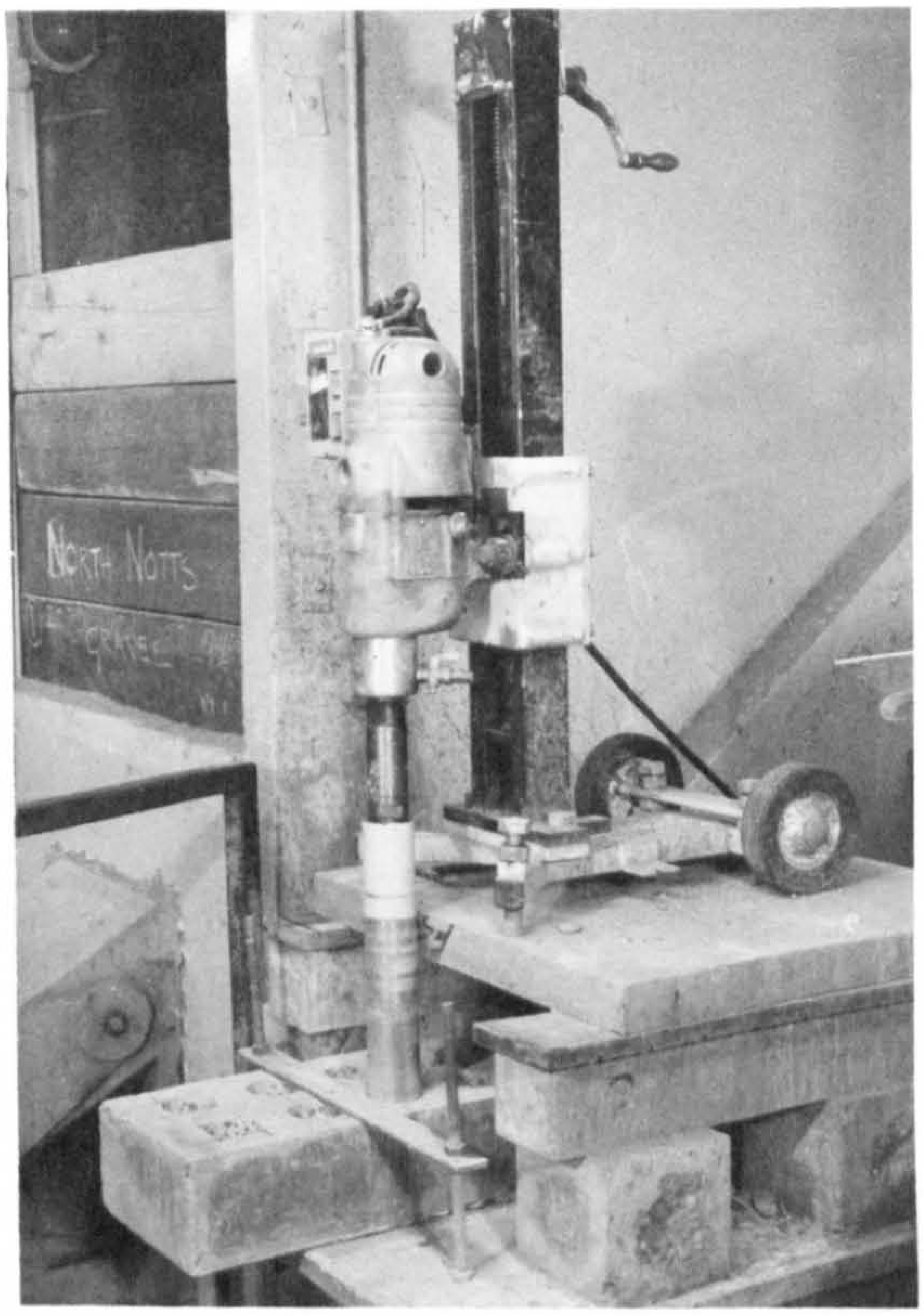


Figure 4.12: Diamond-tipped core cutter

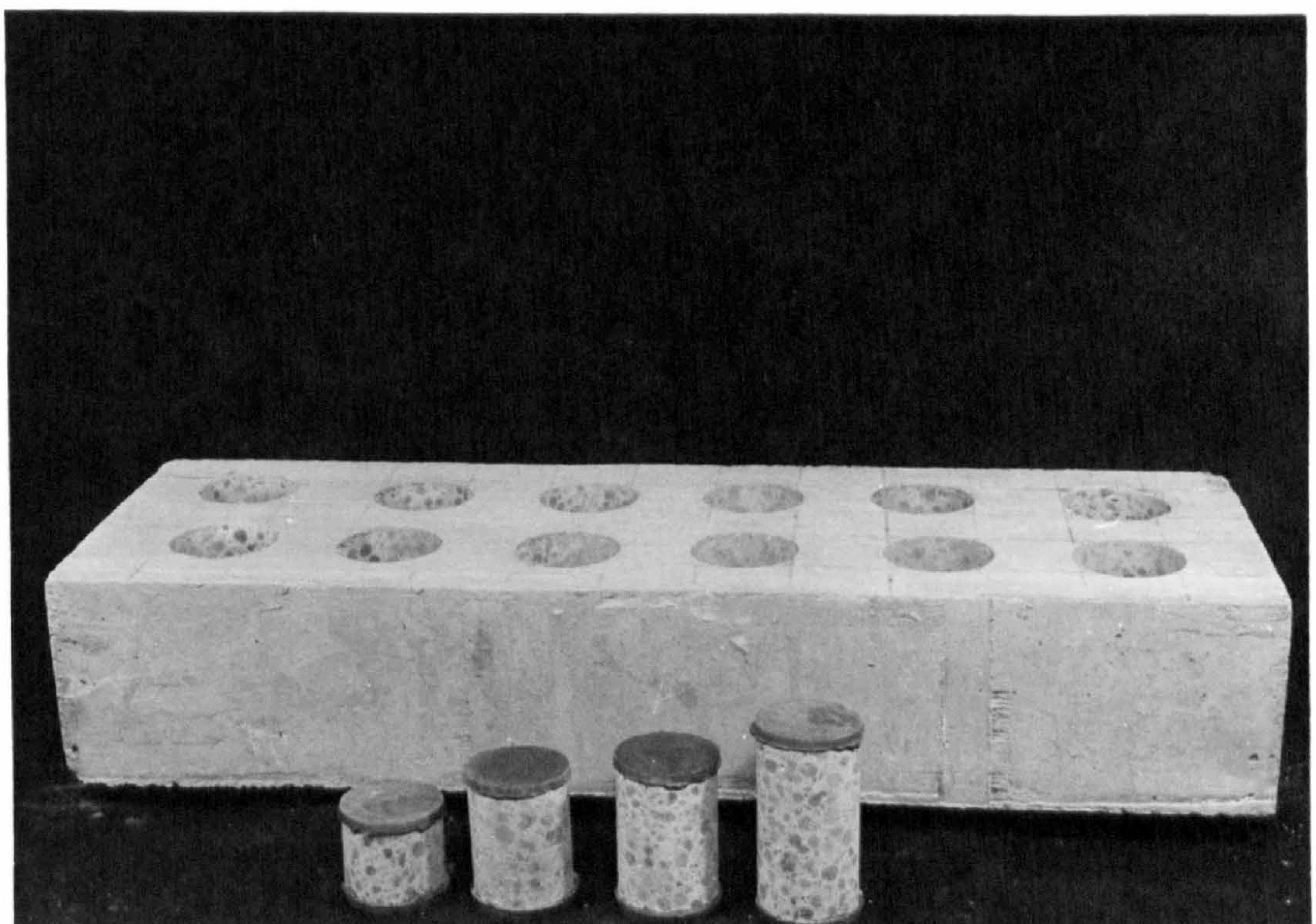


Figure 4.13: 50mm core specimens as drilled vertically from beam

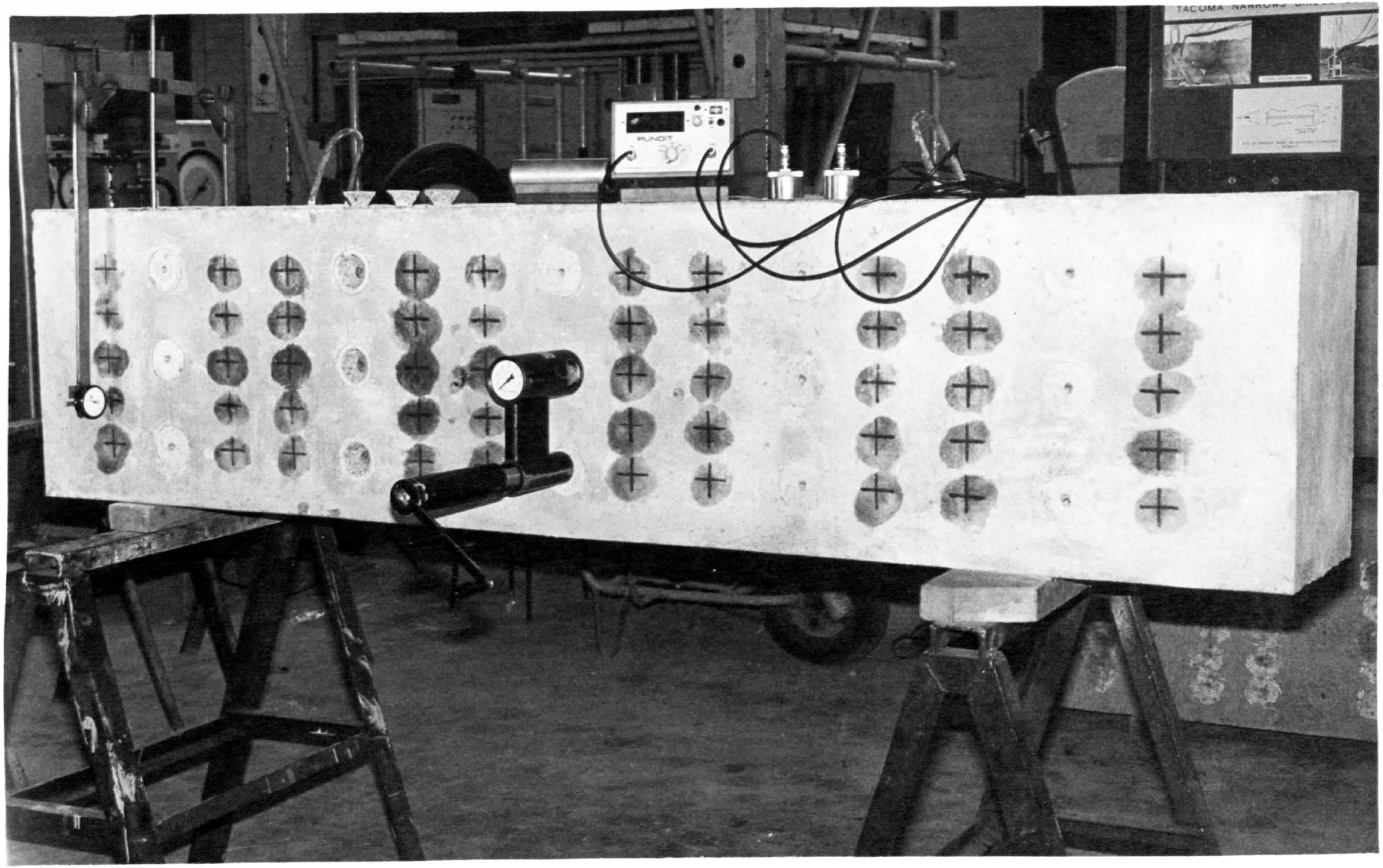


Figure 4.14: Large scale beam under test

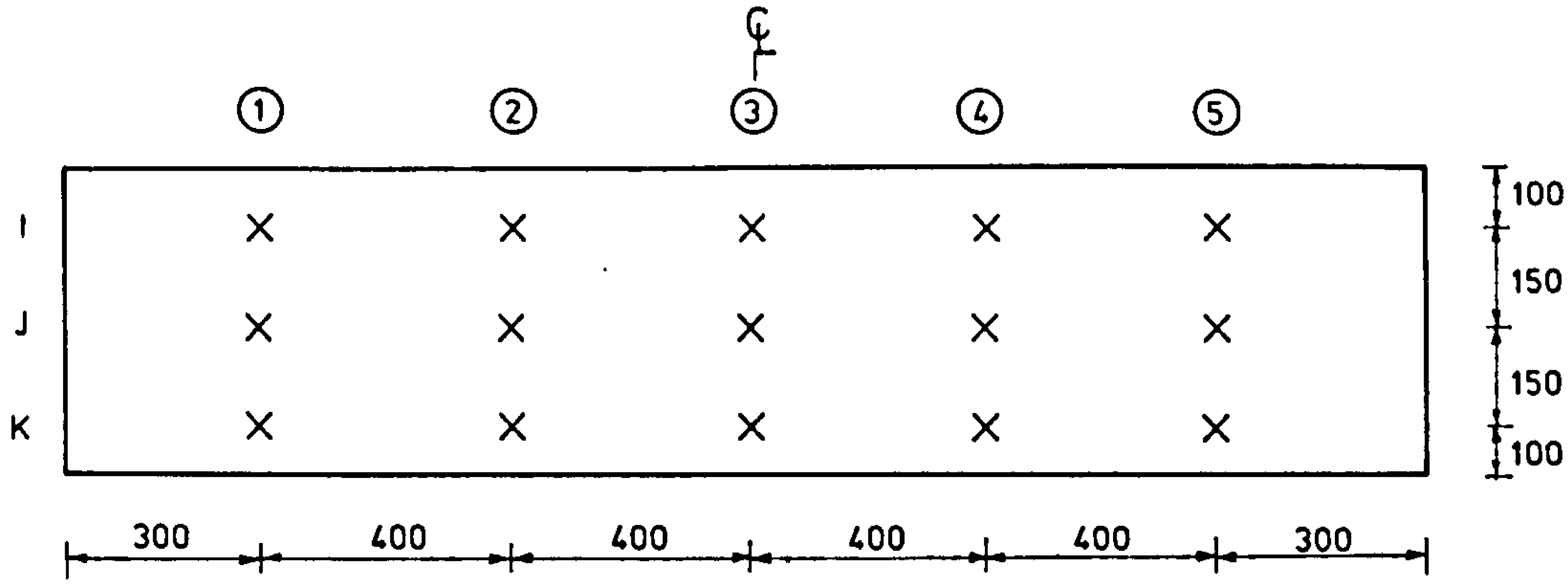


Figure 4.15: Position of Lok inserts for different types of concrete beam tested at 28 days DIMENSIONS IN mm

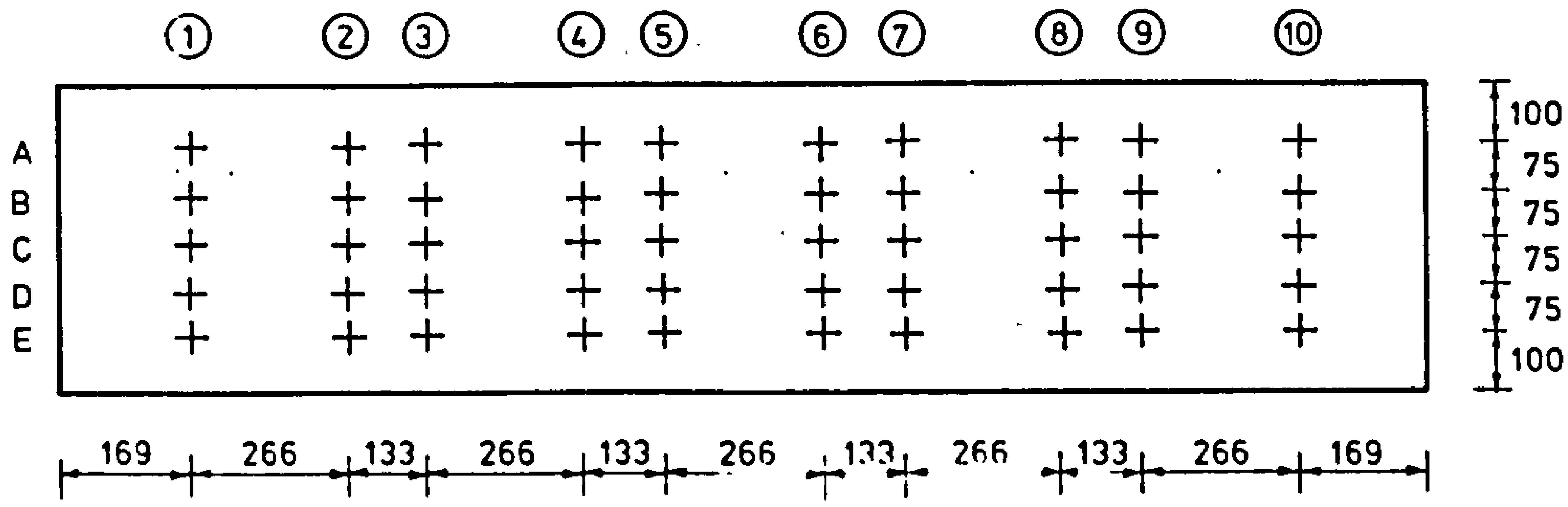


Figure 4.16: Position of pulse velocity tests for different types of concrete beam tested at 28 days DIMENSIONS IN mm

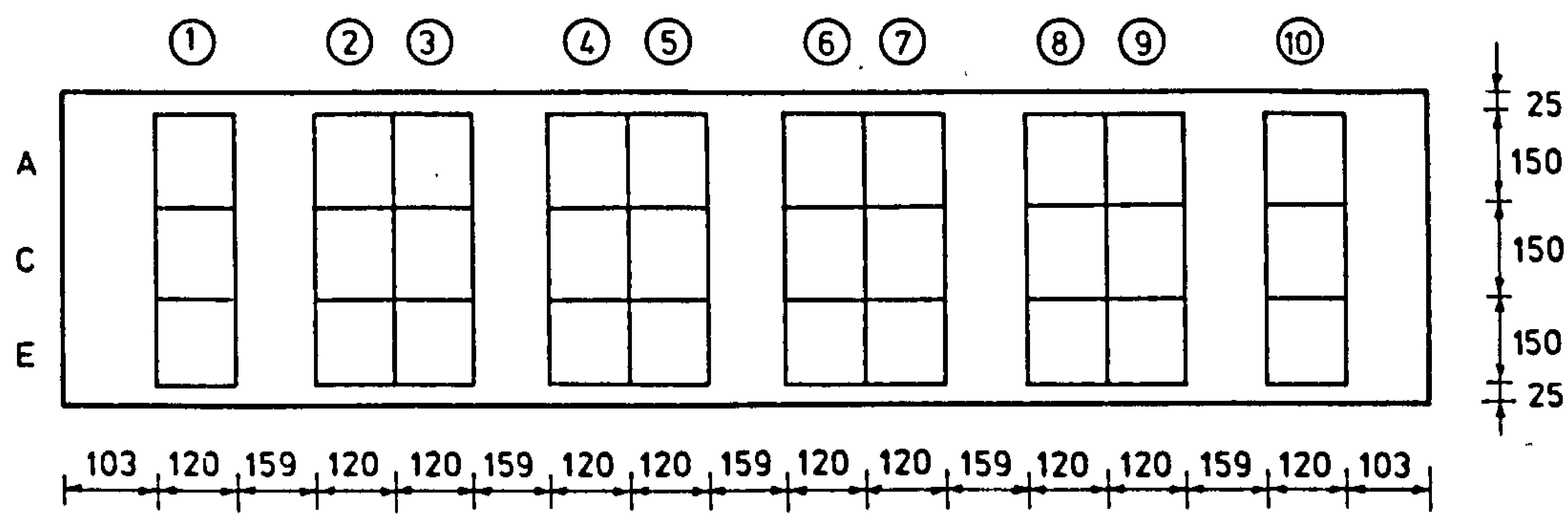
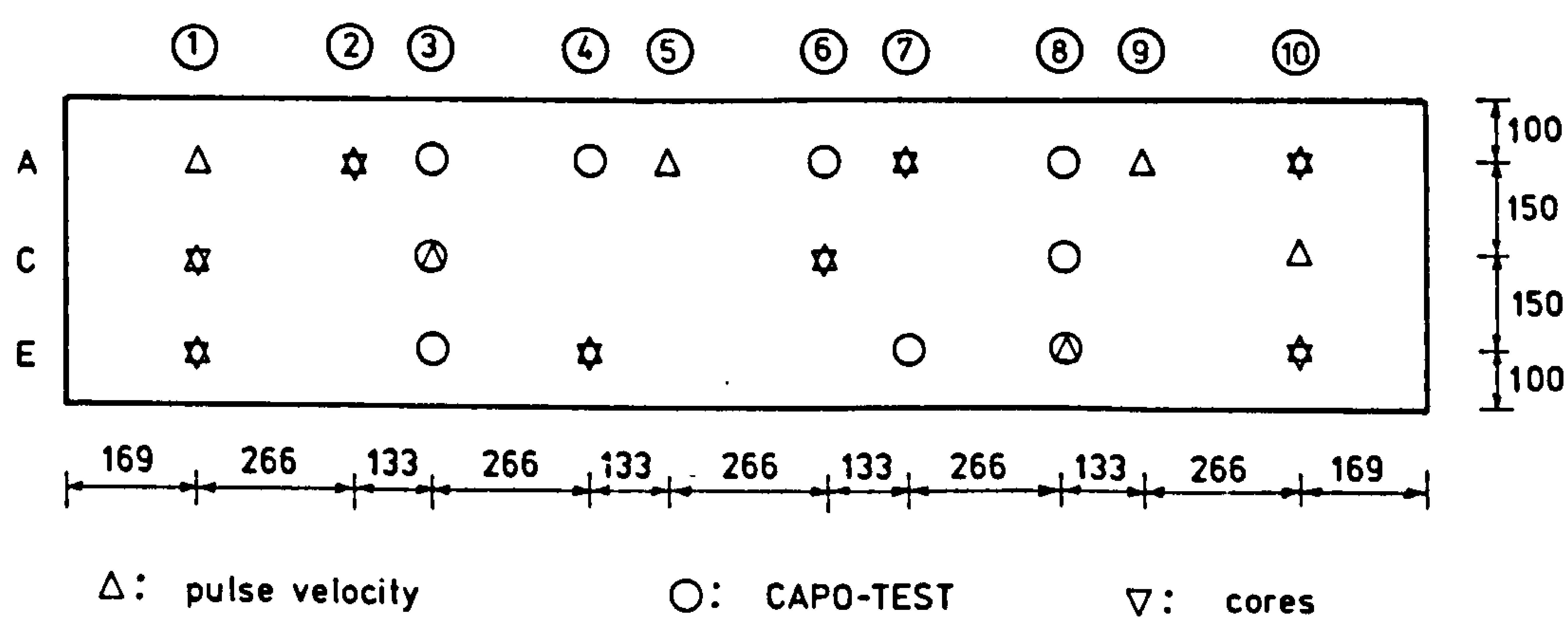


Figure 4.17: Grid layout of rebound hammer tests for different types of concrete beam tested at 28 days DIMENSIONS IN mm



DIMENSIONS IN mm

Figure 4.18: Arrangement of test positions for fully Lytag concrete beam tested at 6 months

CHAPTER 5. MEASURED PROPERTIES OF LIGHTWEIGHT CONCRETE

5.1 INTRODUCTION

The series of tests described in this chapter were designed to augment available information about some basic physical properties of available lightweight concretes. The properties investigated were the compressive strength, splitting tensile strength and elastic moduli (static and dynamic) at various ages under different curing conditions. The effects of sand replacement for fine lightweight aggregate were also considered.

It has been shown in Section 2.3.2 that different types of aggregate often result in very different physical characteristics of concrete. A knowledge of these influences is therefore important to permit full understanding of in-situ test results.

5.2 COMPRESSIVE STRENGTH

It is well known that the compressive strength is very much affected by the same factors as in normal weight concrete. The compressive strength of fully compacted lightweight concrete is related to the cement content and to the total water/cement ratio. The different lightweight aggregates can be arranged in the following order of increasing cement content as shown in Table 4.3 to produce similar compressive strengths for a given workability; Lytag, Pellite and Leca. The lightweight aggregates used throughout this investigation have shown a wide range of water absorptions as tabulated in Table 4.1 and for direct comparison of relationships between water/cement ratio and compressive strength, an estimated free water/cement ratio based on half hour water absorption was considered. These relationships are shown in Figure 5.1 for different types of lightweight concretes. It can be observed from this figure that there is an individual relationship for each aggregate between the strength and (the estimated free) water/cement ratio. This confirms the significance of aggregate strength on overall concrete strength although not in the same proportion as has been observed by Teychenné, (1967). Figure 5.1 indicates that semi Lytag concrete and Leca concrete require higher and lower water/cement ratios respectively than other lightweight concretes to produce the same compressive strength. In the case of Leca there is, however, a significantly lower upper limit to the strength attainable.

The significancy of sand replacement for fine lightweight aggregate on concrete made with Lytag showed that for a given water/cement ratio, a higher compressive strength results for semi Lytag concrete (see Figure 5.1) which is in line with work carried out by Dhir et al (1989).

5.2.1 Effect of Curing Conditions and Age on Compressive Strength

Two curing regimes were used as described in Section 4.5. The effect of these and age on the compressive strength of different types of concretes based on averages of three tests are shown in Table 5.1 and Appendix A (Tables A-1 and A-2) and Figures 5.2-5.8. Table 5.1 shows the development of compressive strengths under wet and dry curing conditions for low and high strength levels in the short term. It indicates that the effect of the curing conditions on different types of lightweight concretes are different from each other. Compressive strength development in Lytag concretes showed high dependency on curing condition and this is more pronounced at low strength level, wherease in Pellite concrete no significant difference between wet and dry cured compressive strength has been detected. In Lytag and Leca concretes the dry cured compressive strength is generally higher than the wet cured, especially for fully Lytag concrete at 28 days. It must be remembered however that the wet cured specimens were tested wet and the dry cured tested dry, and it is generally accepted that specimens tested dry may be up to 15% stronger than those tested wet. This is unlikely to account for the full difference observed here. Higher strengths for dry cured concrete may also be explained by the high water absorption of lightweight aggregates. Presumably, it will allow dry cured specimens to have a sufficient reservoir of absorbed water for the hydration process to go along at the same rate as wet cured concrete, but higher internal friction between the particles will allow a higher compressive strength compared to wet where the moisture may have a lubricating effect which allows the particles to slip by each other in shear more easily. However in Leca concrete at high strength level a clear conclusion on the benefits of dry curing cannot be made due to the high scatter of test result variability as will be discussed later in this chapter. The effect of curing condition on strength for the different aggregate types can be seen by reference to Table 4.1 to be related to their particular absorption values.

Densities are also shown in Table 5.1. The density of properly compacted concrete is primarily affected by aggregate density (see Section 2.2.2.1) which in turn is related to particle porosity. The lightweight concretes used in this investigation can be arranged in order of increasing density from Leca concrete to Pellite concrete. It is also evident from Table 5.1 that the sand replacement for fine lightweight aggregate increases the density of Lytag concrete by an average of approximately 12% which is within the range of 10-20% given by Hanson (1964).

Compressive strength development was examined up to an age of one year. Short term values from 1 to 28 days were considered at eight different ages with long term values at 6 months and 1 year for some mixes. Typical short term strength development for different types of concrete under dry curing (similar to large scale beam curing) is shown in Table A-1 and Figure 5.2. It can be seen that for any type of concrete the cube compressive strength increases rapidly during the first few days after which the gain in strength is at a relatively slow rate. At the age of 3 days, approximately 50% of 28 day compressive strength was developed except for Leca concrete where this percentage of strength was gained at one day. This fast rate of strength gain in Leca concrete can be explained by the higher cement demand. It can also be seen from Figure 5.2 that Leca concrete almost reached the ceiling strength in two weeks. Among the different types of concretes, the fully Lytag concrete showed steadiest rate of increase in strength development. This may again be explained by higher water demand due to absorption by coarse and fine aggregates which provides a reservoir of free water available during curing to assist the hydration of cement. It is interesting to note that the strengths developed at 28 days are generally higher than those for dry curing by covering the specimen only for 3 days under polythene (Tables 5.1 and A-1,2) where it may cause a relatively higher water movement from the paste which affects the reaction of hydration.

Tests for strength assessment are often made at the age of 28 days. However as can be seen from Figures 5.3-5.8 the gain in compressive strength beyond the age of 28 days is significant, especially for wet curing conditions. Three different types of concrete, namely fully Lytag, Leca and normal weight concretes were examined for long term strength development. The largest increases above the 28 day values were obtained with fully Lytag concrete. Figures 5.3-5.6 suggest that in lightweight concretes the stronger the concrete the lower is its proportionate gain of strength after 28 days. Similar effects were reported by other investigators using different types of lightweight concretes (Teychenné, 1967), (Bandyopadhyay, 1974), (Balendran, 1980), (Swamy and Lambert, 1983), (Mayfield and Louati, 1990). Similar behaviour was also observed for normal weight concrete up to 6 months whereas at 1 year slightly higher strength was gained by richer mixes. For lightweight concretes, it is clearly shown that the long term strength development under dry curing, as compared to wet curing, is the opposite to that occuring in the short term. In normal weight concrete similar behaviour has been observed in short and long terms strength development with the exception of low strength mix (mix No. N-1), at 28 days where the dry cured compressive strength was higher than the wet cured. The increase in compressive strength for fully Lytag concrete from 28 days to 1 year ranged from 11 - 23% for dry curing whereas the corresponding increase for wet curing ranged from 41 - 116%. Figure 5.3 shows that the wet compressive strength for mixes L-A, B, C are increased linearly with the logarithm of age from 28 days onwards. Figure 5.3 also shows that the rate of increasing in compressive strength for weaker mix (L-A) is reasonably higher than the short term strength development. For Leca concrete the percentage of increase in compressive strength after 28 days up to 1 year varied between 1 and 13% under dry curing. Figure 5.5 shows that the rate of wet compressive strength development for mixes Le-2, 3 are nearly the same for short and long terms. In normal weight concrete similar strength development was detected as in Leca concrete.

Comparison of strength development at 6 months and one year for fully Lytag and normal weight concretes under wet curing condition showed that the increase in strength from 28 days to 6 months ranged from 40-85% and 5-22% for fully Lytag and normal weight concretes respectively. The corresponding increases at one year were 41-116% and 10-24%. These indicate that the growth of the normal weight concretes was substantially complete after 6 months, whereas a considerable increase over the 6 months took place with fully Lytag concrete. Similar observation has also been reported by Westley et al (1966). Evans and Paterson (1967) suggested that the improved compressive strength growth characteristic of fully Lytag concrete is due to the reaction between the silica in the aggregate and the free lime in cement which causes a more rapid strength development. However, the strength growth for both latter types of concrete under dry curing condition as indicated in Figures 5.4 and 5.8 was small and it can be seen that in some cases there is a loss in compressive strength after 6 months. Similar effects have been reported by other investigators (Price, 1951), (Balendran, 1980), (Swamy and Lambert, 1983) and it is suggested that the slow retrogression of strength may be associated with shrinkage-induced microcracks.

Long term compressive strength development for semi Lytag and Pellite concretes have not been examined here but are reported by other researchers. Under wet curing, Swamy and Lambert (1983) reported that the increase in strength after 28 days up to one year for semi Lytag concrete was 10-40% whereas the corresponding strength development for Pellite concrete was shown to be between 10-26% by Mayfield and Louati (1990). Strength gain as reported is thus relatively greater for semi Lytag concrete than Pellite concrete and they are comparatively lower in strength development than the fully Lytag concrete. This may somehow be related to higher water absorption and possible reaction between silica content in Lytag with free lime in cement.

Variation analysis on compressive strength measurements using coefficients of variation were carried out on fully Lytag, Leca and normal weight concretes and these are given in Table A-2. Although due to limited number of observations (3 in each case) a definite conclusion cannot be given the analyses generally indicates two main things. Among these three different types of concrete, the Leca concrete showed particularly high variability. This is likely to be related to aggregate characteristics and their distribution in the matrix. Also different types of concrete usually result in a high scatter for long term strength development under dry curing whereas under wet curing no significant differences on scatter of results were observed between short term and long term development.

5.3 TENSILE SPLITTING STRENGTH

Tensile strength as one of the basic properties of concrete was determined using the Brazilian split-cylinder test. Results of splitting tensile strength along with compressive strength for different types of concrete employing various mixes under different curing conditions are presented in Table A-3. The strengths quoted are the mean results obtained from three tests. An indication of the consistency and reliability of the results is given by considering the value of coefficient of variation which is relatively higher for tensile splitting strength as compared to compressive strength. The average value of tensile splitting and compressive strength variations for all types of lightweight concretes except the Leca concrete was found to be around 6% and 3.5% respectively. In Leca and normal weight concretes, the tensile splitting variations were obtained to be 8.5% and 8% whereas the corresponding variation in compressive strength was 7% for Leca concrete and 3.5% for normal weight concrete. Once again here, the Leca concrete showed the higher scatter of variation among the different types of concrete.

5.3.1 Relationship between Tensile and Compressive Strengths

The test results for the tensile splitting strength have been plotted against 100mm cube compressive strength in Figures 5.9 to 5.13 for different types of concrete under wet and dry curing conditions. Short term and long term effects on the correlation were considered on two types of concrete, fully Lytag and normal weight, whilst for the remaining types of concrete only short term effects at 28 days were studied.

The relationship between tensile splitting strength and compressive strength is generally expressed in the form

$$f_t = a\ f_c^{\ b} \qquad (5.1)$$

where f_t is the tensile splitting strength in N/mm^2

f_c is the 100mm cube compressive strength

a and b are constants.

The constants a, b are determined using regression analysis (Kennedy and Neville, 1976). Coefficient of correlation is also determined which permits the examination of the accuracy with which the mathematical model fits the laboratory data. The regression analyses for different types of concrete are tabulated in Table 5.2.

Figures 5.9 to 5.12 show that for a given compressive strength there is a distinct difference between the tensile splitting strength of wet cured specimens and that of dry cured specimens and this is more pronounced at high strength levels.

With the exception of semi Lytag concrete at low strength level, the tensile splitting strengths obtained under dry curing conditions were lower than those obtained from wet curing. The reduction of splitting tensile strength under dry conditions is more significant with fully Lytag and Leca concretes. Figures 5.9 and 5.11 indicate that for the two latter types of concrete, at their high strength level under dry curing conditions no significant changes in splitting strengths were observed with increasing compressive strength. Furthermore, long term tensile splitting strength of fully Lytag concrete is shown to be less dependent on curing conditions than in the short term. Figure 5.9 shows that under dry curing at 1 year a relatively higher tensile splitting strength results for a given compressive strength than in the short term. This reduction in tensile splitting strength under dry curing and subsequent recovery at later age has been observed by other investigators including Teychenné (1967), Bandyopadhyay (1974), Swamy and Lambert (1983). The phenomenon may be explained by differential moisture distribution throughout the test specimen. This differential moisture content causes internal stress conditions which result in a lower tensile splitting strength. However at later ages, the strength is regained as the specimens become uniformly dry and the self-induced stress is relieved.

Table 5.3 gives the values of the tensile splitting/ compressive strengths ratio as a function of the curing and the age. The values for different types of lightweight concrete were between 0.05 and 0.11 with an average of 0.09 and 0.08 for wet and dry curing respectively. The average ratio obtained for normal weight concrete under dry curing was 0.09 which is greater than that obtained for lightweight concretes (Figure 5.14). A similar trend has also been reported elsewhere (Neville, 1988). Figures 5.14 and 5.15 present the tensile splitting/compressive strengths relationship for different types of concrete and it is clearly shown that under wet curing the scatter of correlations are lower than for dry curing. The ratio generally decreases with time, indicating that the two strengths were increasing at different rates. The rate of increase of compressive strength was higher than that of tensile splitting strength. This behaviour seems to be similar for both lightweight and normal weight concretes, and is in agreement with the work reported by other researchers (Saul, 1960), (Mayfield and Louati, 1990).

The effect of sand replacement for fine lightweight aggregates on tensile splitting strength was examined on Lytag concretes. The improved tensile splitting strengths are shown in Figures 5.14 and 5.15. These indicate that the significance of sand replacement is greatest under dry curing conditions. This may possibly be due to lower differential moisture content in semi Lytag concrete as well as the direct contribution of the sand combining to give much higher dry cured tensile splitting strength than fully Lytag concrete. The significance of sand replacement on improved tensile splitting strength of lightweight concretes has also been observed by Pfeifer (1967) and Teychenné (1967), (see Section 2.3.2.2.). Results obtained in this investigation are shown by Figure 5.16 to generally lie within a band established by Teychenné (1967) and confirmed by Ibrahim (1972) and Bandypadhyay (1974) for a wide range of lightweight aggregates.

5.4 MODULUS OF ELASTICITY

The moduli of elasticity of lightweight and normal weight concretes were determined using the dynamic and static test methods. The test results of dynamic modulus values at different ages from 1 day to 1 year along with static modulus at 28 days are presented in Table A-4 based on the average of three readings. The dynamic modulus of elasticity was found to be about 25% on average higher than the static modulus from the results obtained on three types of lightweight concrete, fully Lytag, Leca and Pellite concretes. Further discussion on the relation between dynamic and static modulus of elasticity will be given in Section 5.4.2. The development of modulus of elasticity with age under wet and dry curing was monitored by measuring the dynamic modulus since measurements are easier to make than static values and are non-destructive.

5.4.1 Development of Dynamic Modulus with Age

The graphs given in Figures 5.17 to 5.19 show that the dynamic modulus increases with age from one day to one year with the majority of the increase taking place during the first few days. As the age increases, the rate of gain of dynamic modulus becomes progressively less and comparatively much lower than the long term compressive strength development. For example, for fully Lytag concrete at low water/cement ratio (L-D), the dynamic modulus of elasticity was increased by 12% from 28 days to 1 year under wet curing condition as opposed to 41% increase in compressive strength under similar conditions. The corresponding increase in dynamic modulus of elasticity on Leca and Pellite concrete was 9% and 4.5% respectively. In general, the increase with age is more pronounced in wet cured specimens. The dry cured specimens resulted in a loss in dynamic modulus at ages after 28 days. This loss was determined for three types of lightweight concrete; fully Lytag, Leca and Pellite concretes and it was up to 10%. This reduction in the modulus of elasticity has been observed by other investigators Swamy and Bandyopadhyay (1975), Louati (1988). At the age of 28 days, the influence of curing condition on dynamic modulus seems to be higher for Pellite concrete and this is reflected in Figure 5.17 to 5.19. At this age, the difference between wet and dry curing conditions were found to be 6% for fully Lytag concrete, 11% on average for Leca concrete and about 15% on average for Pellite concrete. The small change in dynamic modulus of fully Lytag concrete may be attributed to use of fully lightweight aggregate with high water absorption which may provide internal moisture for dry curing. At one year a considerable change in the dynamic modulus was found for all types of lightweight concretes, however in the short term because of a more uniform moisture content throughout the specimen the differences in dynamic modulus with respect to curing conditions were relatively small for different types of lightweight concrete.

The effect of differing water/cement ratio upon dynamic modulus development with age was examined for Leca and Pellite concretes. This is illustrated by Figures 5.18 and 5.19 which show that in Leca concrete, the form of the dynamic modulus-age relationship is virtually unaffected while only a small difference is seen in Pellite concrete.

5.4.2 Relationship between Dynamic and Static Modulus of Elasticity

The advantages of estimation of static modulus from measurement of dynamic modulus have been indicated in Sections 5.4 and 5.4.1. For normal weight concrete BS 8110 : part 2 (1985) suggests the relationship

$$E_s = 1.25\ E_d - 19 \tag{5.2}$$

where E_s = static modulus; kN/mm^2

E_d = dynamic modulus; kN/mm^2

However the above relation does not apply to lightweight concrete. Swamy and Bandyopadhyay (1975) suggest that for lightweight concrete the relationship is

$$E_s = 1.04\ E_d - 4.11 \tag{5.3}$$

This relationship is applicable to all lightweight concretes used in U.K. and the estimated value of the static modulus using equation 5.3 has been claimed to be within $\pm$ $4kN/mm^2$. The applicability of equation 5.3 has been checked by the Author using the test results obtained on fully Lytag, Leca and Pellite concretes along with published data on semi Lytag concrete (Lambert, 1982) as shown in Figure 5.20. This figure confirms the suitability of equation 5.3 which fits the results reasonably well over most of the range with deviations from the straight line relation being relatively small.

5.4.3 Relationship between Modulus of Elasticity and Compressive Strength

The modulus of elasticity of concrete is mainly related to the compressive strength, the modulus of elasticity of the aggregate and the density. For each type of concrete a relationship between modulus of elasticity and compressive strength may thus exist. For this investigation, the correlation between dynamic modulus of elasticity and compressive strength was established using a power curve relationship similar to equation 5.1. This correlation is illustrated in Figures 5.21 to 5.25 for various types of concretes. Regression analyses were carried out for all types of concrete for different curing conditions and they are summarized in Table 5.4.

The effects of mix proportions and curing conditions were studied on the correlation for fully Lytag, Leca and Pellite concretes and the influences of these factors are shown graphically in Figures 5.21 to 5.24 for the latter two types of concrete. In Figures 5.21 and 5.22 the dynamic modulus is plotted against compressive strength for Leca and Pellite concretes, in which the mix proportions were varied by using two different mixes (low and high water/cement ratio) for each type of concrete. Testing ages from 2 to 28 days are presented on these figures. Both figures clearly define that for either types of concrete, the dynamic modulus/strength relationship alters only very slightly with change of mix proportions, and from this it may be concluded that the dynamic modulus/compressive strength correlation is independent of mix proportion for practical purposes.

In Figures 5.23 and 5.24 the effect of curing conditions on the correlation were examined for Leca and Pellite concretes. For Leca and Pellite concrete two mixes with low and high water/cement ratios were considered. From comparison of the graphs for each mix, it appears likely that separate relationships exist between dynamic modulus and compressive strength, and this dependency is relatively higher at high water/cement ratios. Nevertheless, the regression analysis in Table 5.4 showed that for each type of concrete a single curve can still be fitted through the data points, independent of curing conditions, with a reasonable correlation. It must be borne in mind that the analysis given here is based on short term concrete development which may not be suitable for long term assessment where significant difference was detected between modulus of elasticity and compressive strength development with age.

In view of the influence of aggregate type on the correlation, the test results for dynamic modulus of elasticity of different types of concretes have been plotted against the compressive strength in Figure 5.25. The figure strongly indicates that for each type of concrete a different relationship exists. For a given compressive strength, the highest modulus of elasticity was obtained with normal weight concrete whereas the lowest was obtained with Leca concrete. In the case of a $25 N/mm^2$ compressive strength, the Pellite concrete has a modulus of elasticity of about 70% of the normal weight concrete whereas the

Leca concrete is only about 41% of the normal weight concrete. For fully and semi Lytag concretes, the corresponding values are 46% and 56% respectively. Similar trends have been observed by Shideler (1957) and Teychenné (1967) on some lightweight concretes produced in the U.S. and U.K. as discussed in Chapter 2.

Figure 5.25 also highlights two other important facts. The first phenomenon relates to the sand replacement for fine lightweight aggregate in Lytag concrete. It is clearly shown that the modulus of elasticity improved with sand replacement and this is more pronounced at higher strength levels. The percentage increase in dynamic modulus of elasticity at a normal strength level (30N/mm^2) is about 26% which is within the range given by Hanson (1964) (see Section 2.3.2.3). The second phenomenon is that there is no direct proportionality between compressive strength and modulus of elasticity. The ratio of modulus of elasticity to the compressive strength decreases with an increase in the general level of strength. While studying the effect of age on compressive strength and dynamic modulus of elasticity (Section 5.2.1 and Section 5.4.1) it was observed that after the first few days the modulus of elasticity increases much more slowly than the compressive strength so that the ratio of modulus of elasticity/compressive decreases with time. This is in agreement with the general tendency of the ratio to decrease with an increase in compressive strength.

A limited number of static modulus of elasticity tests were carried out on three types of concretes (see Table A-4). The relationship between static modulus and compressive strength was investigated using the available formulae given by other investigators to correlate these two parameters. Among those, Neville (1988) reported an expression as given below which relates the static modulus of elasticity to the density and compressive strength.

$$E_s = 1.7\ \rho^2 f_c^{0.33} \times 10^{-6} \qquad (5.4)$$

where E_s, ρ and f_c are respectively the static modulus of elasticity in kN/mm^2, the density in kg/m^3 and the compressive strength in N/mm^2.

It has been suggested that the above expression can be used for concretes with density between 1400 and $2300 kg/m^3$ which covers a wide range of available lightweight concretes. The actual static moduli obtained were compared with calculated values obtained from equation 5.4 and are shown in Figure 5.26. It is clearly shown that the points gather around the line of equality which further confirms the applicability of equation 5.4 for estimating the static modulus of different types of lightweight concrete.

Table 5.1 :Effects of Age and curing conditions on the compressive strength

Type of Concrete	Mix Type	Age Days	Density Kg/m^3		cube compressive Strength (N/mm^2)		
			Curing Condition				
			Wet	Dry	Wet	Dry	Wet/Dry
Fully Lytag Concrete	L-A	3	1786	1749	10.5	12.5	0.84
		7	1788	1659	14.7	18.8	0.78
		14	1793	1615	16.8	21.8	0.77
		28	1808	1601	21.2	26.3	0.81
	L-D	3	1833	1764	22.8	23.9	0.95
		7	1837	1725	30.3	32.6	0.93
		14	1838	1723	33.2	41.7	0.80
		28	1842	1704	39.9	46.7	0.85
Semi Lytag Concrete	L-1	3	1982	1895	6.5	8.4	0.77
		7	1985	1832	9.5	13.5	0.70
		14	1987	1828	11.8	15.8	0.75
		28	1997	1823	15.8	17.6	0.90
	L-4	3	1984	1954	29.5	32.6	0.90
		7	1987	1924	35.1	40.2	0.87
		14	2006	1912	41.7	48.1	0.87
		28	2017	1892	47.8	51.3	0.93
Leca Concrete	Le-1	2	1524	1498	5.9	6.3	0.94
		4	1528	1495	7.1	8.0	0.89
		9	1531	1466	9.3	10.7	0.87
		28	1535	1443	12.7	13.1	0.97
	Le-4	2	1625	1637	18.3	19.3	0.95
		4	1661	1634	22.0	21.4	1.03
		9	1665	1628	23.0	24.5	0.94
		28	1668	1615	29.6	25.8	1.15
Pellite Concrete	P-1	3	2064	2022	11.1	11.1	1.00
		7	2070	2007	14.1	14.3	0.99
		14	2072	1992	17.2	17.1	1.01
		28	2075	1981	19.7	19.8	0.99
	P-5	3	2082	2054	27.5	28.6	0.96
		7	2085	2050	35.3	35.9	0.98
		14	2087	2042	40.6	41.0	0.99
		28	2090	2041	44.1	43.4	1.02

Table 5.2:Regression analysis for the relationship between tensile splitting and compressive strengths

Type of Concrete	Age Days	Curing Regime	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	3-28	Wet	$f_t=0.29\ f_c^{0.64}$	0.989
		Dry	$f_t=1.09\ f_c^{0.21}$	0.759
	360	Wet	$f_t=0.90\ f_c^{0.31}$	0.901
		Dry	$f_t=1.06\ f_c^{0.23}$	0.997
	3-360	Wet	$f_t=0.59\ f_c^{0.42}$	0.951
		Dry	$f_t=0.93\ f_c^{0.26}$	0.867
Semi Lytag Concrete	28	Wet	$f_t=0.24\ f_c^{0.70}$	0.992
		Dry	$f_t=0.85\ f_c^{0.32}$	0.931
Leca Concrete	28	Wet	$f_t=0.21\ f_c^{0.74}$	0.980
		Dry	$f_t=0.56\ f_c^{0.35}$	0.990
Pellite Concrete	28	Wet	$f_t=0.24\ f_c^{0.69}$	0.991
		Dry	$f_t=0.32\ f_c^{0.56}$	0.997
Normal Weight Concrete	28	Dry	$f_t=0.59\ f_c^{0.43}$	0.992
	360	Dry	$f_t=0.36\ f_c^{0.56}$	0.968
	28+360	Dry	$f_t=0.47\ f_c^{0.49}$	0.955

Table 5.3:Comparison of splitting/compressive strengths ratio for different types of concretre

Type of Concrete	Mix No.	Age Days	f_t/f_c	
			Wet Cured	Dry Cured
Fully Lytag Concrete	L-A	28 360	0.09 0.06	0.08 0.08
	L-B	28 360	0.09 0.06	0.07 0.06
	L-C	28 360	0.08 0.06	0.06 0.05
	L-D	3 7 28 360	0.09 0.08 0.07 0.05	0.09 0.07 0.05 0.05
Semi Lytag Concrete	L-1	28	0.11	0.11
	L-2	28	0.08	0.07
	L-3	28	0.09	0.07
	L-4	28	0.07	0.06
Leca Concrete	Le-1	28	0.10	0.10
	Le-2	28	0.10	0.09
	Le-3	28	0.09	0.07
	Le-4	28	0.09	0.07
Pellite Concrete	P-1	28	0.10	0.08
	P-2	28	0.08	0.07
	P-5	28	0.08	0.06
	P-6	28	0.07	0.05
Normal Weight Concrete	N-1	28 360	- -	0.10 0.09
	N-2	28 360	- -	0.10 0.09
	N-3	28 360	- -	0.09 0.07
	N-4	28 360	- -	0.07 0.07

Table 5.4: Regression analysis for the relationship between dynamic modulus of elasticity and compressive strength

Type of Concrete	Mix No.	Curing Regime	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	L-D	Wet	$E_d=8.91\ f_c^{0.22}$	0.995
		Dry	$E_d=11.71\ f_c^{0.13}$	0.937
		Wet+Dry	$E_d=11.70\ f_c^{0.14}$	0.747
	All Mixes	Wet+Dry	$E_d=10.40\ f_c^{0.16}$	0.801
Semi Lytag Concrete	L-1	Wet	$E_d=6.14\ f_c^{0.44}$	0.994
	L-4	Wet	$E_d=8.91\ f_c^{0.28}$	0.999
	All Mixes	Wet	$E_d=8.53\ f_c^{0.30}$	0.986
Leca Concrete	Le-1	Wet	$E_d=3.76\ f_c^{0.48}$	0.983
		Dry	$E_d=4.34\ f_c^{0.38}$	0.973
		Wet+Dry	$E_d=4.22\ f_c^{0.41}$	0.920
	Le-4	Wet	$E_d=2.66\ f_c^{0.57}$	0.909
		Dry	$E_d=3.66\ f_c^{0.46}$	0.927
		Wet+Dry	$E_d=2.87\ f_c^{0.54}$	0.894
	Le-1 + Le-4	Wet+Dry	$E_d=4.00\ f_c^{0.44}$	0.980
Pellite Concrete	P-1	Wet	$E_d=8.08\ f_c^{0.41}$	0.993
		Dry	$E_d=12.19\ f_c^{0.23}$	0.962
		Wet+Dry	$E_d=9.83\ f_c^{0.33}$	0.826
	P-5	Wet	$E_d=8.88\ f_c^{0.35}$	0.9997
		Dry	$E_d=17.82\ f_c^{0.14}$	0.934
		Wet+Dry	$E_d=12.40\ f_c^{0.25}$	0.713
	P-1 + P-5	Wet+Dry	$E_d=11.30\ f_c^{0.28}$	0.940
Normal Weight Concrete	All Mixes	Wet	$E_d=23.75\ f_c^{0.16}$	0.967

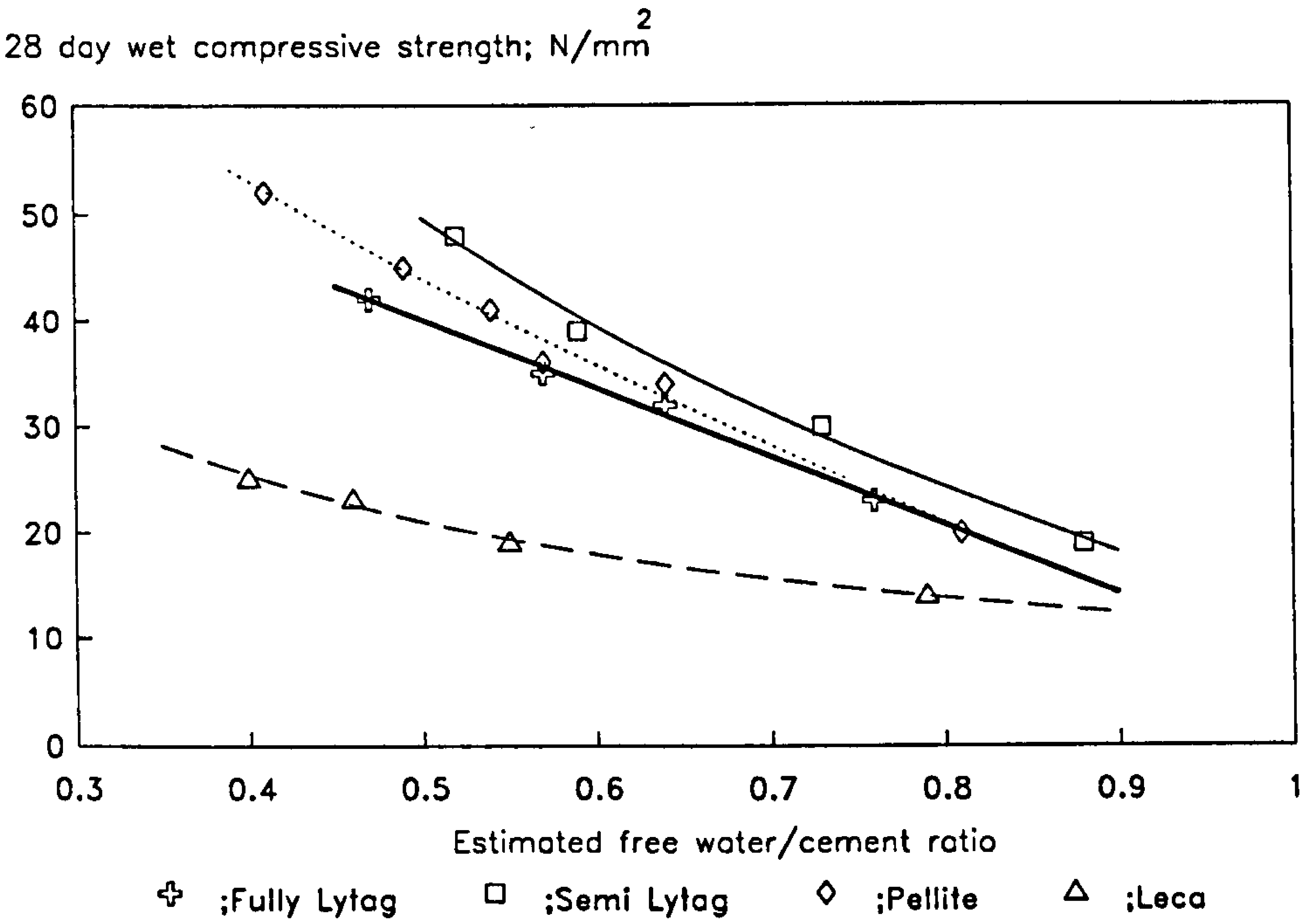


Figure 5.1:Typical relations between 28 day wet cube compressive strength and estimated free water/cement ratio

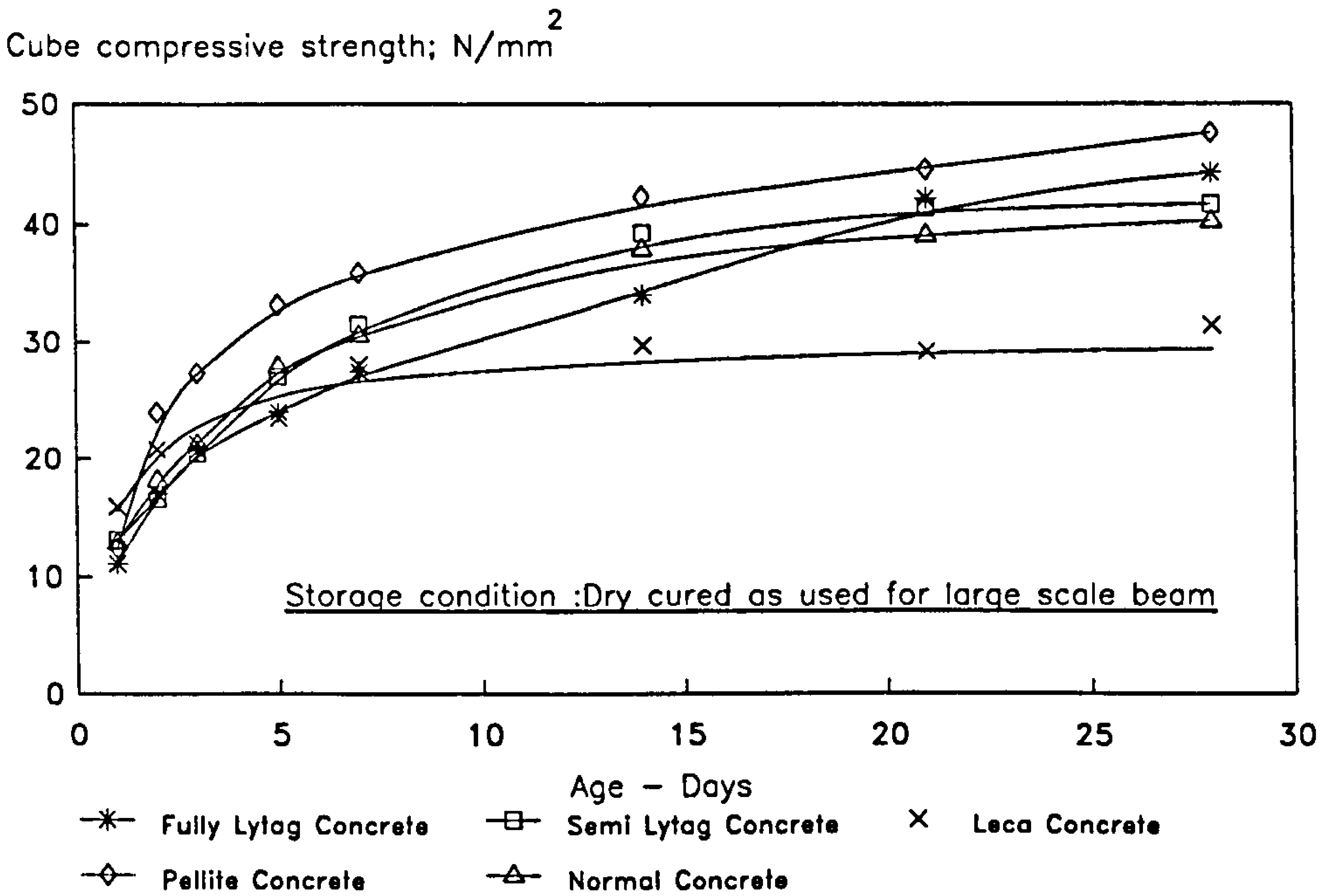


Figure 5.2:Development of compressive strength with age

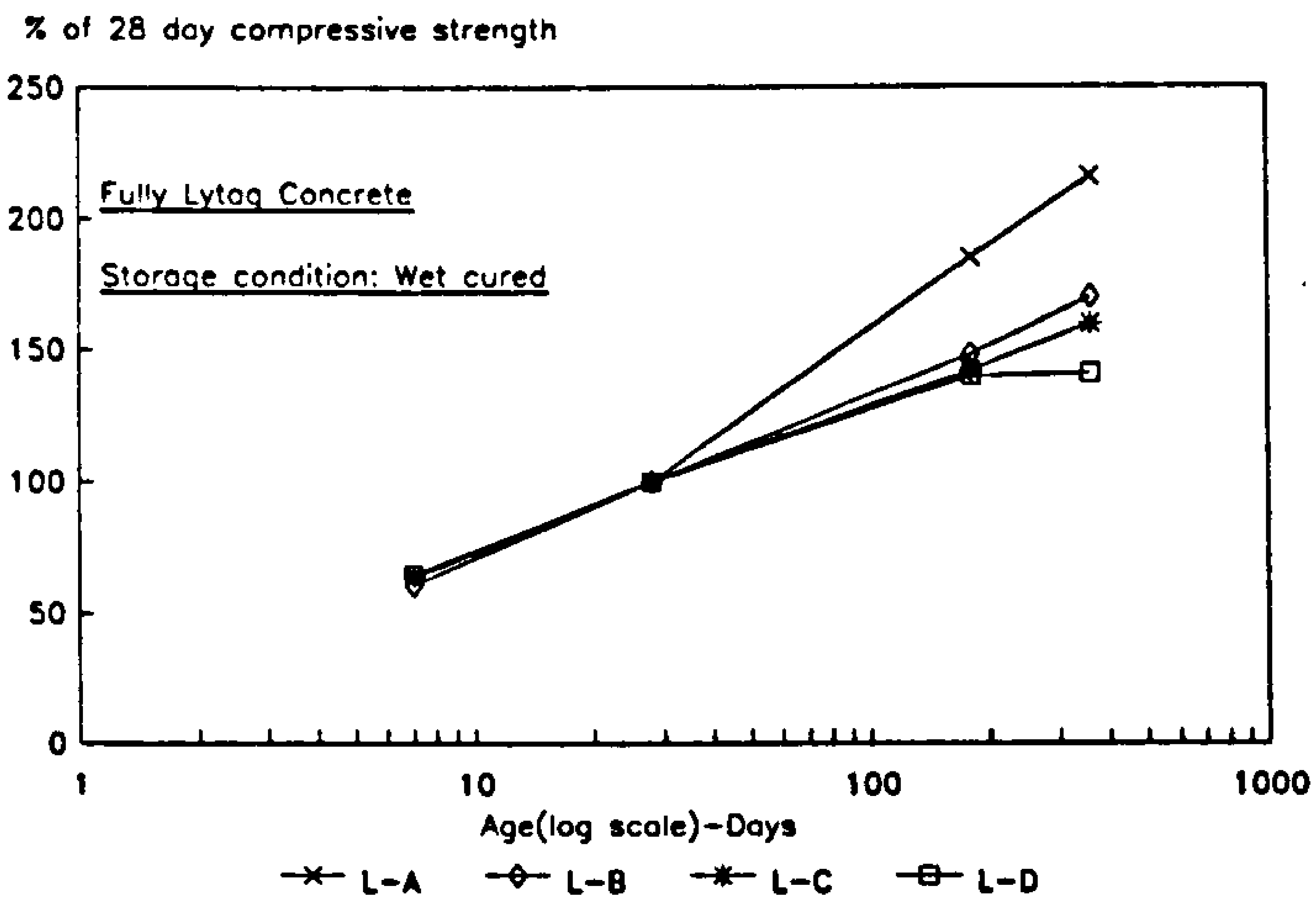


Figure 5.3.Development of compressive strength with age (Lytag-Wet)

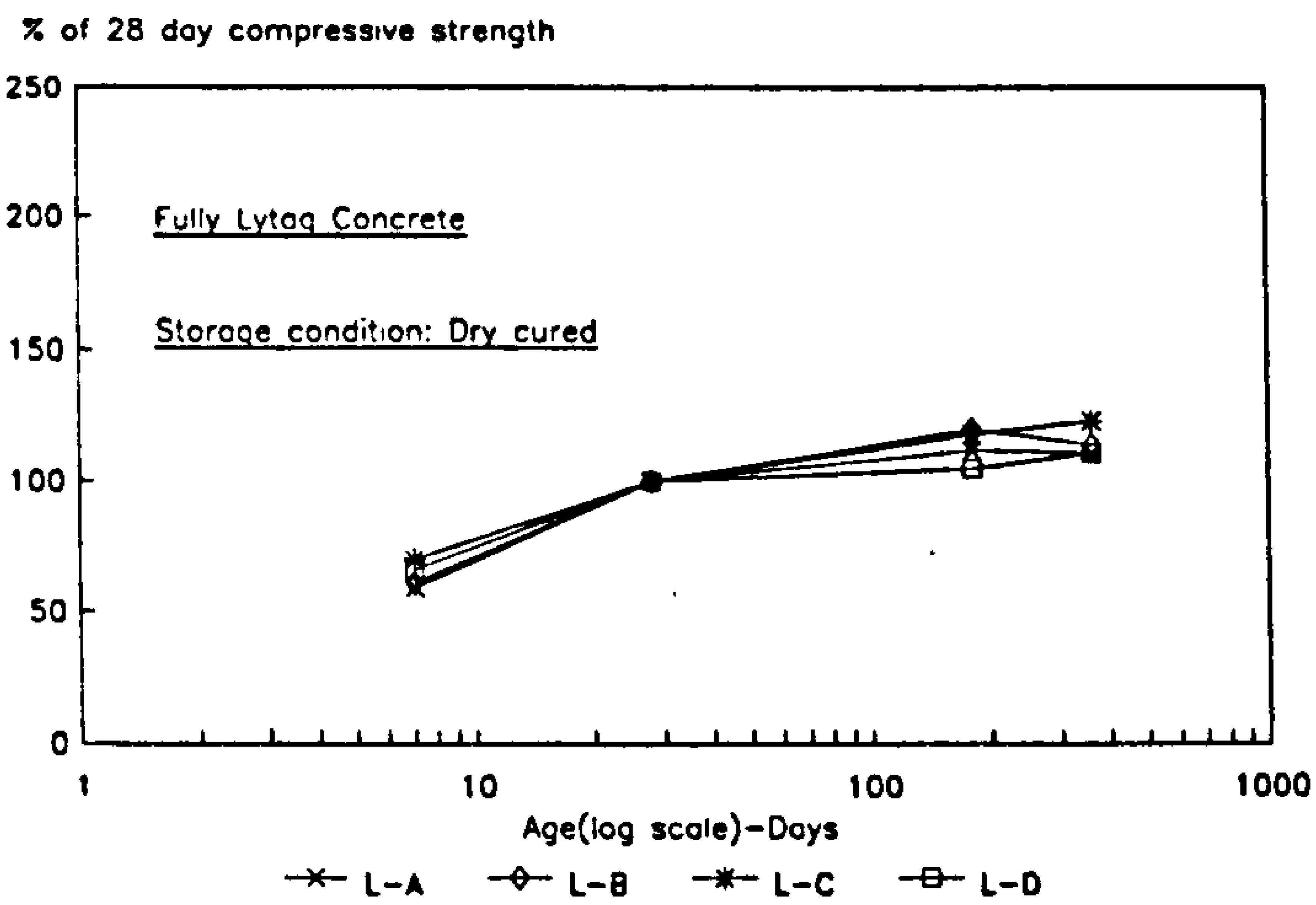


Figure 5.4.Development of compressive strength with age (Lytag-Dry)

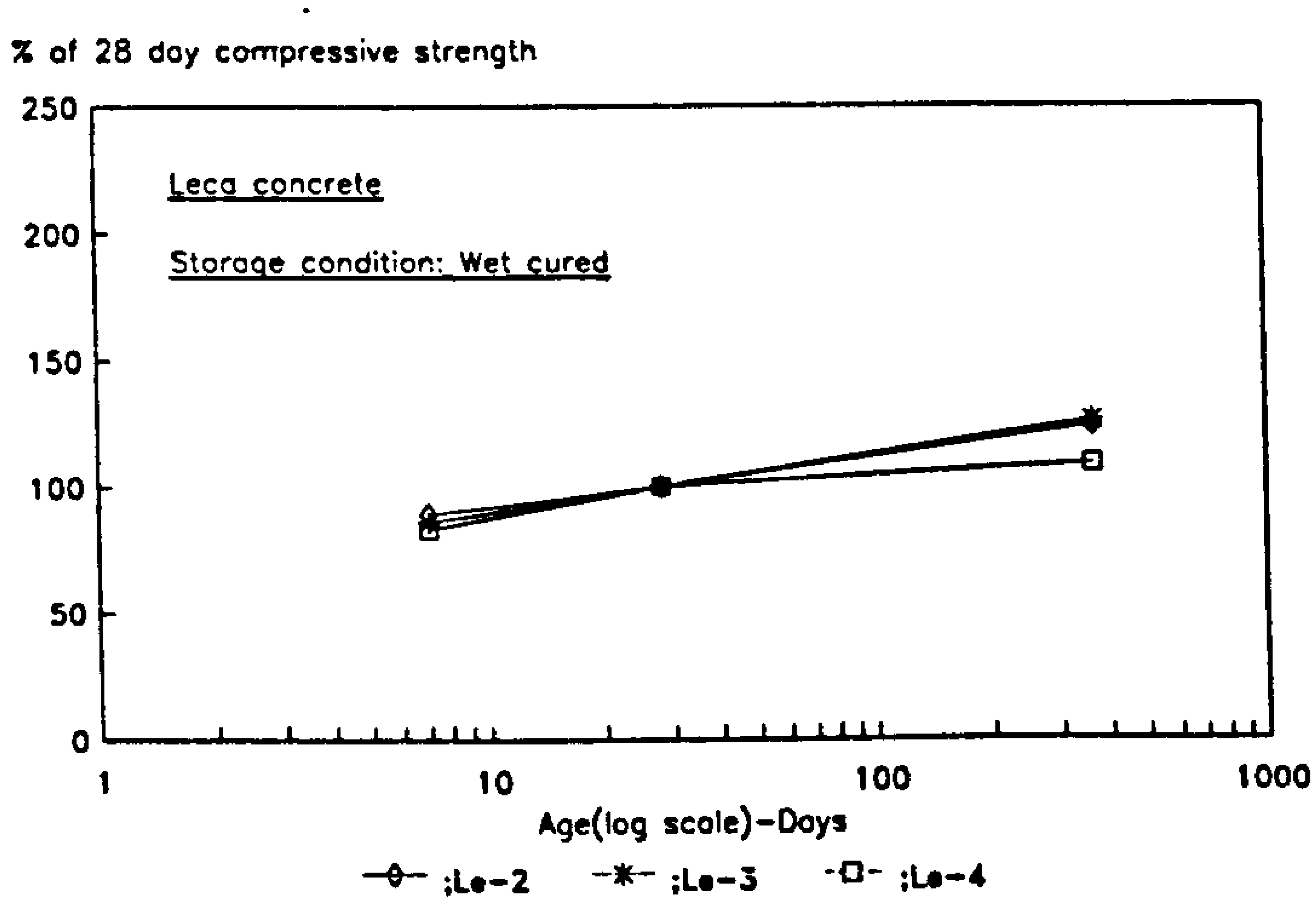


Figure 5.5:Development of compressive strength with age (Leca-Wet)

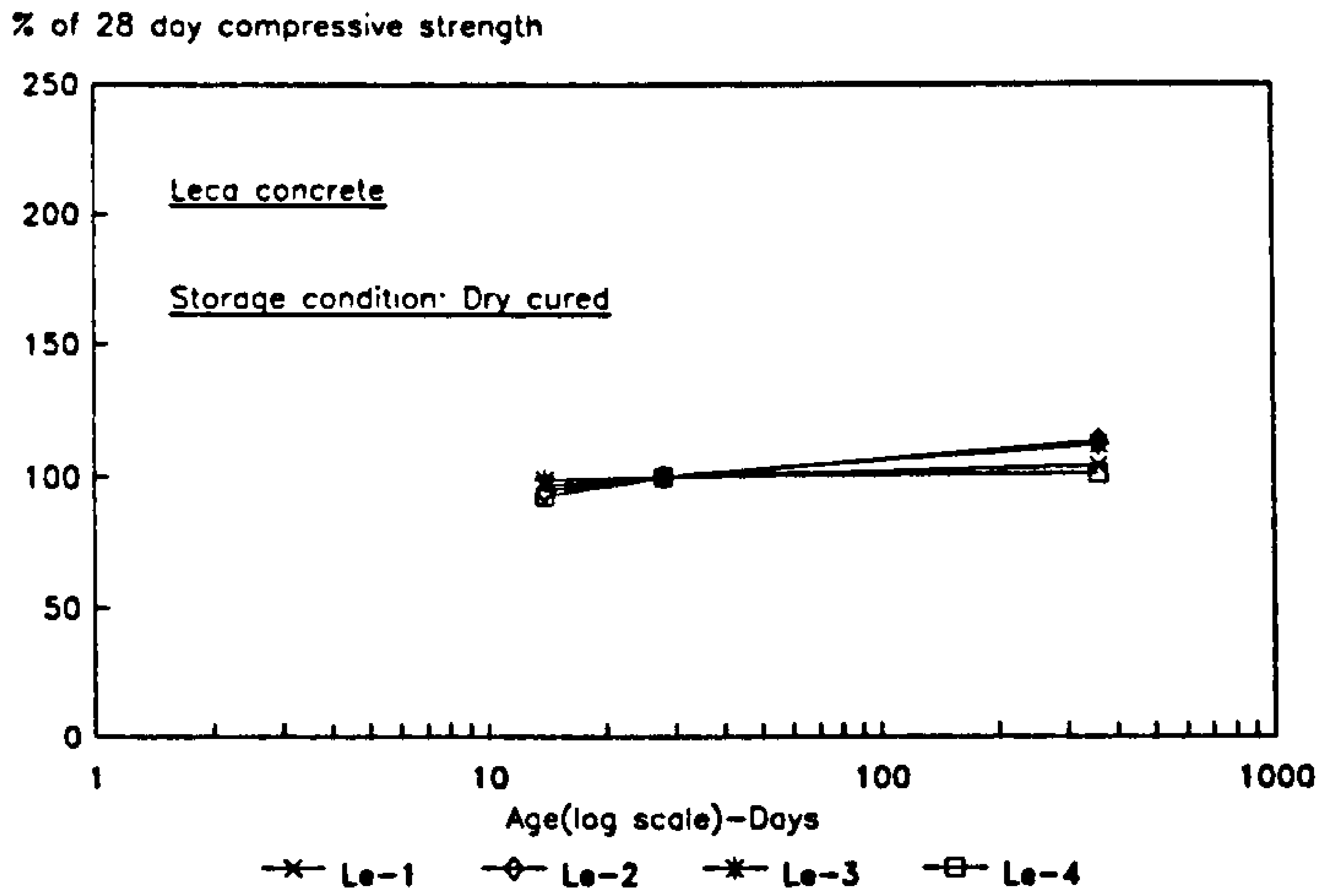


Figure 5.6:Development of compressive strength with age (Leca-Dry)

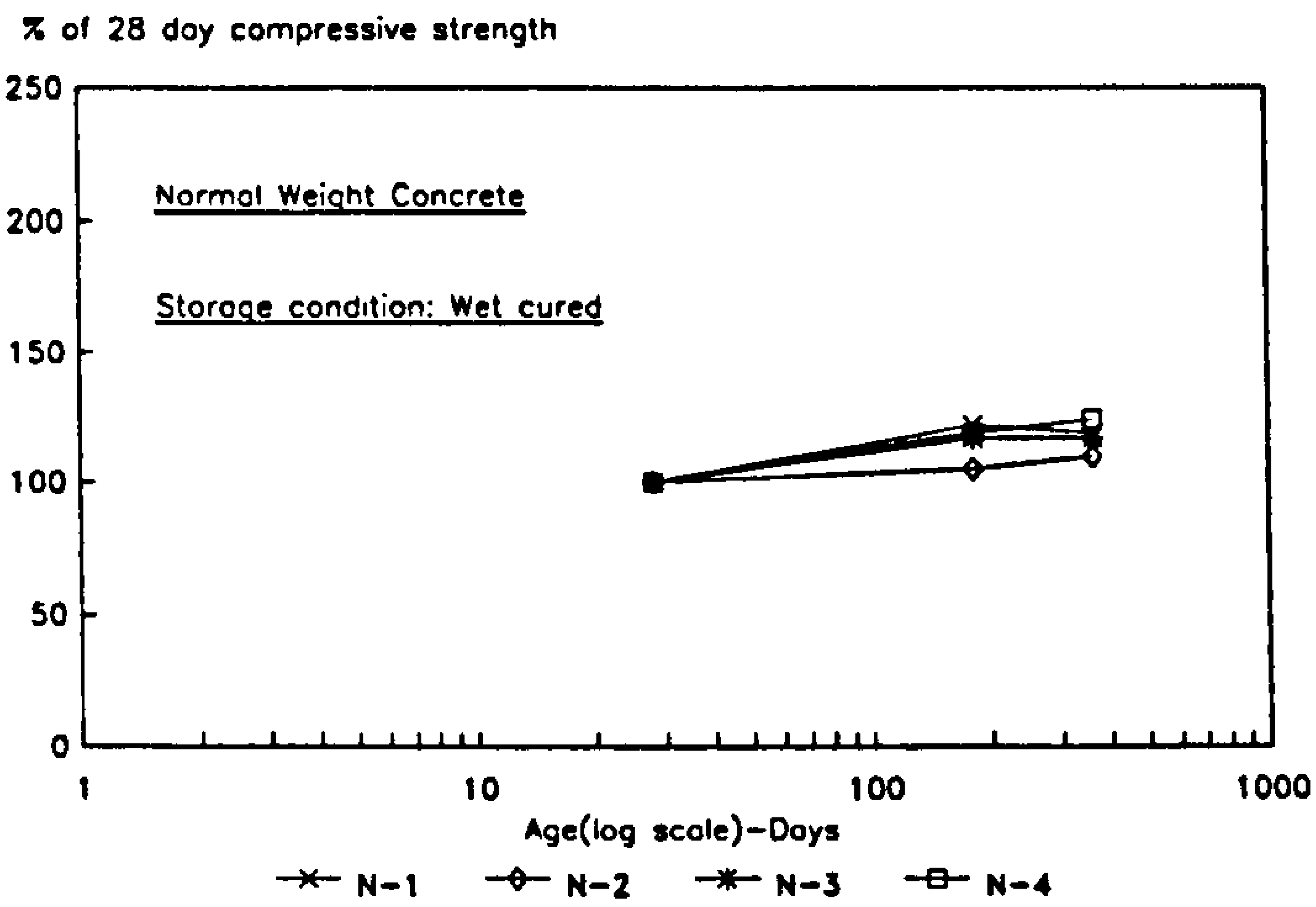


Figure 5.7:Development of compressive strength with age (Normal-Wet)

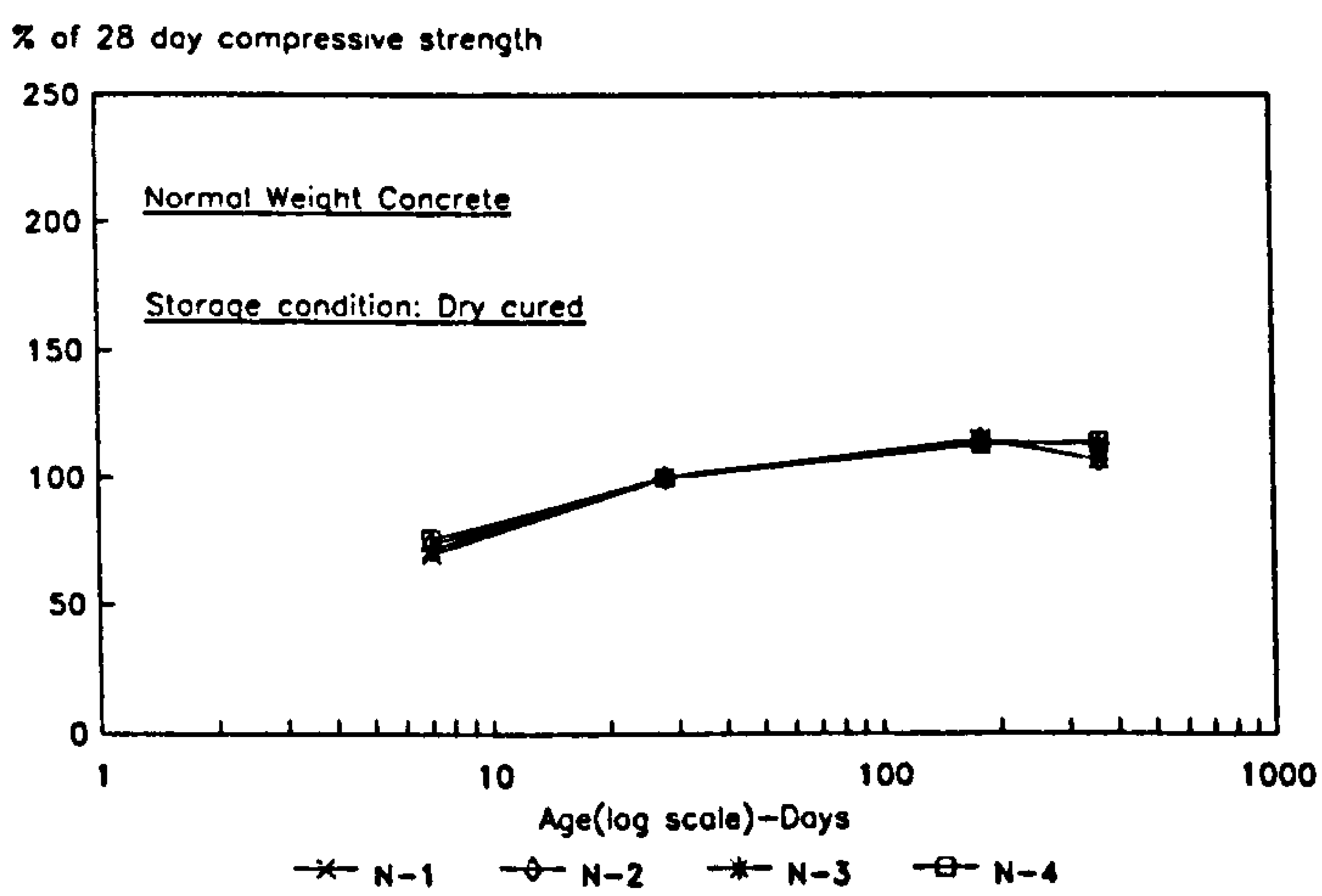


Figure 5.8:Development of compressive strength with age (Normal-Dry)

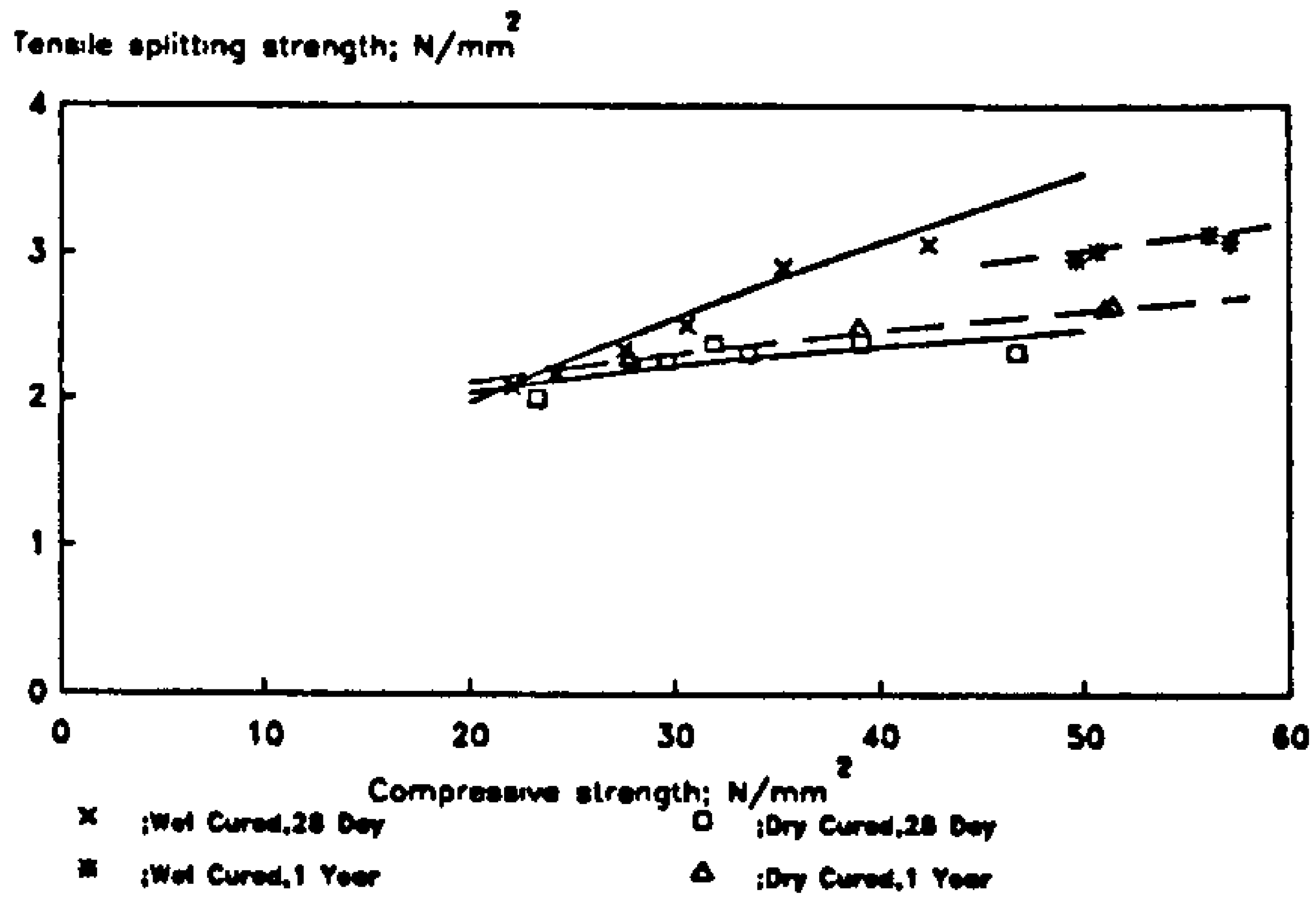


Figure 5.9:Relationships between tensile splitting and compressive strengths for fully Lytag concrete

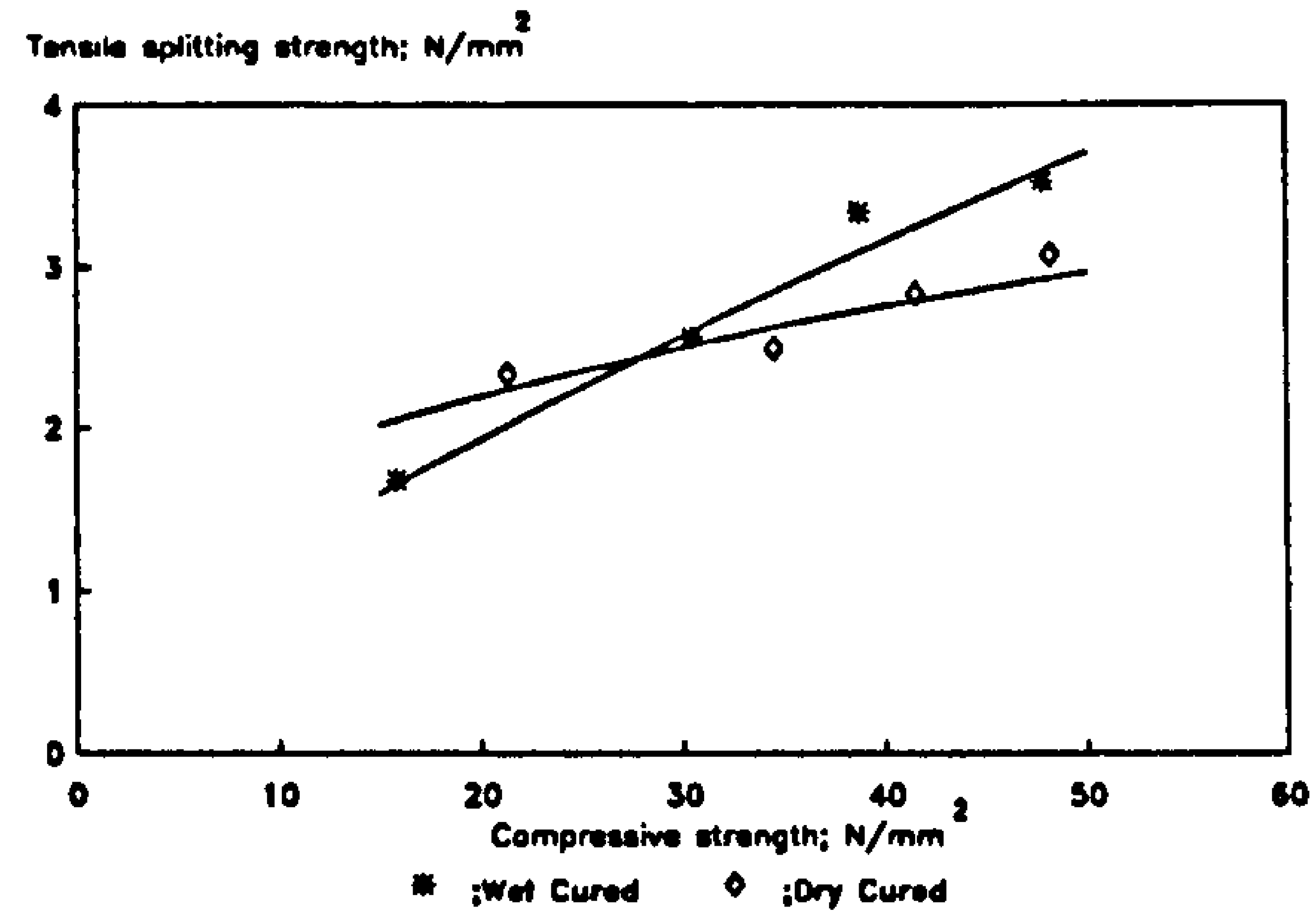


Figure 5.10:Relationships between tensile splitting and compressive strengths for semi Lytag concrete

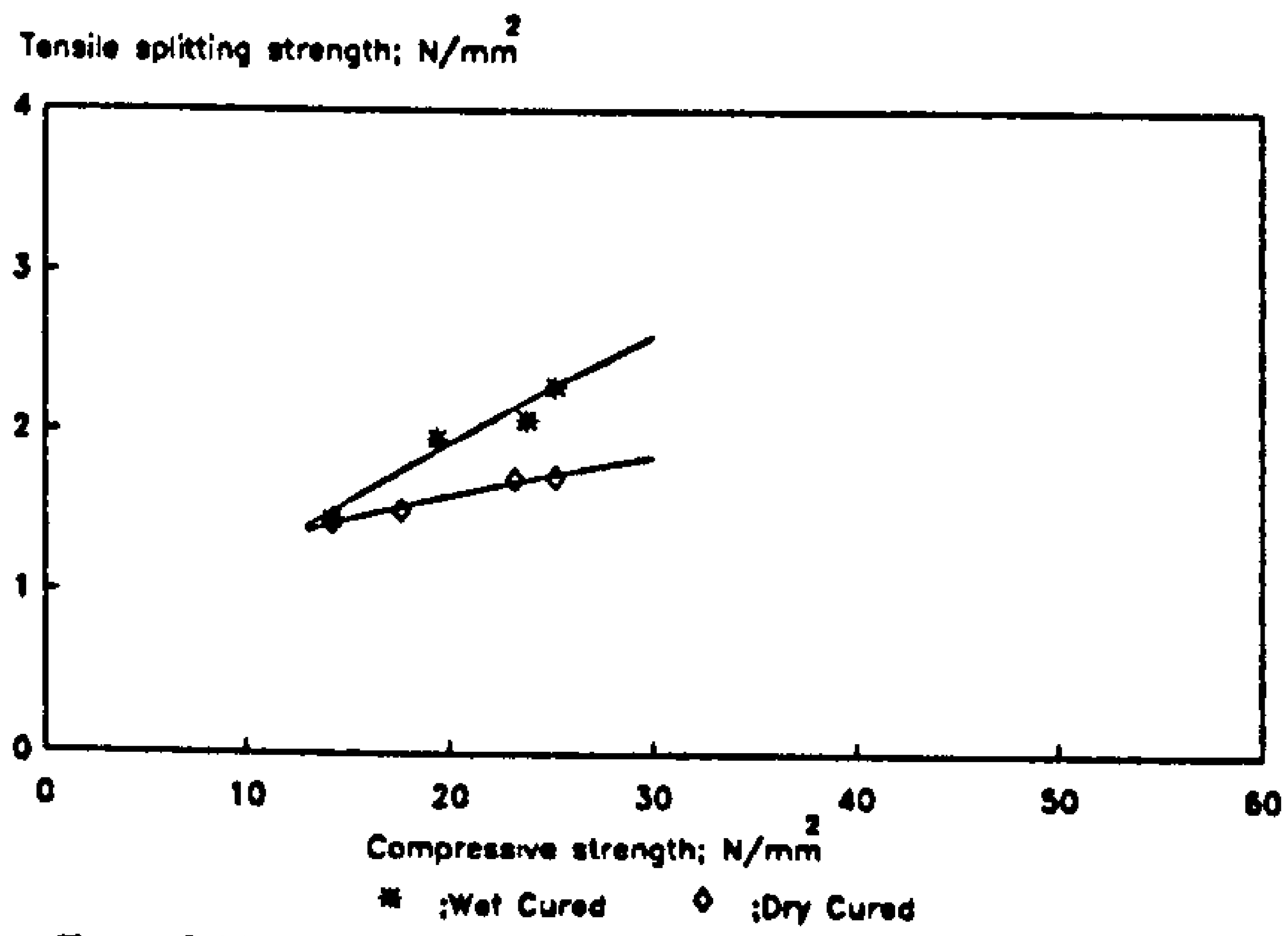


Figure 5.11:Relationships between tensile splitting and compressive strengths for Leca concrete

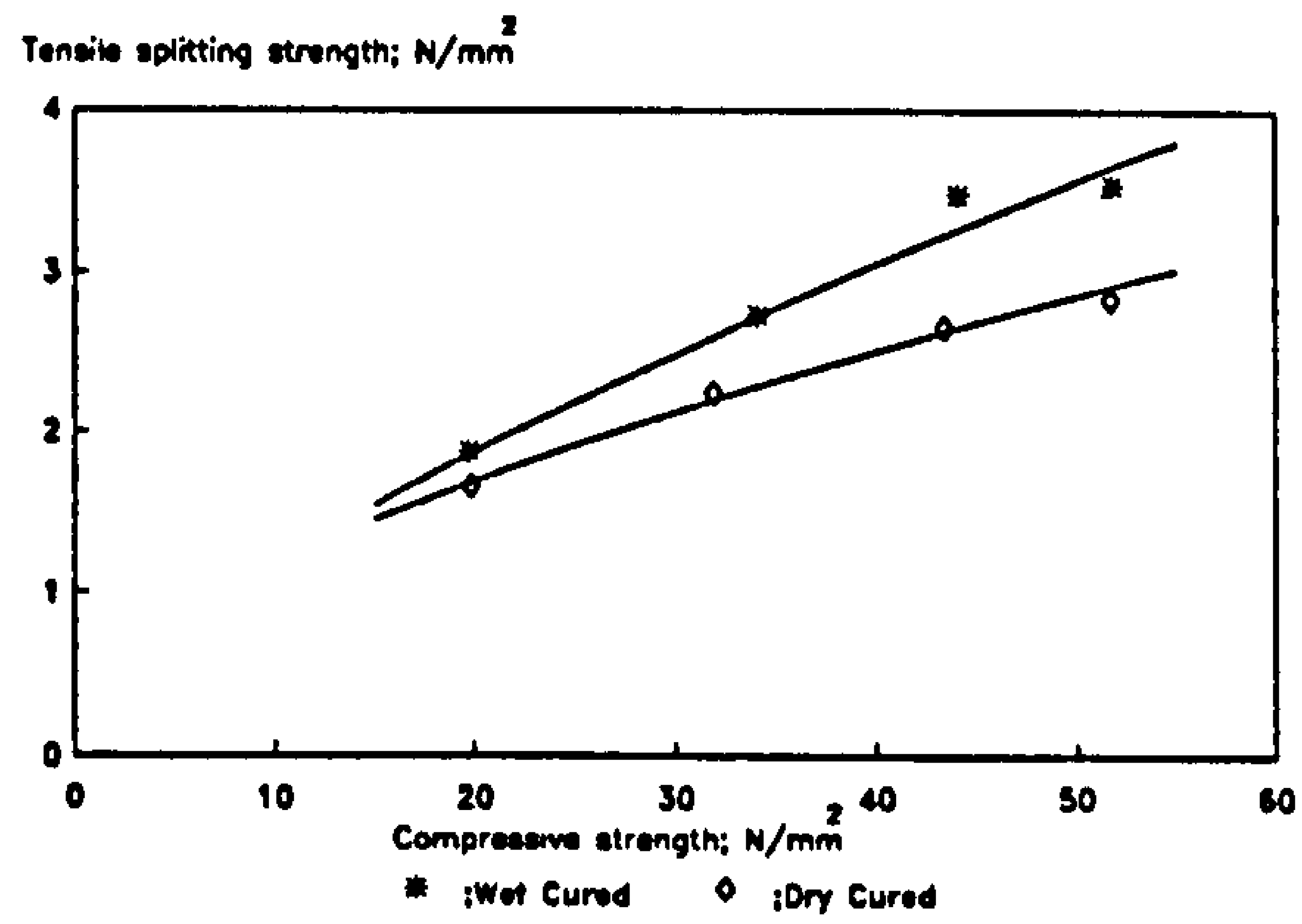


Figure 5.12:Relationships between tensile splitting and compressive strengths for Pellite concrete

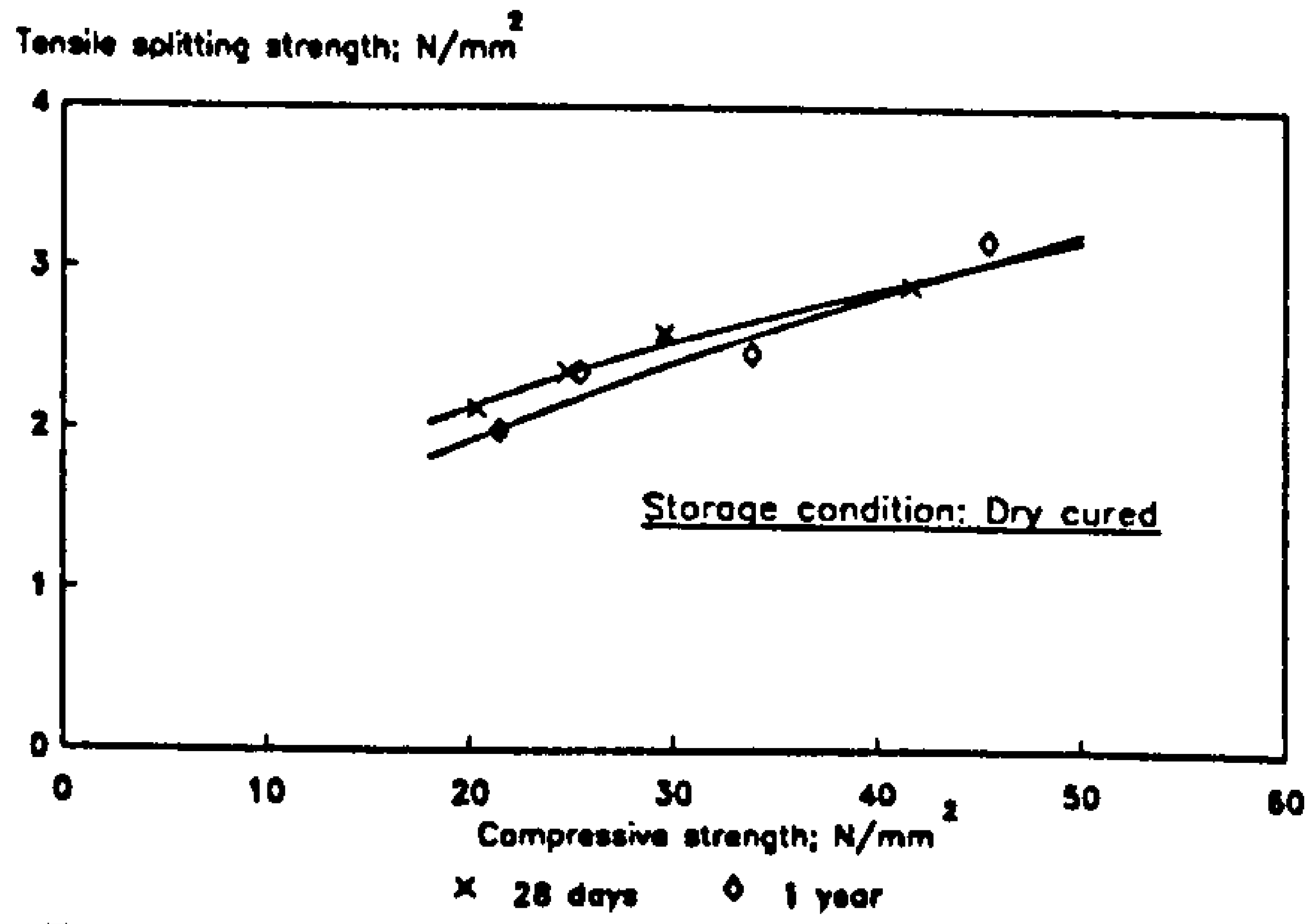


Figure 5.13:Relationship between tensile splitting and compressive strengths for normal weight concrete

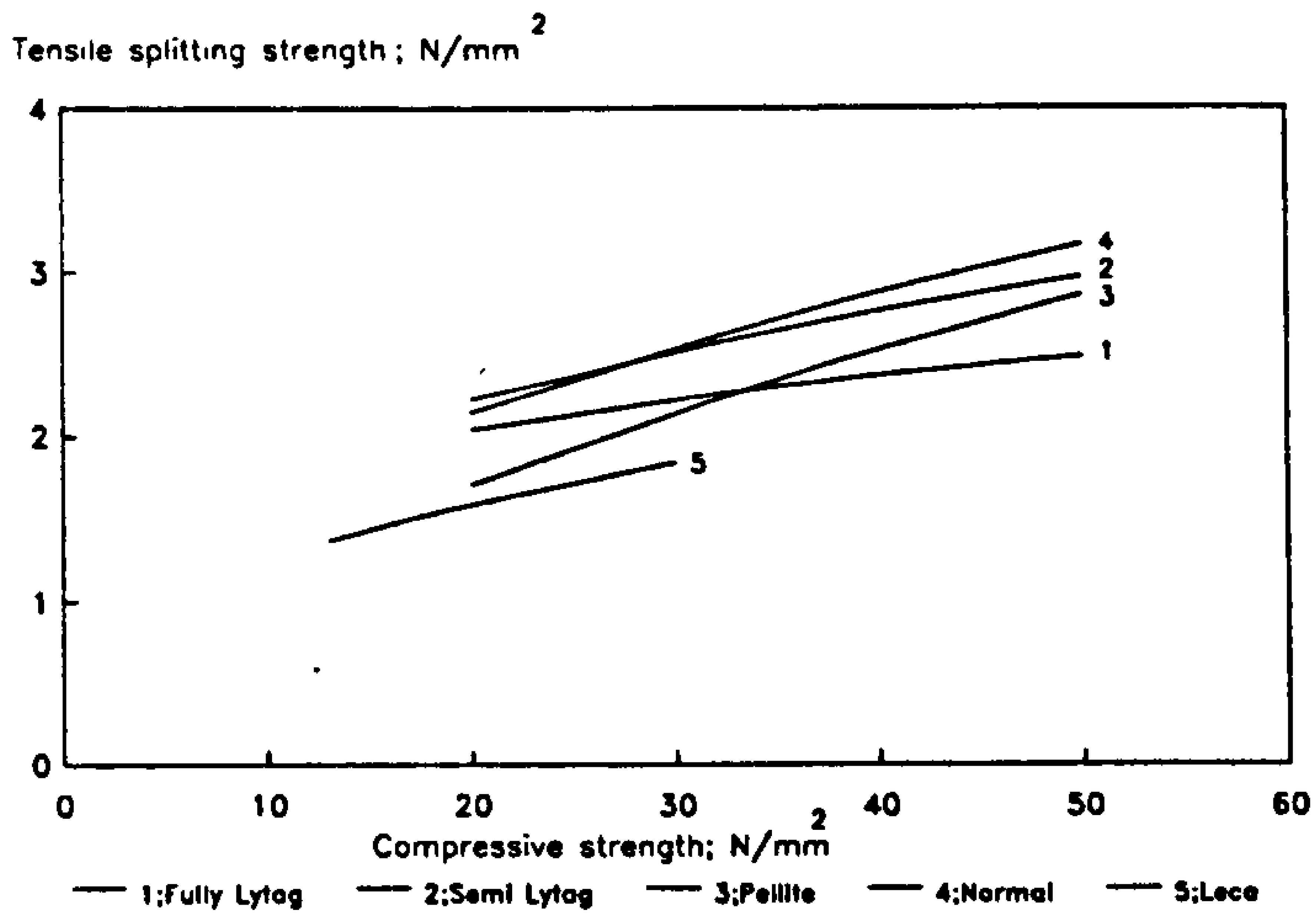


Figure 5.14:Relationship between tensile splitting and compressive strengths under dry curing condition

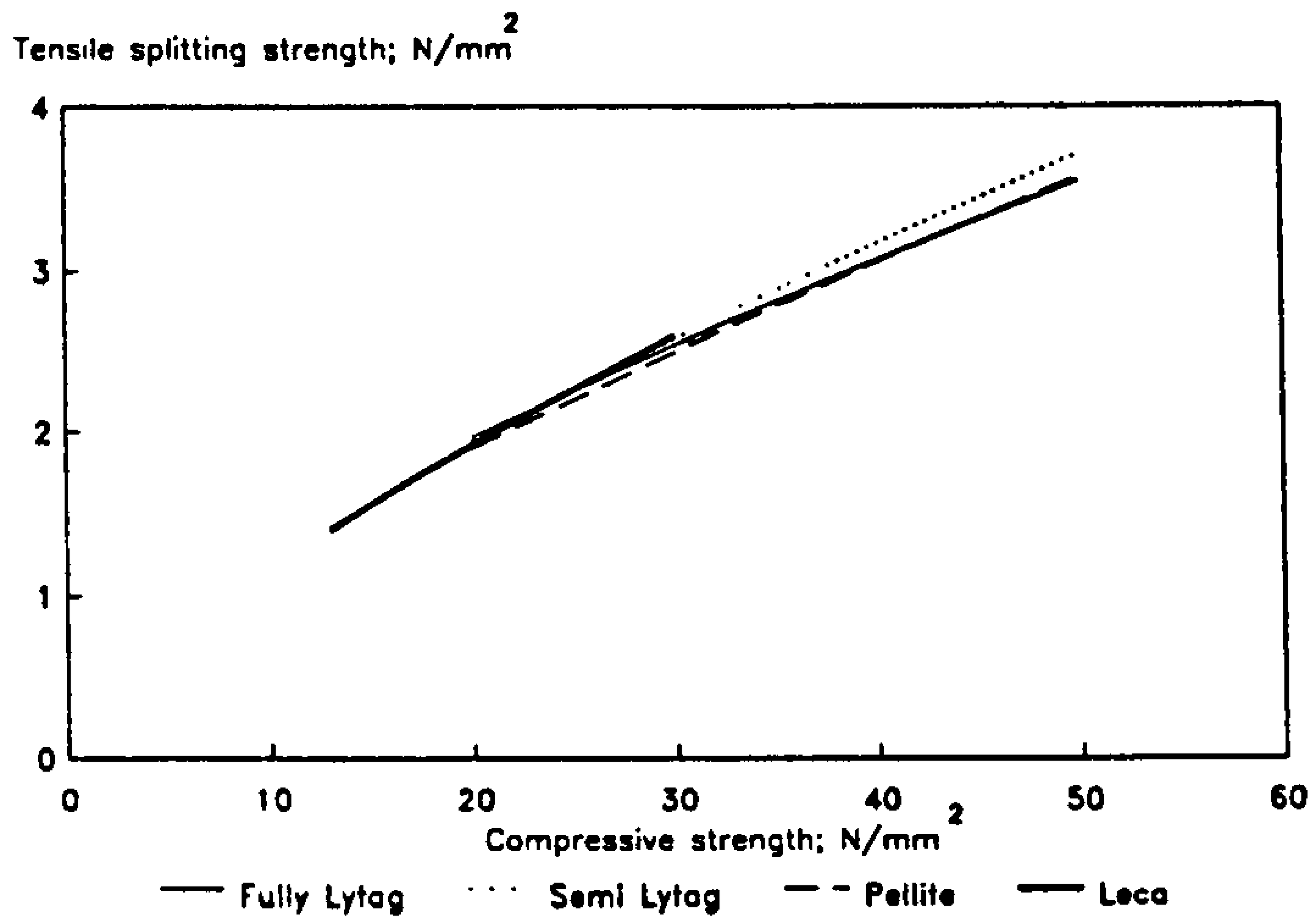


Figure 5.15:Relationship between tensile splitting and compressive strengths under wet curing condition

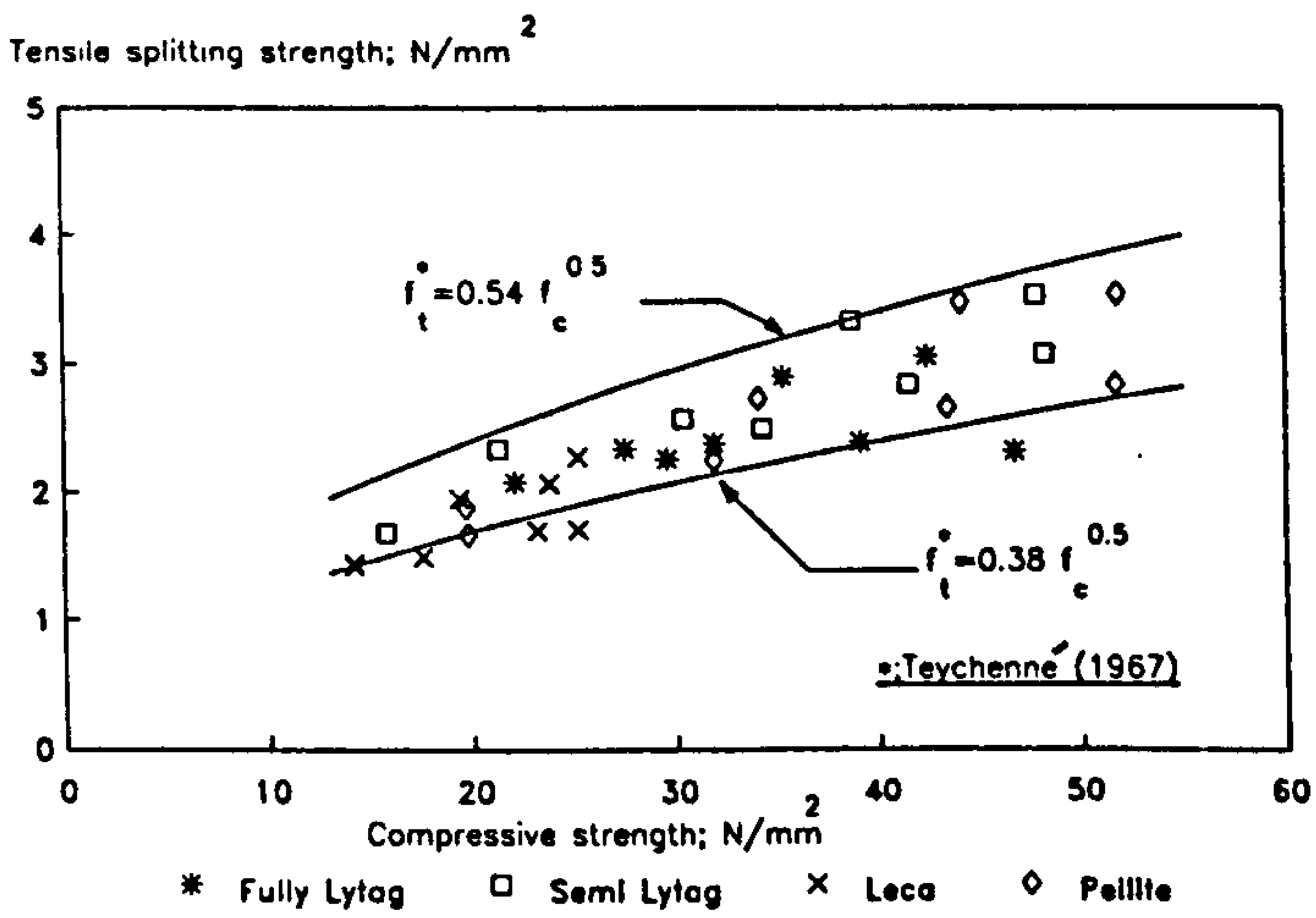


Figure 5.16:Relationship between tensile splitting and compressive strengths of different lightweight concretes

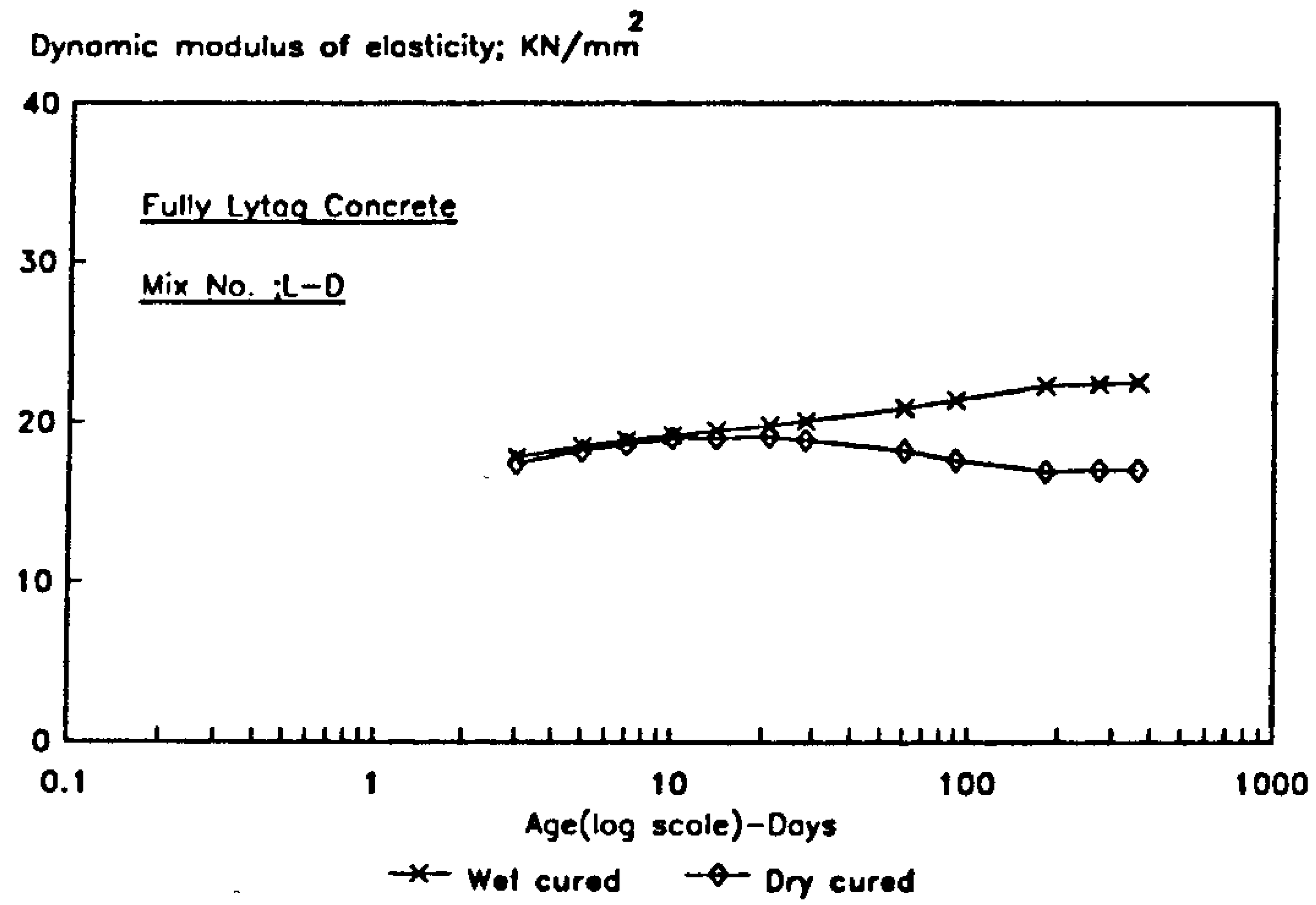


Figure 5.17:Development of dynamic modulus of elasticity with age for fully Lytag concrete

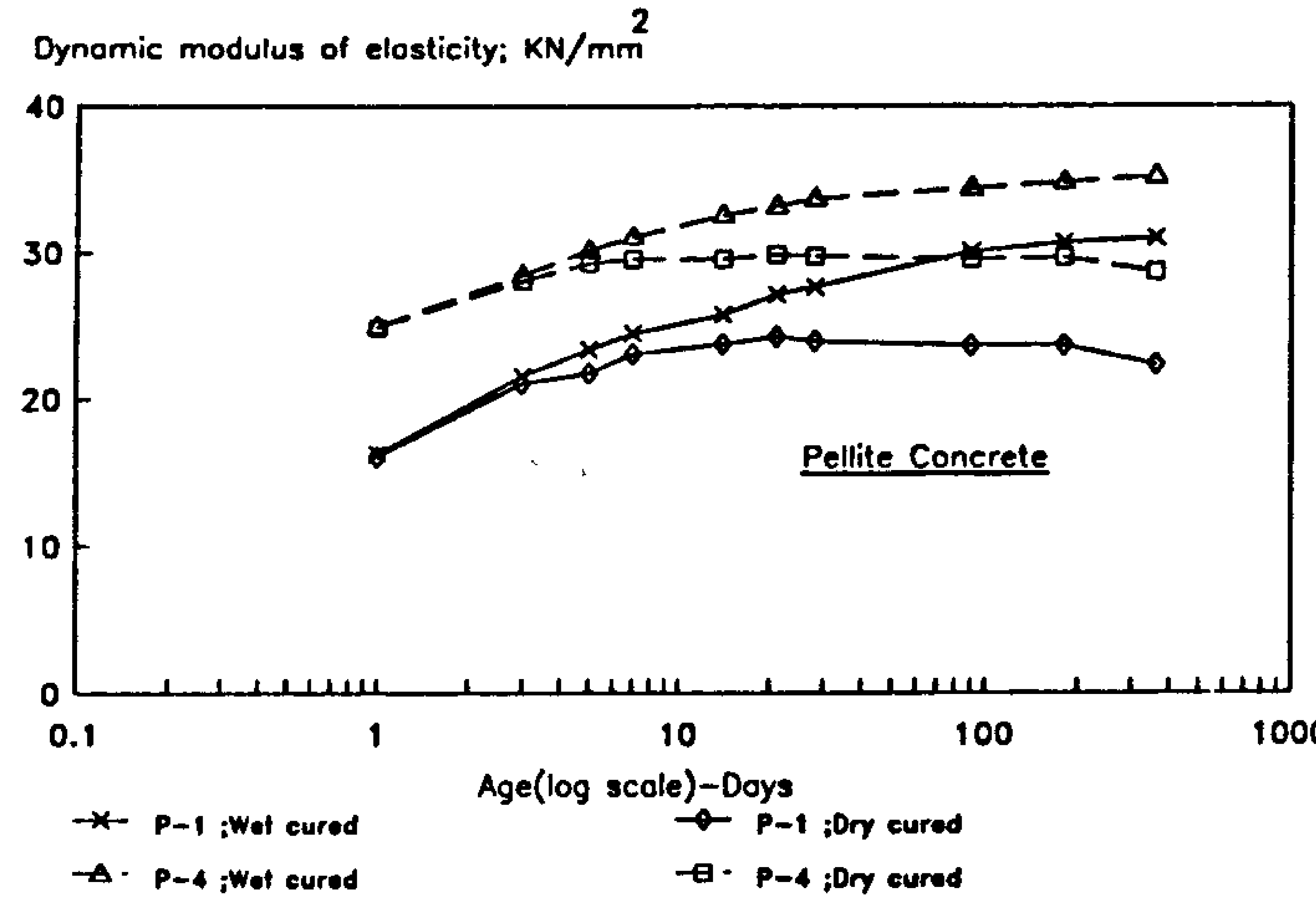


Figure 5.19:Development of dynamic modulus of elasticity with age for Pellite concrete

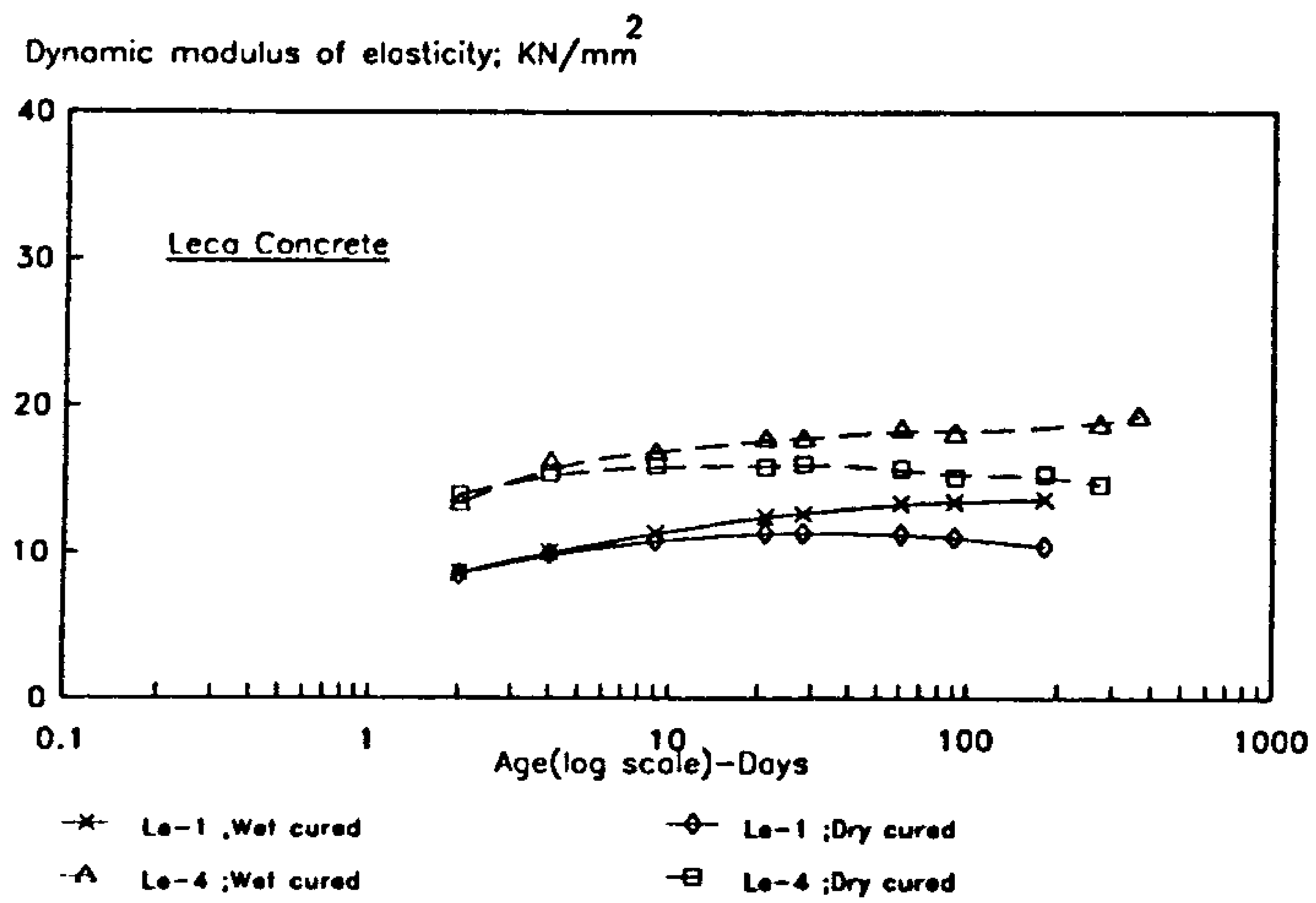


Figure 5 18:Development of dynamic modulus of elasticity with age for Leca concrete

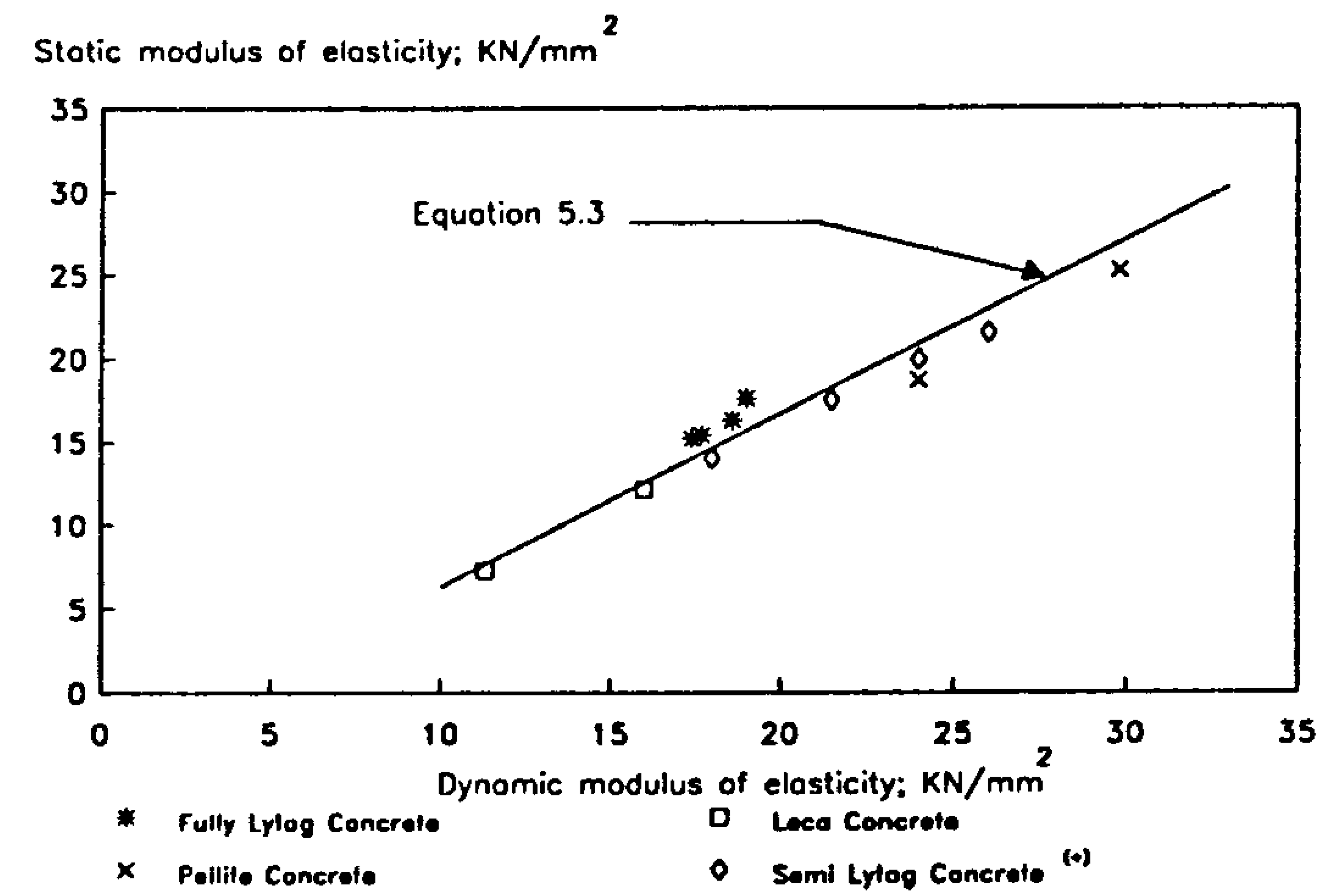


Figure 5 20 Relationship between static and dynamic modulus of elasticity

+;Lambert(1982)

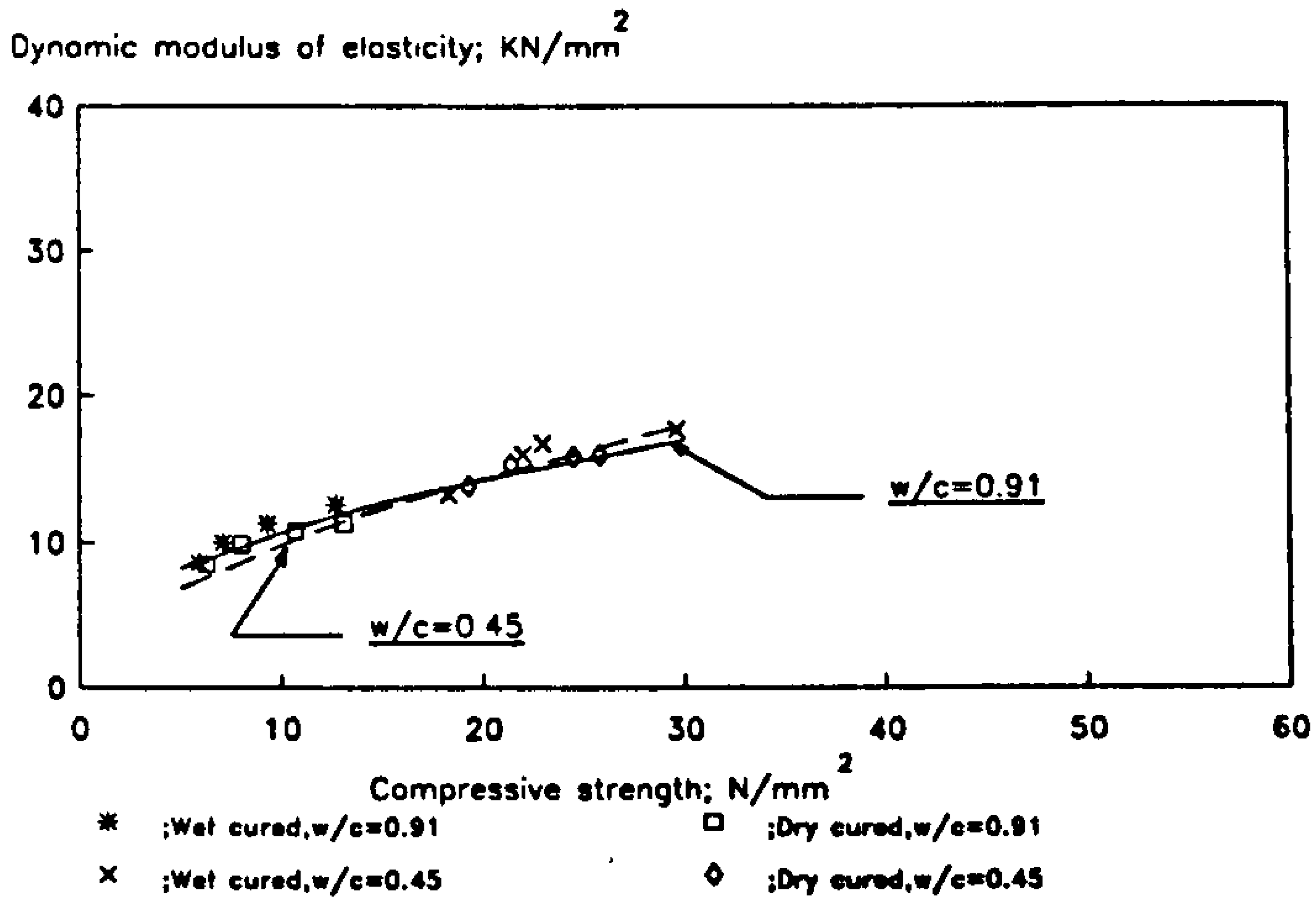


Figure 5.21:Relationship between compressive strength and dynamic modulus of elasticity for Leca concrete

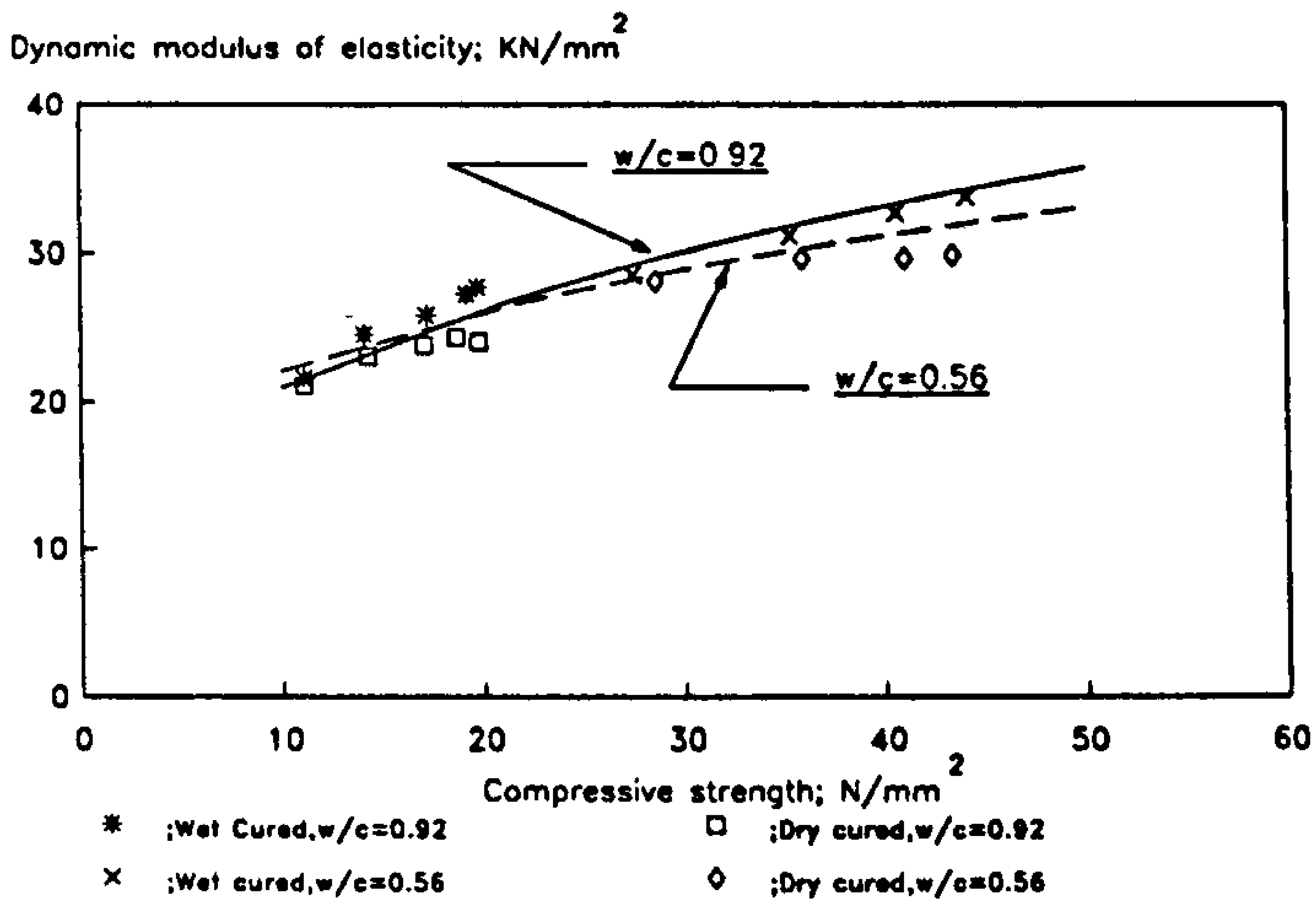


Figure 5.22:Relationship between compressive strength and dynamic modulus of elasticity for Pellite concrete

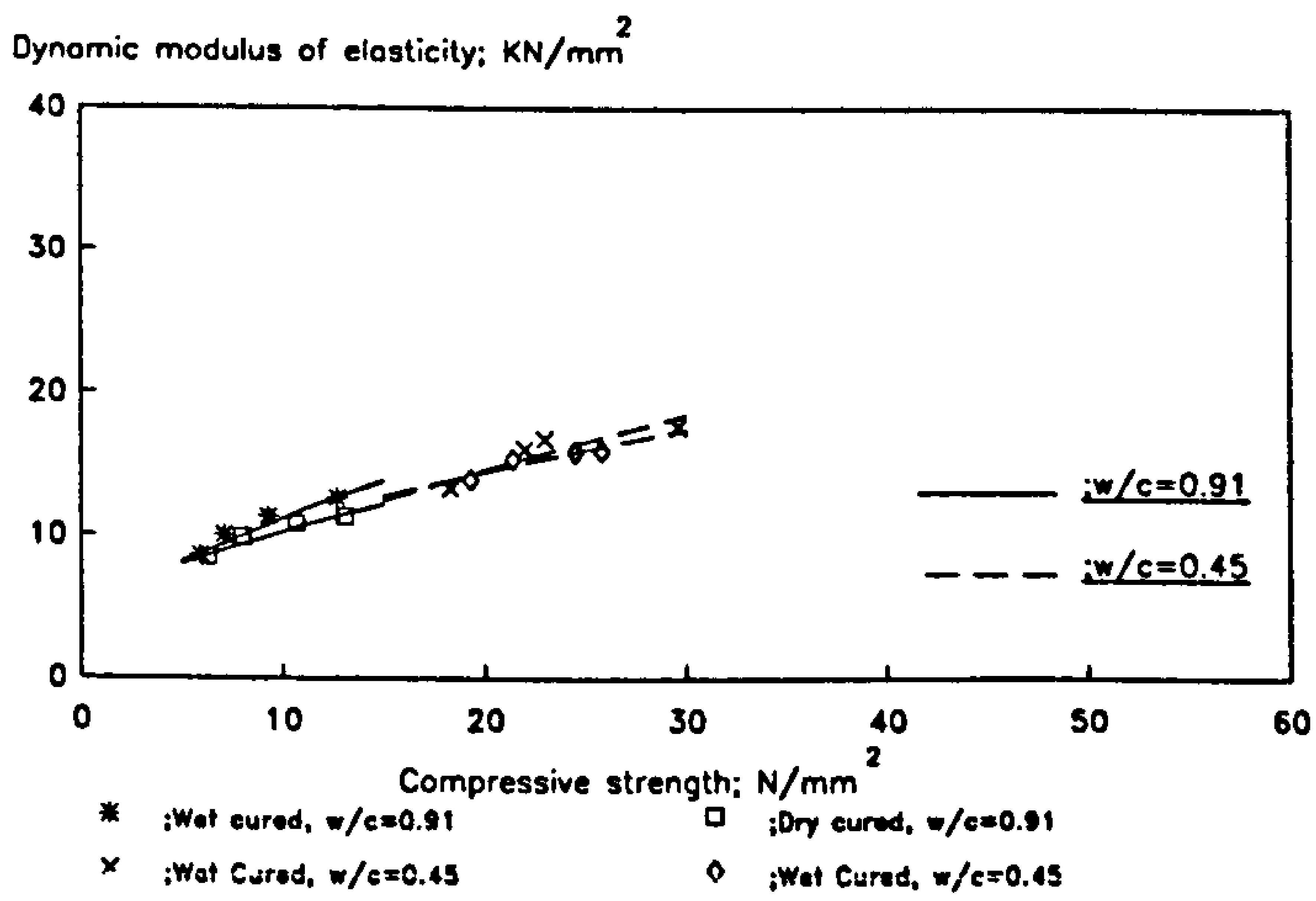


Figure 5.23;Relationship between dynamic modulus of elasticity and compressive strength for wet and dry cured Leca concrete

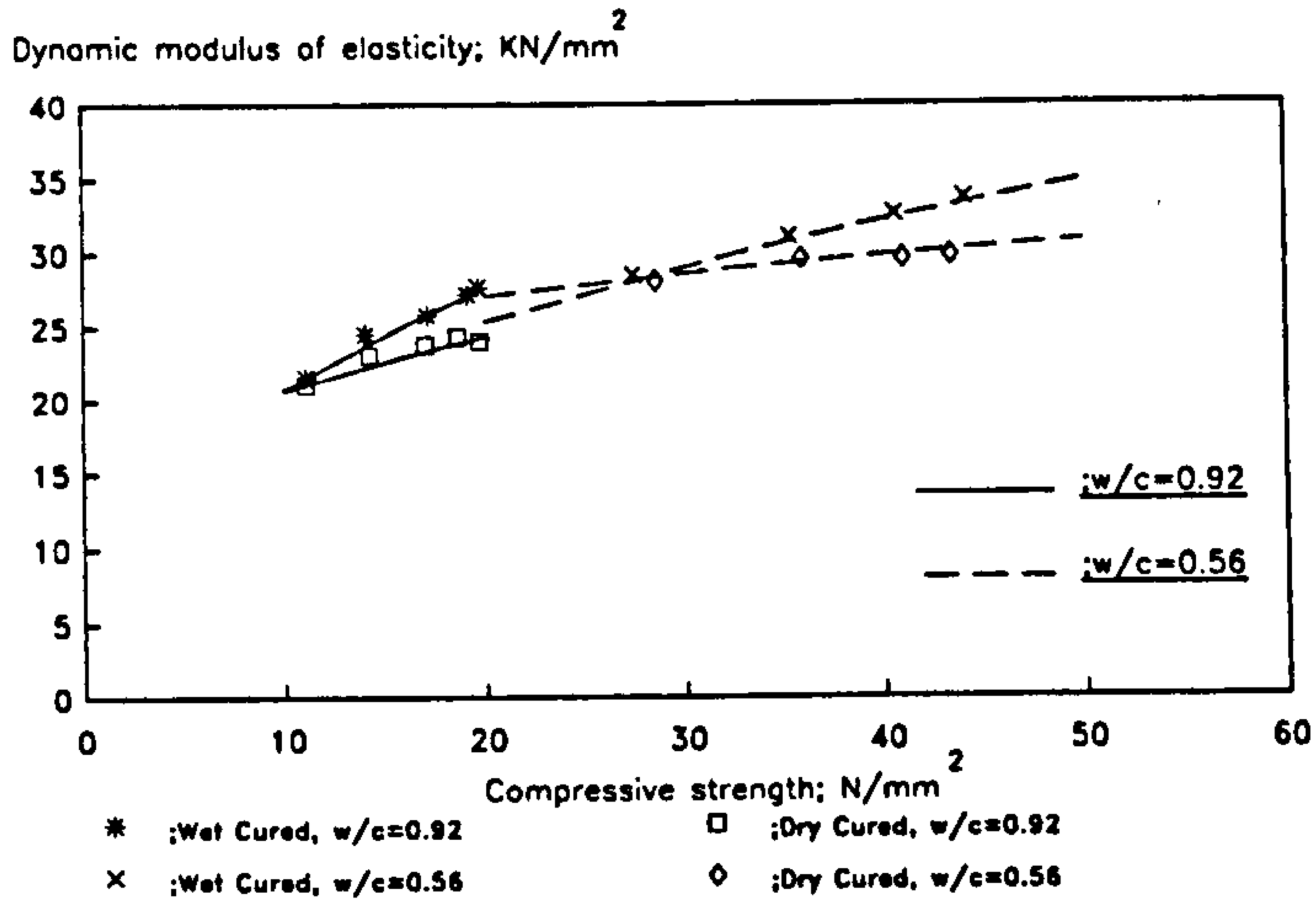


Figure 5.24:Relationship between dynamic modulus of elasticity and compressive strength for wet and dry cured Pellite concrete

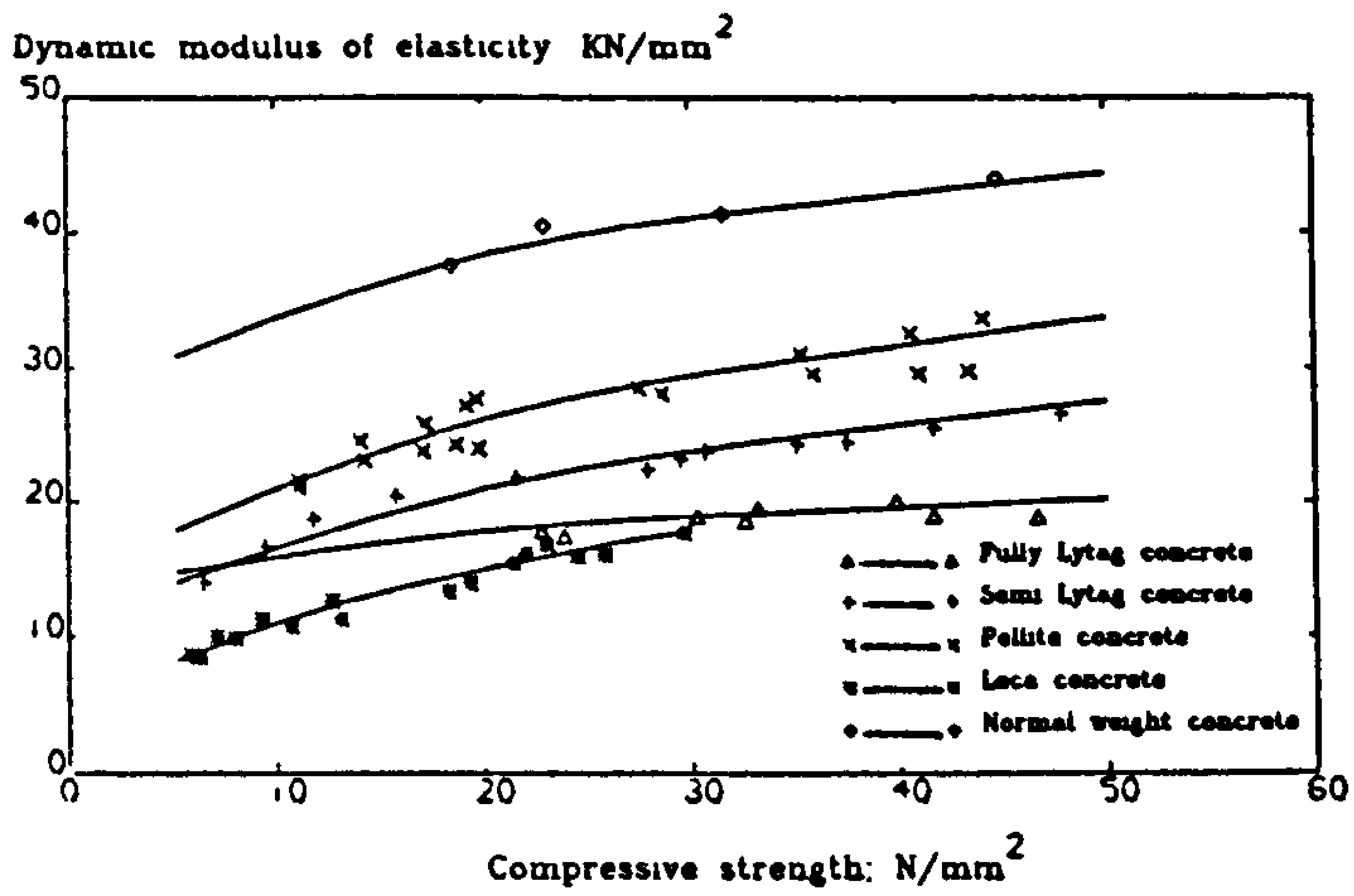


Figure 5.25 Relationships between dynamic modulus of elasticity and compressive strength for different types of concrete

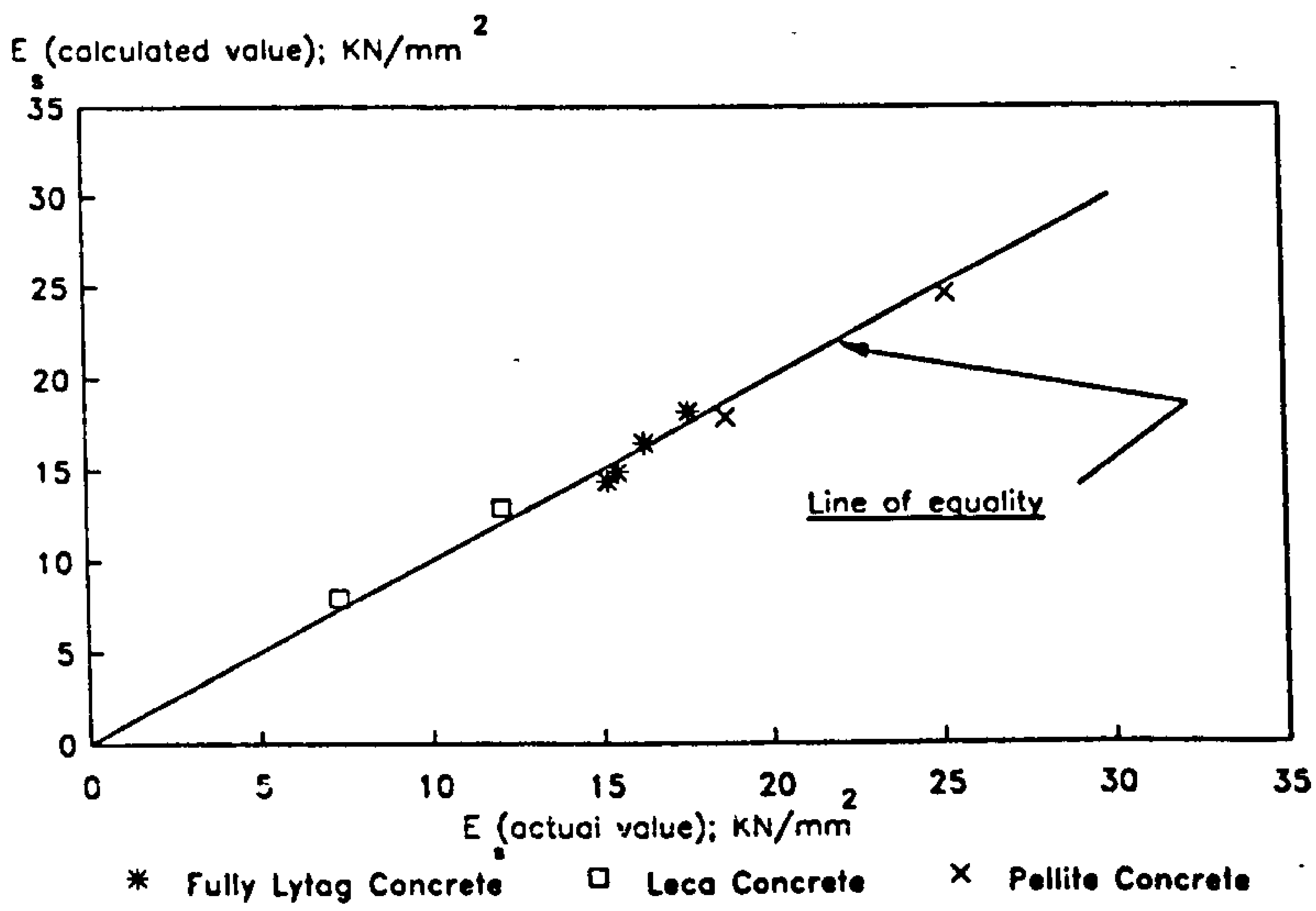


Figure 5.26:Comparison between actual and calculated static modulus of elasticity based on equation 5.4

CHAPTER 6. EXPERIMENTAL INVESTIGATION ON INSITU TEST METHODS

6.1 INTRODUCTION

The experimental work presented in this Chapter has been carried out to assess the validity of a number of insitu test methods described in Chapter Three when evaluating insitu strength of lightweight concrete. All available types of lightweight concrete, and in some instances normal weight concrete, have been used in the test programme described in Chapter 4 (see Table 4.5).

Relationships between the insitu test parameter measurement and the cube compressive strength were examined for a number of factors such as age effect and curing conditions. Attention has been given to significance of aggregate type on the correlations as well as the effect of sand replacement.

6.2 NON-DESTRUCTIVE TEST RESULTS

A summary of 100mm cube compressive strength, rebound hammer, and pulse velocity results together with within-test coefficients of variation are presented in the following sections. A non-linear regression analysis was performed to correlate the compressive strength with test parameter measurements in each case.

6.2.1 Rebound Hammer

The limited applications to insitu strength assessment of normal weight concrete have been discussed in Chapter 3. Similar shortcomings seem to be likely for lightweight concrete although there are few published results. Greene (1954) and more recently Nasser and Al-Manaseer (1987) have however indicated that different strength correlations are required for concrete made with expanded clay or shale aggregates. More thorough investigation of the rebound hammer is justified to support its role as a supplementary test.

Four types of lightweight concrete were taken along with normal weight concrete for comparison. One typical mix was considered for each type of concrete as illustrated in Table 4.6. Rebound hammer tests were carried out at varying ages from 1 to 28 days on cubes cast from a single batch and cured under dry conditions (similar to large scale beam curing). Before any reading was taken each cube was securely clamped in a testing maching by applying a minimum restraining load of $7N/mm^2$, as recommended by BS 1881: Part 202 (1986). Five readings taken on each of two side faces and the bottom face were averaged separately according to ASTM recommendations (see Section 3.2.1.). These results are tabulated in Appendix B (Table B-1) along with coefficients of variation calculated after discarding unrepresentative readings. It appears that the rebound readings on the bottom face are 5 to 43% higher than those taken on the side faces. The lower limit was obtained with fully Lytag and Pellite concretes whereas the higher limit was created with Leca concrete. For semi Lytag and normal weight concretes, the rebound readings on the bottom face gave values 11% higher than those on the sides. Hence only the results taken from side faces were considered for analysis in all cases. In Leca concrete, considerable variations between side and bottom faces were also detected with other test types as discussed later in this Chapter. This phenomenon appears to be related to aggregate characteristics. Due to the low density of Leca, it was found that there is a tendency for it to float and mirgrate to the top of the specimen during vibration, as shown in Figure 6.1. Consequently, there is a non-uniform distribution of aggregate throughout the concrete with a layer of mortar at the bottom of the sample (see Figure 6.2). The high coefficients of variation of rebound readings obtained on side faces compared with low variations from bottom faces further confirms this heterogeneity of Leca concrete. Such problems have not been found with concretes made from Lytag and Pellite for which relatively more uniform concretes were obtained.

After completion of rebound hammer tests the sample cubes were crushed in the standard manner to obtain compressive strengths.

6.2.1.1 Rebound Number Versus Compressive Strength

The data obtained from the rebound hammer and compressive strength tests are shown in Figure 6.3 to 6.7. A power curve model was used to correlate rebound number with compressive strength, i.e.

$$f_c = aR^b \tag{6.1}$$

where R = rebound number

f_c = compressive strength; N/mm^2

a,b = constants.

The equations obtained for different types of concrete are summarized in Table 6.1 together with coefficients of correlation.

The regression curves and the confidence limits for the curves have been fitted to the data obtained from each type of concrete. The 95% confidence limits have been calculated in the manner described by Kennedy and Neville (1976) and using a transformation approach explained by Chabowski and Bryden-Smith (1980).

The accuracies of strength estimation based on 95% confidence limits are shown in Table 6.2 for a strength level of $30N/mm^2$ (except Leca, where a strength level of $20N/mm^2$ was used). With the exception of Leca concrete, are all generally of the same order for both lightweight and normal weight concretes, but with fully Lytag concrete being highest. For Leca the 95% confidence limits were high as expected due to non-uniformity of the concrete.

Table 6.1 indicates that statistically the relationships between rebound number and compressive strength are good as evidenced by high coefficients of correlation. It must be borne in mind however that the high correlations obtained here were based on specimens cast from a single batch for each type of concrete and cannot be generalized. In addition, Bellander (1979) showed that the relationship between rebound number and compressive strength in the finished structure is different from that obtained on standard cube specimens. Further unreliability of the rebound hammer test was experienced by the Author when assessing the strength variations on large scale beams and is discussed in Chapter 8.

Figure 6.8 compares the relationships obtained for different types of concrete along with the calibration given by the manufacturer. This figure confirms that different relationships exist for various types of concrete, including different types of lightweight concretes.

It is noted that fully Lytag concrete yields a higher rebound number than normal weight concrete for a given compressive strength. The reason for this is not fully understood but is believed to be related to the mechanism of test. Two possible contributory factors may be the high percentage of fine particle in fine Lytag which creates fewer minute air pockets, and the high degree of carbonation which has been experienced by the Author in that concrete (carbonation depth was approximately 20% higher than that obtained in normal weight concrete).

The calibration curve supplied by the manufacturer was found to be in reasonable agreement with that obtained with fully Lytag concrete and they are in good agreement up to a strength level of 25N/mm^2. However for other types of concrete, the manufacturer's curve would have caused an underestimation of the strength which is most pronounced for Leca and Pellite concretes.

6.2.1.2 Within-Test Variability

The within-test variability referred to here basically relates to the single laboratory/operator/material precision of the test method. This has been based on coefficients of variation which have been calculated from thirty rebound readings taken on the side faces of three 100mm cube specimens. Two opposite side faces with five tests on each face were considered for each cube. Figures 6.9 to 6.13 present graphically the variation of test results on different types of concrete along with their coefficients of variation.

Comparing the variation in test results for different types of concrete, the coefficients of variation were in the order of 6 to 22.5%. The lower and upper extreme values correspond to fully Lytag and Leca concretes respectively. This difference is likely to be related to aggregate characteristics and its distribution within the concrete.

6.2.2 **Pulse Velocity**

A study of pulse velocity development with time and its relationship to some mechanical properties of hardened concrete was undertaken for different types of concrete. The experimental details have been given previously in Table 4.5 and the detailed results obtained are presented in Tables B-2 and B-3.

6.2.2.1 Pulse Velocity Versus Age

Pulse velocity development as a function of age was studied for different types of lightweight concrete. Two mixes were used for each type of concrete and the study was made separately on each mix cast from a single batch to provide the low and high strength concretes mixes. The pulse velocity was measured on wet and dry cube specimens at different ages from 1 to 360 days. Typical pulse velocity development with time for Pellite and Leca concretes are shown in Figures 6.14 and 6.15. It can be seen that for each mix the pulse velocity increases rapidly at the early age and, in contrast to strength, is not linear when plotted as a function of the logarithm of curing time. The dry cured concrete showed a drop in pulse velocity after three to four weeks and this is more pronounced at low strength owing to the higher permeability of the concrete. For wet cured concrete, the long term pulse velocity development for high strength mix was at a slower rate than low strength concrete. This might be explained by the pulse velocity at high strength level tending to reach a limiting value.

6.2.2.2 Pulse Velocity Versus Mechanical Properties of Concrete

In practice there are generally two parameters which are of particular interest to the structural engineer, namely modulus of elasticity and strength. In the current investigation efforts were made to correlate these two parameters with pulse velocity and the results obtained for different types of concrete are summarized in Tables B-2 and B-3.

6.2.2.2.1 Pulse Velocity Versus Dynamic Modulus of Elasticity

The test results have been plotted for different types of concrete in Figure 6.16. The combined data from lightweight concretes were used to generate a regression equation for predicting dynamic modulus of elasticity from pulse velocity.

This was in the form

$$E_d = 0.51\ V^{2.82} \qquad \text{(with } r = 0.984\text{)} \qquad (6.2)$$

Where E_d in KN/mm^2 and V in Km/sec

Experiments have shown that there is a good correlation between pulse velocity and dynamic modulus of elasticity for a wide range of lightweight concretes. 95% confidence limits on estimated dynamic modulus of elasticity were found to be ± 13%. While studying the effect of age on pulse velocity and dynamic modulus of elasticity (Section 6.2.2.1 and Section 5.4.1) it was seen that the pattern of increase was fairly similar for both parameters. In fact the pulse velocity is very closely related to the elastic properties of the medium as recognised by other investigators (Elvery and Forrester, 1971), (Bungey, 1984).

A limited number of tests were also carried out on normal weight concrete for comparison and these are given also in Figure 6.16 along with the general relationship given by BS 1881 : Part 203 (1986) for normal weight concrete. It seems that the relationship found for lightweight concretes yields dynamic modulus of elasticity values below those expected of normal weight concretes by an average of 14%. The results obtained for normal weight concrete in this investigation lay mid-way between the relationship given by the Author for lightweight concrete and the one suggested by British Standard for normal weight concrete.

6.2.2.2.2 Pulse Velocity Versus Compressive Strength

The various factors affecting the relationship between pulse velocity and compressive strength have been extensively discussed in Section 3.2.2.2.1. The current investigation examined the influence of aggregate type and curing regime on the pulse velocity/compressive strength relationship for different types of lightweight concrete. Two mixes were used for each type of concrete to provide low and high strength specimens. The relationship between pulse velocity and compressive strength was investigated separately on each mix with specimens cast from a single batch of concrete. The tests were performed under wet and dry curing conditions at various ages from 3 to 28 days (Table B-3). The pulse velocity measurements were taken across a pair of opposite side faces of 100mm cubes with the exception of Leca concrete where the tests were carried out between both pairs of opposite side faces and the average of measurements were considered. This was because of relatively greater non-homogeneity of concrete associated with non-uniform distribution of aggregate and differences in properties of aggregate and matrix. This difficulty was less significant when the tests were performed along the length of prisms or larger masses of concrete such as the beams used for strength variation studies. Further discussion on this will be given in Chapter 8.

Figures 6.17 to 6.20 show the pulse velocity/compressive strength data based on average values. A regression analysis for pulse velocity versus compressive strength was performed and the correlation was established using an exponential curve model described in Section 3.2.2.2. The results of this are shown in Table 6.3 for each type of lightweight concrete under different curing conditions. High correlations between pulse velocity and compressive strength were obtained for each given mix under specific curing conditions except for fully Lytag concrete under dry curing conditions. A possible explanation for this particular poor correlation might be due to a relatively high difference in nature of pulse velocity-compressive strength development in the short term. In other words, in short term strength development, the compressive strength of fully Lytag concrete under dry curing is comparatively greater than the wet cured compressive strength, whereas this behaviour is reversed for pulse velocity development, especially at ages above two weeks.

Table B-2 and Figure 6.17 to 6.20 show that for concretes with Lytag and Leca the influence of curing is less significant at early age (up to 7 days), possibly due to the large reservoir of water absorbed in the aggregate. However at later ages the significance of curing condition is evident and this is more pronounced for the low strength mix. Significant dependency on curing condition has also been detected for Pellite concrete as shown in Figures 6.20. Also from the above figures, it is clearly shown that the pulse velocity under dry curing condition varies in a narrow range compared to that obtained under wet curing condition. Hence a steeper pulse velocity/compressive strength relationship was obtained for dry curing conditions and this explains the smaller influence of strength change on pulse velocity measurements when passing through dry cured specimen. Further assessment on the influence of curing conditions on the pulse velocity/compressive strength relationship was made on cubes cured under dry conditions similar to that used for the large scale beams. Different types of concretes were considered and one typical mix similar to that used in the large scale beam (see Table 4.6) was examined in each case. Pulse velocity along with compressive strength measurements were taken at various ages from 1 to 28 days and the results are presented in Table 6.4. An exponential curve model was used to correlate pulse velocity to compressive strength and the summary of regression analyses together with coefficients of correlation are presented in Table 6.5. The results indicate that for each type of concrete, a higher pulse velocity reading would result compared to that obtained under dry curing. Figures 6.21 and 6.22 compare the pulse velocity/compressive strength relationship for two types of concrete, Leca and Pellite under the three different curing conditions. It suggests that for the two types of dry curing, separate relationships exist, although in Leca concrete the effect of dry curing is less significant. This is likely to be due to the high reservoirs of water which exist inside Leca.

The test results given in the Figures 6.17 to 6.20 suggest that in lightweight concretes under wet curing condition, the pulse velocity measurements are less dependent on mix proportion of concrete and this is most clearly seen in concretes made with Lytag. This phenomenon is believed to be related to the water absorption of aggregate. In fact aggregate occupies at least three-quarters of concrete volume and the lightweight aggregate with its high porosity absorbs a large amount of water. As a consequence the voids in aggregates will be filled up with water and this may mask the influence of mix proportions on pulse velocity readings.

A limited number of pulse velocity measurements along with compressive strength tests were carried out on fully Lytag concrete in the long term at ages of 180 and 360 days and the measured cube strength were compared with calculated cube strengths from short term pulse velocity/compressive strength relationships. These are shown in Figure 6.23. It is shown that under wet curing the short term calibration may be used for long term wet cured compressive strength predictions. The maximum percentage of error was found to be 18% for the low strength mix and 11% for the high strength mix. However under dry curing condition, because of significant drop in long term pulse velocity measurement, the percentage of error was quite high (above 70%) and it is thus considered inappropriate to use a strength/pulse velocity relationship developed in short term for long term strength assessment since the drying out effect may be misleading.

It has also been found that in the long term under dry curing conditions the pulse velocity development in massive concrete elements, such as large scale beams used in this investigation, was different from that in cube samples. An increase in pulse velocity was obtained when sending the pulse wave through the width of the element tested at 180 days (see Tables D-1 and D-11 in Appendix D). This phenomenon may be explained by the available moisture content in the interior body of concrete element whereas thin specimens such as standard cube samples reach a uniform humidity distribution in a shorter period. Bazant et al (1973) stated that massive concrete elements do not reach uniform humidity distribution even after decades. This would lead to the conclusion that in long term strength assessments, a pulse velocity/compressive strength relationship established on concrete cube specimens with similar dry curing as concrete in an element, even with the same maturity, may not be used with a high degree of accuracy to predict the strength of concrete in field structures. Therefore, any such relationship should be determined on specimens taken from the structures for a more reliable insitu strength prediction.

The influence of aggregate type on the pulse velocity/compressive strength relationship has already been demonstrated in Chapter 3 (see Section 3.2.2.2.1(b)). Here, an attempt has been made to compare the effects of a number of lightweight aggregates on different types of lightweight concrete and to compare them with normal weight concrete. The influence of sand replacement on pulse velocity measurements is also considered and results are presented graphically by comparing the results of semi Lytag with fully Lytag in Figure 6.24. Before any further discussion, it should be emphasized that due to the complexity of pulse velocity measurement as influenced by so many factors, a precise indication on the degree of influence of each type of lightweight aggregate may not be possible. As a consequence the numerical values given in Figure 6.24 may be valid only for the given mix, nevertheless a general indication may be true for other mix proportions.

Figure 6.24 shows the range of pulse velocity for a given compressive strength from 20 to 50N/mm² for each type of lightweight concrete with the exception of Leca concrete where the variation in pulse velocity was examined up to its ceiling strength of 30N/mm². Three interesting points are brought out by the results in Figure 6.24 and will be discussed below.

The test results indicate that for different types of lightweight concrete, a wide range of pulse velocity readings would exist. In all cases it can be noted from Table 6.4 that pulse velocities are significantly lower than expected with normal weight concrete of comparable strengths. In comparison between three types of semi lightweight concrete, Figure 6.24 suggests that concretes can be arranged in the following order of increasing pulse velocity; Leca, Lytag and Pellite. Also shown in Figure 6.24 is that the change in pulse velocity from upper to lower limits of the chosen strength range is similar for different types of concrete with the exception of Leca concrete where this change is slightly higher. This may be explained by the need for a greater range of cement contents required for Leca concrete.

Sand replacement for fine lightweight aggregate showed an increase in pulse velocity. The data obtained on Lytag concretes indicated that the pulse velocity measurements on semi Lytag concrete is higher by 7% as compared to fully Lytag concrete. This percentage increase was found to be the same at different strength levels.

For two types of concrete, namely fully Lytag and Leca concretes, Figure 6.24 shows that the pulse velocity results are very close together and this is somehow in conflict with the phenomenon of the influence of the aggregate type on the pulse velocity reading. However it is important to state that apart from aggregate type, cement paste quality has a significant effect on the pulse velocity and this factor was discussed in Chapter 3. As a consequence a larger cement requirement with relatively lower water/cement ratio, linked with sand replacement for fine lightweight aggregate in Leca concrete, might be the reason for the similar pulse velocity in fully Lytag and Leca concretes.

6.2.2.3 Within-Test Variation

The within-test variations for the various types of lightweight concrete are shown in Figures 6.25 to 6.28 along with coefficients of variation. Each of the computed variations were based on 12 observations. Coefficients of variation are relatively low compared to other test methods. With the exception of Leca concrete, they are also below 1.5% as suggested by Tomsett (1980) for normal weight concrete. Once again, a high variation in test results was obtained on Leca concrete and this is shown with a higher scatter of test results in Figure 6.27.

6.3 PARTIALLY DESTRUCTIVE TESTS

This section presents results of a study to assess the reliability of Windsor Probe, pull out, B.R.E. internal fracture, direct pull internal fracture and pull off methods. Generally all the results of a set have been averaged and individual values have only been omitted if there was some reason for doubting the validity of the result such as the failure of the adhesive in pull off tests. The influence of physical characteristics of lightweight aggregates upon test performance and strength measurement have been considered. Correlations between partially destructive test measurements and compressive strength are obtained using the simplest graph that gives a reasonable fit to the data.

6.3.1 Windsor Probe Test

The Windsor Probe tests were carried out on four mixes of fully Lytag concrete only under wet and dry curing conditions at the age of 7 and 28 days. At each stage, the exposed probe length was measured and the variability of three measurements analysed by considering the difference between the highest to the lowest reading. All were well below the maximum allowable difference of 5.08mm specified by the manufacturer.

6.3.1.1 Relationship between Exposed Probe Length and Compressive Strength

The results of the tests are shown in Table B-4. The compressive strength measurements given in this table were carried out on 100mm cube specimens. Regression analysis was undertaken to correlate exposed probe length to cube compressive strength using a simple linear regression. Table 6.6 gives a summary of regression lines along with the coefficients of correlation.

Relationships at age of 7 days are shown graphically in Figure 6.29. Based on the coefficients of correlation, better correlation is obtained for the wet concrete. From Figure 6.29, it can be seen that except for one data point (mix L-C at dry condition), one line may reasonably fit through all points and hence, it could be said that the relation is independent of curing condition.

Table B-5 shows the cube strengths derived from the probe-strength relationship given in the manufacturer's manual by applying suitable conversion factors. Initially, for a given exposed probe length the standard cylinder strength was computed by factoring the strength value given for Moh's hardness No.3 by 0.66 as recommended by the manufacturer for lightweight concretes with density below $1840kg/m^3$ (as was the case for fully Lytag concrete). Subsequent conversion to a cube strength was made using the relationship proposed by Munday and Dhir (1984) as;

$$f_c = 1.5 f_{cy} - 0.007 f^2_{cy} \qquad (6.3)$$

This makes allowance for the influence of the strength level on the specimen shape factor and is a refinement of the single factor of 1.25 recommended in BS 1881 : part 120 (1983). These strengths are compared with the actual 7 day cube strength and are graphically presented in Figure 6.30 for a given exposed probe length. This clearly shows the deficiencies of the calibration suggested by the manufacturer, which correctly predicted the actual cube strength only in a small region (about $17\text{-}22N/mm^2$) with maximum percentage of error of 7%. Outside this region, the actual cube strength prediction will be either over or under-estimated. Research was carried out on semi Lytag concrete by Swamy and Al-Hamed (1984a), who indicated that the manual predictions over-estimated the cube strength at all strength levels. From these two observations on two types of lightweight concrete, it thus seems that the manual calibration is unlikely to be satisfactory for strength prediction of lightweight concretes. Hence, calibrations would need to be established for each type of lightweight concrete as is necessary for normal weight concretes (see Section 3.3.1.1).

At the beginning of the test programme, the plan was to use standard power for mature concrete or concrete strength above 30N/mm^2 and low power for other cases as recommended by the manufacturer. For mix L-D, at the age of 7 days, the Windsor Probe test was carried out at standard power and resulted in a visible crack all around the beam (except the bottom face). The same also happened for the mature concrete (mix L-D). To overcome this problem, a reinforced concrete beam was substituted for the plain concrete beam. The same problem was also seen, but not to the same degree as previously. At this power level, the probe penetration at both ages was also too deep, so that it was very difficult to measure the exposed probe length by using the calibrated depth gauge. Therefore, for the remaining tests it was decided to use low power for all conditions. Test results in Table B-4 show that for dry conditions, for strength above 35N/mm^2, the Windsor Probe measurements would not follow the same pattern of increasing with the strength of concrete. This is linked to the mechanism of test which is quite complex as discussed later in Chapter 7. However a lower resistance of dry cured concrete against probe penetration may be related to tensile strength of concrete as shown in Section 5.3.1 where a drop in tensile strength may occur at high concrete strength levels under dry curing conditions.

It may be concluded from the above observation that since this type of beams cross section as well as dry curing is a reasonable simulation of site conditions, this test method may not be satisfactory for measuring the insitu cube strength of fully Lytag concrete in a mature stage under either low or standard power. However it might be worthwhile to develop an intermediate power by placing the probe/driving head assembly at a different distance into the driver barrel. Due to a limited time availability, this study had not been considered and further assessments of the Windsor Probe for other types of lightweight concrete were omitted.

For fully Lytag concrete, under low power, it was decided to find the best fitting line at age of 28 days. The wet cured specimen for the rich mix (L-D) was damaged during the test when it was initially planned to use standard power. Therefore, the regression equation for wet curing was not found due to limited number of data. A relationship for combined conditions (wet and dry cured) is shown in Figure 6.31 along with the 7 day relationship. Analysis continued to find the one best fitting graph for all conditions and this is shown in Figure 6.32 including the 95% confidence limits. The accuracy of strength estimation based on 95% confidence limit for strength level of $30N/mm^2$ was found to be ± 26%.

6.3.1.2 Comparison with Published Data

Figure 6.33 shows a comparison of the relationship obtained in this investigation for fully Lytag concrete with those using low power for semi Lytag (Moh's No.3) and normal weight (Moh's No.7) concretes obtained with other investigators

(Swamy and Al-Hammed, 1984a). It can be seen that significant differences exist between different types of concrete. Comparing fully and semi Lytag concretes, it clearly indicates that the sand replacement increases the resistance of concrete against the probe penetration.

6.3.1.3 Within-Test Variation

The variation in Windsor Probe test results was studied by firing 12 probes on fully Lytag concrete under low power. The results are shown in Figure 6.34. The dispersion of test results was calculated using the standard deviation and coefficient of variation and they were found to be 1.38mm and 2.5% respectively. The standard deviation given here was well below the value of 2.54mm specified in ASTM C803 (1982). Also the coefficient of variation found for fully Lytag concrete is well within the range reported for semi Lytag concrete by Swamy and Al-Hamed (1984a).

6.3.2 Pull Out Test

The results obtained from the Lok-Test with four different types of lightweight concrete will be discussed in this section. The influence of curing conditions as well as age were examined on fully Lytag concrete. The significance of aggregate characteristics on pull out resistance has been studied and compared with pull out resistance of normal weight concrete for a given compressive strength.

6.3.2.1 The Effect of Aggregate Characteristics on Concrete Testing

In this test method, as well as other partially destructive test methods, the lightweight aggregates have been shown to have a significant effect on test results including the mode of failure. For concretes made with Lytag and Pellite, it was found that at low strengths aggregate-matrix bond failure predominated whilst at high strengths aggregate fractures were predominant. This characteristic was not the same for concrete made with Leca where due to weakness of aggregate the failure passed through the aggregates for all cases. Figure 6.35 shows some typical truncated cones of lightweight concretes which were completely extracted following testing showing the failure passing through the aggregate particles.

In Leca concrete the non-uniformity of aggregate distribution along with the surface zone nature of the test method has some influence on the test results. Tremendous differences in test results were found between bottom face and remaining faces (see Table B-6), due to different characteristics of these regions and the difference averaged 70%. Figure 6.36 shows typical truncated cones extructed from side and bottom faces of a Leca concrete specimen. The pure matrix at the bottom layer of concrete will explain the high pulling resistance value. Therefore, the test results relating to bottom faces may be misleading and they have not been included in the statistical analyses for concrete made with Leca. In other types of lightweight concrete only slightly higher pulling resistances were created at the bottom faces and this is to be expected due to more uniform aggregate distribution following compaction.

6.3.2.2 Relationship between Lok Force and Compressive Strength

For each type of lightweight concrete, averaged values of Lok force and compressive strength along with coefficients of variation are listed in Table B-6. The results were analysed using statistical regression to find the best fit graph for the data. When the data points were plotted it appeared that a linear correlation would be suitable in most cases. Previous research on the pull out method (Petersen, 1984), had also led to a linear calibration graph. It was, therefore, decided to use linear regression and the summary of these analyses are listed in Table 6-7. As the straight lines were chosen purely to provide the best fit for the data there was no guarantee that they would pass through the origin even though it was obvious that they should as indicated before (Section 3.3.2.3) since the pull out test is a measurement of static strength property of concrete. Nevertheless, it will be seen that correlation lines fit the data with a reasonable accuracy in the strength range under consideration by not forcing them through the origin. Furthermore, Petersen (1984) and Bungey (1989a) showed that the slope of relationship between Lok force and compressive strength at low strength range is different from the relatively higher strength range. Hence it may be sensible to use a linear regression. Figures 6.37 to 6.46 show the relationship between Lok force and cube (100mm) compressive strength for different types of lightweight concrete.

The significance of curing condition and age effect were studied on fully Lytag concrete. The relationships between Lok force and cube compressive strength were established under wet and dry curing conditions. The tests were carried out at the ages of 7 and 28 days, and 6 and 12 months and high correlations were obtained between Lok force and compressive strength. In the short term stage (up to 28 days) Figures 6.37 and 6.38 show the dependency of the Lok-Test upon curing regime. Based on the 7 day test calibration, the Lok-force at dry condition will over-estimate the wet cured cube strength and from the 28 day tests calibration, at dry condition, the Lok force prediction for wet cured cube strength above $35N/mm^2$ is over-estimated and below this strength level is under-estimated. The combination of 7 and 28 day test results however show less dependency of the Lok-Test to the curing regime (see Figure 6.39). Therefore, one calibration graph may be satisfactory for all conditions. Figure 6.40 shows the relationship between Lok force and cube strength for all conditions along with 95% confidence limits. The accuracy of a compressive strength results of $30N/mm^2$ for fully Lytag concrete using short term calibration was found to be ±17%.

Further investigation has also been undertaken to examine the correlations obtained in the long term, specifically 6 months and 1 year cured under wet and dry conditions. These calibrations are plotted in Figure 6.41 and once again clearly demonstrate the independency of the calibration with age. The combined correlations in short and long terms has been compared with short term correlation in Figure 6.42. This figure highlights the reliability of short term calibration for long term strength prediction.

The relationships between Lok force and compressive strength for other types of lightweight concrete have also been studied and Figures 6.43 to 6.45 illustrate the calibrations along with the 95% confidence limits. High correlation was obtained between Lok force and compressive strength. The accuracies of the compressive strength result at $30N/mm^2$ for semi Lytag and Pellite concretes were found to be ± 13% and ± 18% respectively, whilst the corresponding accuracy for Leca concrete was found to be ± 23% for a strength level of $20N/mm^2$.

The calibration graph drawn for different types of lightweight concrete in Figure 6.40 and Figures 6.43 to 6.45 have been shown together in Figure 6.46 for comparison along with the calibration for normal weight concrete recommended by the Lok-Test manufacturer as given by equation 3.2. The accuracy of this relationship has been examined by performing a limited number of tests on normal weight concrete at the age of one year. The test results are shown in Figure 6.46 from which it can be seen that the manufacturer's prediction on equivalent cube strength is in remarkably close agreement with the measured values.

As noted from Figure 6.46, although of the same general form, the relationships between Lok force and compressive strength for lightweight concretes are significantly different to that for normal weight concrete. The reduced pull out resistance achieved at a given strength level for lightweight concretes may be explained by the differences in failure mechanism, with little or no aggregate interlock occurring (see Section 7.3).

In Figure 6.46, among the different types of lightweight concrete, marginally higher pull out resistance was detected for semi lightweight concretes. This is somehow related to the failure mechanism of the pull out test which seems to be very complex, as reported by Krenchel and Shah (1985). However one possible reason for this might be linked to the improved physical behaviour of the concrete resulting from sand replacement as discussed earlier in Chapter 2. Among the semi lightweight concretes, the pull out resistance of concrete with Pellite is slightly higher than the others. This may be related to the aggregate characteristics which involve open surface pores into which a portion of mortar may penetrate. The consequent greater volume requirement of mortar may provide higher pulling resistance.

6.3.2.3 Within-Test Variability

The variations of test results on lightweight concretes were examined at an age of 28 days. Twelve Lok tests were carried out in each case on two specimens, with the exception of Leca concrete where fifteen Lok-Tests were performed on three specimens excluding the bottom faces which have already been indicated to be unrepresentative. Figures 6.47 to 6.50 show the variation of measured Lok force along with coefficients of variation of different types of lightweight concretes. The results show that the least variability was obtained for fully Lytag concrete which was lower than that reported for normal weight concrete (Bungey, 1987). With semi lightweight concretes using Lytag and Pellite, the results show that the variabilities are similar and are of the same order as obtained for normal weight concrete. In Leca concrete, as in previous test methods, high scatter was obtained for similar reasons despite the procedures outlined above.

6.3.3 Internal Fracture Test

The internal fracture tests (B.R.E. and direct pull) were performed on 150mm cube specimens according to the test programme given in Table 4.5. After the completion of tests, the same cubes were subjected to compression tests along the undamaged side faces of cubes. To see if any large differences in compressive strength arise due to the damaged area on the faces of the cubes, sound 150mm cubes were crushed for some mixes made from similar batches. The reduction in strength on damaged cubes at different strength levels for various lightweight concretes are shown in Figure 6.51. An average reduction of 4.3% was found throughout this investigation which is similar to the one reported elsewhere (Bungey, 1981a) for normal weight concrete. This factor was used to determine the true cube compressive strength. Tables B-7 and B-8 summarize the test results for B.R.E. and direct pull internal fracture tests. The pull out force for direct pull internal fracture is the conversion of gauge reading by means of calibration chart given in Figure 4.11. The test results for internal fracture are based on six readings with the exception of tests on Leca concrete where the average of four readings were given because of the unrepresentative nature of bottom faces of specimen as already described in the previous test method (Section 6.3.2.1). The compressive strength is the mean of four measurements on fully and semi Lytag concrete whereas on Leca and Pellite concretes it is the average of two readings.

6.3.3.1 Influence of Aggregate Properties on Test Behaviour

In all the tests in this investigation the loading was continued past the peak and complete failure was created in concrete with the exception of Leca concrete which will be discussed later on in this Section. In most cases an intact cone of concrete was pull out. Figures 6.52 and 6.53 show the mode of failure for two types of lightweight concrete at low and high strength levels. As in the Lok-Test, these figures indicate an aggregate-matrix bond failure at low strength level while at high strength level the failure was dominated by aggregate fracture.

The tests carried out on Leca concrete were mostly unsatisfactory. This is related to the mechanism of test as well as the aggregate characteristics. Recalling the test procedure, the first step involves drilling, leaving a hole with diameter equal to that of the drill. However, when the soft aggregate was encountered there was a tendency for the hole to become oversize at that point. Furthermore, when tensile load is applied to the anchor bolt this is mainly transferred from the expanded portion of the anchor bolt to the embedding material. After maximum load is reached, the anchorage typically fails by rupturing the concrete and the formation of 'pull out' concrete cone. However this behaviour was not found most of the time when testing on the side faces of Leca specimens. This is because when the expanded portion came into contact with Leca, the wedging forces exceeded the strength of the Leca, resulting in local crushing so that the Leca provided enough room for expansion of the clip and resulted in the pulling out of the center bolt part of the fixing through the outer clip with little or no fracturing of the concrete.

6.3.3.2 Relationship between Internal Fracture Test Measurements and Cube Compressive Strength

The correlation between parameters determined by internal fracture tests and cube compressive strength are shown in Figures 6.54 to 6.70. Regression analysis based on a power function was carried out for each set of data and these equations with coefficients of correlation are summarized in Tables 6.8 and 6.9.

The effect of curing regime was studied throughout the tests on fully Lytag concrete and in some instances on semi Lytag and Pellite concretes. Here the influence of curing regime are presented graphically for both test methods in Figures 6.54 to 6.57 for fully Lytag concrete. The relationships at age 7 and 28 days were found to have small dependency on the curing regime. This dependency was found to be greatest at 28 days due to differential moisture distributions (see Section 5.3.1) but less significant than the effect upon the tensile/compressive strength ratio. This may be attributed to the small zone of concrete under stress which will suffer less from moisture differential. Further analysis was also undertaken to study the effect of curing on the relationship between internal fracture test measurements and compressive strength by combining the test results under different conditions. It has been found that a relationship with fairly high correlation could be expected. Hence a single relationship was considered for practical purposes. Figures 6.58 to 6.63 show the calibration for B.R.E.

or direct pull internal fracture for fully and semi Lytag, and Pellite concretes. For Leca concrete, the relationship between direct pull internal fracture force and compressive strength was studied only under wet curing condition and this is given in Figure 6.64. A poor correlation was obtained for Leca concrete and this is because in most cases the maximum applied load has not reached the failure criteria of concrete as discussed above. The reliability of both test methods were assessed for each type of concrete using 95% confidence limits and they are presented in Table 6.10 at the strength level of 30N/mm^2 for all types of lightweight concrete with exception of Leca concrete where the corresponding limits were at the strength level of 20N/mm^2. From these results it is likely that the direct pull internal fracture test will give better strength estimation.

Long term measurement test parameters on fully Lytag concrete were obtained at 6 and 12 months and the results are shown in Figures 6.65 to 6.68. These figures show a relatively large scatter of data points around the mean curve and this was confirmed with relatively low coefficients of correlation as indicated in Tables 6.8 and 6.9. For direct pull internal fracture test a close agreement of the varying age correlation curves is shown in Figure 6.67 but for B.R.E. internal fracture test some small dependency on age effect is indicated in Figure 6.65. Further assessment on age effect was undertaken to compare the calibrations given in short and long terms and they are presented in Figures 6.66 and 6.68. The results obtained from this study indicate that the short term calibration for direct pull internal fracture test is reasonably reliable for long term strength prediction whereas for B.R.E. internal fracture it is likely that separate calibrations may be required for strength prediction.

Figures 6.69 and 6.70 compare graphically the calibrations for different types of concrete. It is clear from these figures that the failure loads for both internal fracture loading methods applied to lightweight concretes are reduced in comparison with normal weight concrete. The amount of reduction is related to the type of concrete as well as test loading method applied. Within different types of lightweight concrete, fully Lytag and Pellite concretes showed a similar correlation for direct pull loading whereas for B.R.E. loading a higher torque resistance was obtained for Pellite concrete. Sand replacement for fine lightweight aggregate on Lytag concrete showed a relatively higher ultimate load, however the degree of influence on both test methods is not the same. For direct pull a marginally higher pulling resistance was obtained for semi Lytag concrete (Figure 6.70) whilst for B.R.E. loading the influence was significant as shown in Figure 6.69. The ratio of pull out force to torque was examined at different strength levels for fully and semi Lytag concretes as shown in Figure 6.71 and 6.72. The average ratio was found to be 1.75 and 1.37 for fully and semi Lytag concretes respectively. The corresponding ratio was also measured for Pellite concrete and was found to be an average of 1.44 which is nearly the same as the value obtained for semi Lytag concrete and similar to the 1.4 reported for normal weight concrete (Bungey, 1981a). The different ratio for fully and semi lightweight concrete may be related to the influence of the fine aggregate. This may be explained by the fact as described in Section 3.3.3.1 that in B.R.E. internal fracture test some torsion would be created and possibly natural sand may provide higher torsion resistance as compared to lightweight fines. This might also explain the different behaviour of both tests on Pellite concrete when compared to fully Lytag concrete. For direct pull tests, another interesting factor brought out by Figure 6.70 is that for all types of lightweight concrete, with the exception of Leca concrete, a similar behaviour is obtained as that given for correlation between tensile splitting and compressive strengths (see Figure 5.15). This phenomenon may indicate a tension failure for the direct pull internal fracture test, and will also be shown analytically in Section 7.4 using basic engineering mechanics.

6.3.3.3 Variability of Internal Fracture Tests

The variability of B.R.E. and direct pull internal fracture tests obtained with different types of lightweight concrete are shown in Figure 6.73 to 6.79. Each of the computed variations were based on sets of 12 readings and are presented in terms of the coefficient of variation. Comparing the coefficients of variation, it can be seen that these values for different types of lightweight concrete, with the exception of Leca, are significantly lower than those anticipated for normal weight concrete (Bungey and Madandoust, 1989; see Appendix E). Considering the different types of lightweight concrete, the lowest variation was obtained for fully Lytag and the highest for Leca concretes.

For semi lightweight concretes, there is a sign of increase in variation for B.R.E. internal fracture as opposed to direct pull internal fracture test and this was also confirmed by Bungey (1981a) for normal weight concrete. However in this investigation and elsewhere similar variations have been obtained for both test methods applied to both fully Lytag and normal weight concretes (Bungey and Madandoust, 1989; see Appendix E).

6.3.4 Pull Off Test

Pull off tests were performed on different types of concrete by means of surface and partial cored pull off methods (see Section 4.6(a)). The tests were generally performed on 150mm cube specimens throughout the investigation, although some 100mm cube specimens were also used in some instances for further investigation on possible dependency of pull off strength on specimen size which was suggested by Murray's work (1984).

50mm pull off disks were used throughout the investigation with the exception of four cases where 40mm disks were used for two mixes of fully Lytag concrete (L-C and L-D) at the age of 180 days and 360 days when concretes were gaining high strength. According to variational analysis carried out by Murray (1984) on normal weight concrete using 10mm aggregate, only slight increase in variation of test results was observed in case of 40mm disk as compared to 50mm disk. Hence this slight increase in variation may not be significant for the purposes of analysis.

After the completion of surface pull off tests on 150mm cubes, the same cubes were used for compression tests across two undamaged moulded faces. In some cases the compression test was also performed on sound cubes and comparison of the results indicates that the pull off tests did not cause any significant reduction in the compressive strength of 150mm cubes. Those cubes where the partial cored pull off tests were performed have not been used for measuring the compressive strength of concrete. This is because of the relatively large quantity of concrete removed and the effect of possible stress concentrations reducing the measured concrete strength. The partial cored pull off tests were correlated to compressive strength measured on undamaged 150mm cube specimens.

For both surface and partial cored pull off tests, the results have been presented in terms of nominal pull off strength. In the case of the surface pull off tests the area of the disk was used and for the case of the partial cored pull off tests the cross sectional area of the core was used. The test results are summarized in Tables B-9 and B-10 and represent the averages of three cube strengths and six pull off strengths.

It is essential for the success of these tests that the adhesive strength created by the resin as interconnection between aluminium disk and concrete specimen should be greater than the tensile strength of the concrete. Since most of the plastic adhesives available on the market have a strength well above that of concrete, this requirement really comes down to the quality of the preparation of the concrete surface and the application of the adhesive. In a search for a more suitable adhesive, a series of tests using three different glues were performed. These were

Ordinary Araldite epoxy resin, Febset 'Nonflow' and Devcon 5 minute epoxy resin. It was concluded that the Devcon 5 minute epoxy resin is the most suitable of the glues studied. Its double syringe container made mixing easy and the bond strength was found to be sufficient for testing after about 1½ hours. This glue was used satisfactorily throughout the investigation with the exception of few tests where full or partial debonding occurred and these are indicated in Table B-9 where they were not considered in the calculations.

As glueing forms part of preparation, it is required for the pull off test to be performed in a completely dry condition for effective adhesive performance. It is therefore a limitation of this test that it can be used in a dry condition only. Murray (1984) showed the performance of pull off test on normal weight concrete under wet-dry conditions. In his investigation, the wet cured specimens were removed from the tanks 48 hours before the pull off tests could be performed to allow for drying and for the adhesive to cure sufficiently. Similar procedures were also examined in this investigation for lightweight concrete. It was observed that because of high water absorption more time would be required for allowing the specimen to be dried. Therefore, this would not be feasible and only dry curing regime was adopted throughout the test programme. For partial cored pull off tests where water is required for lubricating, the drilling was undertaken two weeks before performing the tests (at age of 14 days).

In fully Lytag concrete, at age of 7 days it was observed that the natural indoor dry condition was not enough for a satisfactory test and full or partial debonding resulted. To overcome this problem a heat lamp was applied for those cubes to be tested at 7 days. The heat lamp was applied for 24 hours just before the test. However, this facility may not be available for a site condition. Moreover the test treated in this way may overestimate the insitu strength of concrete where the heat applied at early age can speed the hydration and would create higher pull off and compressive strengths as experienced through this investigation. For semi Lytag, Pellite and normal weight concretes where the tests were carried out at the age of 7 days, no such problem was found. It is thus advisable not to use the test at an early age (7 days) for fully Lytag concrete or similar types of concrete.

6.3.4.1 Influence of Aggregate Characteristics on Failure Mode

For different types of concrete close attention was paid to the failure process. For low strength lightweight concretes, with the exception of Leca concrete, it was mostly a bond failure between mortar and aggregate. This was due principally to the lesser quality of the cement and the high porosity. However for high strength concrete, the process was a combination of failure through aggregate as well as bond failure between mortar and aggregate. For normal weight concrete at all strength levels the failure was mostly a bond failure between mortar and aggregate, whereas in Leca concrete for all cases the failure passed through the aggregates. Figures 6.80 and 6.81 show the typical mode of failure for low and high strengths of semi Lytag concrete.

In respect to the depth of failure surface, study shows that the depth is larger for low strength concrete compared to high strength and overall, the depth of failure surface in the lightweight concrete was larger compared to the normal weight concrete. The measured failure depth covering different types of concrete was ranged from 2mm to 7mm.

Figure 6.82a shows typical failures of aluminium disks pulled off with their concrete residues for lightweight concrete. The first two right hand pictures are more likely to be the case, although it has been observed that this has no significant effect on the measured failure force.

The non-uniform distribution of Leca aggregate in the matrix has again been observed through this test and is shown in Figure 6.83. Apart from tests carried out on the side faces of cubes, a limited number of tests have also been performed on bottom faces of Leca cube specimens. A significantly higher pull off resistance was created as a result of pure matrix at the bottom layer of concrete specimen and the results corresponding to bottom faces were not considered in the overall analysis. The non-uniform distribution of Leca has however also influenced readings taken from side faces of specimens giving a high scatter as will be shown later in Section 6.3.4.4.

In surface pull off tests, the area of concrete removed by each test was always greater than the area of the pull off disk itself. This is known as overbreaking. The amount of overbreaking varied approximately from 1mm to 8mm. In lightweight concrete overbreaking was generally found to be larger, with Leca concretes exhibiting the largest overbreaking. Overbreaking results in an irregular and undefined area value for calculating the pull off strength. However the pull off strength is of little use on its own and therefore it might be sufficient to calculate this parameter on the basis of pulling force acting over the area of disk.

6.3.4.2 Effect of Specimen Size

The influence of specimen size on pull off strength has been brought out in work done by Murray (1984). It has been indicated that pull off strengths on 100mm cubes are higher than those for 150mm cubes by about 10%. This is unexpected as indicated by Murray (1984) . In fact it is believed that concrete under pull off test (similarly in other partially destructive tests) is subjected under a localized stress which is unlikely to be dependent on sample size provided sufficient edge distances are provided. This is recommended to be one diameter of disk from center of test position (BS 1881 : part 207 (1991)). Nevertheless, further investigations were undertaken by the Author to clarify this phenomenon, Figure 6.84 shows the results of pull off tests on 100mm cube specimens plotted against the pull off strength on 150mm cube specimens performed on a number of concretes. The results give a straight line relationship with a correlation coefficient of 0.99. On average the pull off strengths on 100mm cube specimens were not significantly different than the pull off strengths on 150mm cubes. The analysis of the results shows the tests on two different specimen sizes not to be statistically significant at the 5% level. Study of stress analysis on these two specimen sizes, as it will be shown later in Chapter 7, also indicates the lack of significance of specimen size on pull off strength.

6.3.4.3 Relationship between Pull Off Strength and Cube Compressive Strength

a) Surface pull off tests

The relationships between pull off strength and 150mm cube compressive strengths for different types of concrete are shown in Figures 6.85 to 6.96. Regression analysis was carried out using a power function for each set of data and these equations with coefficients of correlation are summarized in Table 6.11.

The results were separated in terms of age where the short term and long term performance were considered. In the short term, age dependency on the relationship was investigated on fully and semi Lytag, and normal weight concretes. On fully Lytag concrete, a discrepancy is shown between the two relationships for 7 and 28 day tests (Figure 6.85). The 28 day relationship resulted in a lower pull off strength when compared to 7 day relationship. The possible reason for lower pull off strength at 28 days may be due to differential moisture content as previously shown for concrete under the splitting tensile tests (Section 5.3.1). Slightly discrepancy was also found for semi Lytag and normal weight concretes where generally a lower pull off strength was obtained for 28 day old concrete (Figures 6.86 and 6.87). However based on practical strength ranges, the test results show that a single relationship might be suitable for these cases and these are shown in Figures 6.88 to 6.92 along with 95% confidence limits for different types of concrete. 95% confidence limits for strength level of 30N/mm^2 for all concretes except Leca for strength level of 20N/mm^2 are summarized in Table 6.12. From this, it seems that there is similar accuracy for assessing equivalent cube strength of the different types of lightweight concretes.

Long term assessment on the test performance was carried out on fully Lytag and normal weight concretes at the age of 180 and 360 days and also on Leca concrete where the tests were performed at the age of 360 days only. The relationships obtained on these concretes are shown in Figures 6.93 to 6.95 along with the relationships obtained in the short term study. From these figures a similarity in relationships obtained in short term and long term can be seen. However from the long term relationship there is a slight increase in pull off strength of concrete. Similar behaviour was also observed when the tensile splitting test was carried out on concrete (see Section 5.3.1).

Comparing the relationships established (in the short term stage) for different types of concrete (Figure 6.96) indicates that the pull off strength for a given compressive strength varies from one type of concrete to another. The measured pull off strength on normal weight concrete was found to be lower as compared to lightweight concretes. This behaviour was initially assumed to possibly be due to high porosity of lightweight aggregates which may permit deeper adhesive penetration below the surface of concrete, and hence increased pull off strength (Bungey and Madandoust, 1989, 1989a; see Appendix E). However later work on computation of stress distribution (see Section 7.5.1.1) indicates the significancy of concrete stiffness on pull off resistance, where a lower stiffness results in a higher pull off strength, which reasonably agrees with the experimental finding. Within different types of lightweight concrete, the significance of concrete stiffness is clearly indicated with the exception of fully Lytag concrete at high strength level above $37N/mm^2$ or low strength level below $20N/mm^2$. This discrepancy is somehow related to other factors which seem to have more influence on fully Lytag strength behaviour. For instance, weakening in pull off resistance at high strength level is possibly related to additional stress due to high differential moisture content. On the other hand, increased pull off resistance at low strength level may be due to the earlier assumption of glue penetration.

b) Partial cored pull off tests

Figures 6.97 to 6.100 show pull off strength plotted against cube compressive strength for partial cored pull off tests with 20mm core length on lightweight concretes at the age of 28 days along with the corresponding relationship for surface tests for comparison. Power fit curves were modelled to correlate pull off strength to cube compressive strength and the regression analyses are given in Table 6.13. From the above figures, it is clearly seen that the partial coring of specimens tends to reduce the pull off strength. This observation was also made by Stehno and Mall (1977), and Keiller (1985). The possible reasons for this reduction may be outlined as follows:

1. In surface pull off tests, because of overbreaking when pulling off the testing disk, border zones of the area participate in stress carrying, thus producing an apparently higher pull off strength. In partial cored pull off tests, the existence of a defined area and the absence of edge stress development will result in a lower pull off strength when compared to surface pull off tests.

2. The stress concentration created at the interfacial area between cored concrete and the remaining body of concrete (see Section 7.5.1.2) will further reduce the pull off strength in partial cored pull off tests.

3. The coring action would tend to weaken the concrete mainly by vibrating the aggregates and affecting matrix-aggregate interface.

4. Increasing the core length creates a lower pull off strength. This has also been shown in stress analysis as will be discussed later in Section 7.5.1.2.

The influence of the possible weakening of both aggregate and matrix-aggregate bonds on the pull off strength was assessed on two types of concrete, namely fully Lytag and normal weight concretes. The study has been made to compare the measured pull off strength obtained from 20mm long drilled cores with 20mm long formed cores. In the case of the formed cores, a split former as shown in Figure 6.101 was designed to provide the formed pull off area. The split former was screwed to the base plate of a 150mm cube mould before concreting and was removed from the concrete at the age of 3 days when the concrete had gained about 50% of its 28 day strength. The base plate position was chosen in preference to the side of the mould because of the difficulty in ensuring good compaction around and inside the split former which may weaken the concrete and hence give low pull off strengths.

For fully Lytag concrete, mix No. L-C was used and in the case of normal weight concrete mix No. N-4 was used modified by using only 10mm aggregate. A series of 12 tests were performed on each type of concrete, 6 drilled cores and 6 formed cores.

For normal weight concrete, pull off tests showed that drilled cores gave 10% lower value of pull off strength than formed cores. However, fully Lytag concrete exhibited no difference in pull off strength between the two types of core.

From the correlations obtained for surface and partial cored pull off tests (see Figures 6.97 to 6.100), the results showed that the amount of reduction of pull off strength in partial cored pull off tests is dependent on the type of concrete. This phenomenon was clarified by means of theoretical analysis as will be discussed in Section 7.5.1.2.

To determine the effect of core length on pull off strength, 3 different lengths were chosen, namely, 5, 20 and 50mm. The concrete chosen for these tests was fully Lytag with a 28 day strength of $40N/mm^2$. The pull off strength of each of the 3 core lengths was compared with the pull off strength for surface tests. It was found that 5mm core length gave a reduction in pull off strength of 23% with 20mm and 50mm core lengths giving reduction of 32% and 33% respectively. From these results it seems that the reduction in pull off strength does not significantly alter with core lengths above 20mm. Figure 6.82(b) shows an example of an aluminium disk along with partial core pulled away from concrete at the core base. However, this was not always the case as the point of separation varied along the core length. Fracture mostly occurred however either at the interface between partial core and remaining body of concrete, or beneath the aluminium disk where the stress was high (see Section 7.5.1.2).

6.3.4.4 Within-Test Variation

To examine test variability, a series of twelve readings were collected on several different types of lightweight concrete for the surface pull off tests. Scatter of test results for the partial cored pull off tests were also taken into account when the tests were carried out on fully Lytag concrete with the same number of readings as in the surface pull off tests. The variation of test results together with mean pull off strengths and coefficients of variation are indicated in Figures 6.102 to 6.106.

Regarding surface pull off tests, the lowest variability was obtained with fully Lytag with the variation below that for normal weight concrete which is reported to be 8% (Murray and Long, 1987). With semi lightweight concrete, the data shows that the variabilities are increased. For semi Lytag and Pellite concretes, the variations are of the same order as obtained for normal weight concrete where for Leca concrete, the variation was significantly higher.

In fully Lytag concrete, Figures 6.102 and 6.106 illustrate that variation in pull off strength obtained from surface and partial cored pull off tests are reasonably similar. A higher scatter of variation on partial cored pull off test has however been reported for normal weight concrete (Maxwell, 1977). This high variation is possibly related to damaging of aggregates and matrix-aggregate bonds under drilling action.

6.4 DESTRUCTIVE TESTS

6.4.1 Core Test

50mm diameter cores with different ℓ/d ratios were drilled from different types of lightweight concrete as illustrated in Table 4.5 and tested at the age of 31 days. A limited number of cores were also tested at the age of 360 days on fully Lytag concrete only. The influence of curing conditions were assessed on core strength measurements for two types of concrete namely fully Lytag and Pellite concretes whereas other types of lightweight concrete wet curing were used only. As previously mentioned in Section 4.6(a), those cores which were drilled from dry cured concrete were tested dry to simulate similar moisture content as in concrete elements. Prior to testing, all cores were visually compared to Figure 1 of BS 1881 : part 120 (1983) and the estimated excess voidage of each was observed to be less than 0.5% except for Leca concrete where it was likely to be 1.5%. Tables B-11 and B-12 summarize the average of three cores for a specified ℓ/d ratio.

6.4.1.1 Effect of Length/Diameter Ratio (ℓ/d)

The influence of ℓ/d ratio on core strength was assessed on fully Lytag concrete with ℓ/d = 1.0, 1.4, 1.6 and 2.0 for all mixes whereas in other types of lightweight concrete ℓ/d ratio of 1.0 and 2.0 were considered for only two mixes with low and high strength levels.

From Table B-11, as expected, core strengths were generally found to increase with decreasing ℓ/d ratio, although for dry cores from fully Lytag concrete the effect was not as large and not always as consistent as anticipated. This may be due to lack of uniformity in moisture content resulting from air drying and this corresponds to relatively higher variability in test results as will be shown in Section 6.4.1.4. Hence this may emphasize the importance of use of standardized specimens soaked for at least 48 hours.

In order to obtain corrected strengths, the measured core strengths for different ℓ/d ratios were expressed in terms of the percentage of the corresponding value for ℓ/d = 2.0 and these correction factors are shown in Table 6.14 for different types of lightweight concrete. The results show that the correction factors are variable and depend upon a number of factors which are discussed below. The ℓ/d correction factor depends on type of concrete. However in concretes made with Lytag, the available data for wet curing condition indicate no significant change in correction factors between fully and semi Lytag concretes.

Similarly under dry curing conditions, the published correction factor on semi Lytag concrete (Swamy and Al-Hamed, 1984b) was shown to be reasonbly close to the corresponding values for fully Lytag concrete obtained in this investigation. Considering curing condition, it appears that dry curing generally requires less correction, and a similar observation was also reported for normal weight concrete (Swamy and Ali, 1984). Munday and Dhir (1984) indicated that concrete strength is important in determining the correction factors for cores having low ℓ/d ratios; in particular, low strength concretes require greater correction than do higher strength concretes. For this investigation, the limited results show a fairly consistent trend in respect of this effect for different ℓ/d ratios on lightweight concretes (see Table 6.14). However for fully Lytag concrete under dry curing this effect is not clearly defined and it would be prudent to keep the ℓ/d ratio as close to 2.0 as possible.

To give some comparison of the correction factors obtained from these tests with the data for small cores of normal weight concrete reported by Bungey (1979), a single set of average correction factors were adopted and they are presented in Table 6.15. It appears that lightweight concretes need considerably less correction than normal weight concrete. A similar observation was also indicated by Kesler (1959) and Swamy and Al-Hamed (1984b). Recommended correction factors according to A.S.T.M. C42 (1984) and BS 1881 : part 120 (1983) are also included in Table 6.15 and it can be seen that widely accepted British Standard values over-estimate those required, even for wet specimens of lightweight concrete. The effect of ℓ/d ratios in all lightweight concretes was similar to suggestions given by A.S.T.M. Standard, except for Pellite concrete under wet curing condition which more closely agreed with the British Standard.

6.4.1.2 Effect of Direction of Drilling

A limited number of horizontal cores are shown plotted against vertical core strength in Figure 6.107. The cores corresponding to both directions were drilled from the same concrete at the age of 31 or 360 days under either wet or dry curing conditions. From the above figure, it is clearly shown that the direction in which the cores were drilled in relation to the direction of placement of concrete made no significant difference to measured core strength. Although this might be a preliminary indication of independency of core strength of lightweight concrete with respect to the direction of core drilling, further experiments would be required to clarify this on a variety of types of lightweight concrete. In normal weight concrete, as already mentioned in Section 3.4.1.1(c), there are some contradictions in the published data. BS 1881 : part 120 (1983) applies a correction to the measured core strength to allow for the effect of core orientation (see Section 3.4.1.1(c)). A.S.T.M. C42 (1984) however requires that the direction of loading of the core with respect to the horizontal plane of the concrete as placed should be stated in reporting the strength results, but recommends no conversion factor.

6.4.1.3 Relationship between Core and Cube Strengths

Figures 6.108 to 6.113 show the relationship between core strength with $\ell/d = 2.0$ and 100mm cube compressive strength.

Regression analysis based on coefficient of correlation showed a linear relationship for fully and semi Lytag concrete whereas a power fit curve was found for Pellite and Leca concretes. The latter type of curve was selected for all types of lightweight concrete as the curve passes through the origin and satisfied the logical requirement. The equations of these correlation curves are given in Table 6.16.

Figures 6.108 and 6.109 show the typical relationships between core and cube strengths for fully Lytag and Pellite concretes at the age of 31 days under wet and dry curing conditions. The relationships were found to be independent of curing conditions and a single curve may be drawn through data points as shown in Figures 6.110 and 6.111. 95% confidence limits at the strength level of 30N/mm^2 were found to be ± 12% and ± 11% on fully Lytag and Pellite concretes respectively. Additionally a limited number of tests were carried out on fully Lytag concrete at the ages of 180 and 360 days covering two mixes with low and high strength levels and they are shown in Figure 6.112 along with short term calibration. From the figure it is apparent that the short term calibration fitted the data well with the exception of those results obtained from low strength concrete under dry curing conditions. At low strength level, the lower cube strength as compared to core strength may be explained due to the smaller volume of the concrete cube specimens coupled with the relatively higher porosity of the low strength concrete. These features make the concrete dry out quickly and hence interrupt the hydration of the cube strength specimens leading to a lower strength than the larger blocks from which cores were cut.

Figure 6.113 compares the given correlations for different types of lightweight concrete. For fully and semi Lytag concretes, it seems that a good agreement exists between the relationships whereas in other types of lightweight concrete different relationships were found.

The strengths of cores, as expected, were shown to be lower than the cube strength and the amount of reduction was found to be different from one type of concrete to another. At the cube strength of $25N/mm^2$, the strength reduction in core strength with $\ell/d = 2.0$ ranged from 5% to 44% where the lower and upper limits correspond to fully Lytag and Leca concretes respectively. The high reduction in core strength of Leca concrete is probably linked to the effect of the drilling action damaging the weak Leca aggregate. Another factor is the non-uniform distribution of Leca particles. This non-uniform distribution of Leca aggregate may cause the core to be subjected to a bending moment as opposed to a pure normal load due to the difference in stiffness between aggregate and matrix.

The relationship between equivalent cube strength (obtained from cores with $\ell/d = 2.0$) and the measured cube strength from different types of lightweight concrete is shown in Figure 6.114. the core strengths were corrected by applying the correction factor of 1.15 as recommended by BS 1881 : part 120 (1983). The results show that the British Standard correction factor under-estimates the cube strength of Leca concrete as much as nearly 40%. In other types of lightweight concrete, the results are gathered around the line of equality with excellent agreement found with semi Lytag concrete.

6.4.1.4 Within-Test Variability

The variability of the core test results on fully Lytag and Leca concretes are presented graphically in Figures 6.115 to 6.119 along with coefficients of variation. Fully Lytag concrete showed less variability than Leca concrete. Also, when compared to 7% coefficient of variation of small core on normal weight concrete (Yip and Tam, 1988), fully Lytag concrete showed less variation in test results.

The effect of curing conditions on the variability of core strength was assessed on fully Lytag concrete. The dry cured cores showed higher variation. This is possibly related to non-uniform moisture content as was mentioned in Section 6.4.1.1.

The influence of ℓ/d ratio on the variability of the core tests was investigated on fully Lytag concrete at two extreme ratios ($\ell/d = 1.0$ and $\ell/d = 2.0$). The results as indicated in Figures 6.115 to 6.118 show no significant change in variability of results. A similar observation was also reported elsewhere on normal weight concrete (Bungey, 1979).

6.5 COMPARISON OF INSITU TEST METHODS

From the results presented in the previous sections, the reliability of three categories of insitu test methods (non-destructive, partially destructive and destructive test methods) were examined for the purpose of insitu strength assessment. Among these test methods, partially destructive and destructive test methods seem to be more reliable for strength assessment as they suffer less from the influence of mix proportions, curing regime and surface condition.

Comparison of core test results with partially destructive test results showed the greater reliability of core tests for measuring the equivalent cube strength. The accuracy of strength measurement based on 95% confidence limit at strength level of $30N/mm^2$ was found to be around ± 10%.

Table 6.17 compares the reliability of different partially destructive test methods on lightweight concretes by means of coefficient of variation and strength accuracy measurement based on 95% confidence limits for strength level of $30N/mm^2$ except for Leca which considered for strength level of $20N/mm^2$. These values have been compared with those obtained for normal weight concrete (see Table 6.17) and show a greater reliability of these tests for lightweight concretes than for normal weight concrete with the exception of Leca concrete, for which the tests are less reliable. Windsor Probe test results are not included in the table due to the difficulty encountered during the testing on fully Lytag concrete (see Section 6.3.1.1). From the results shown in Table 6.17, it seems that the pull out and pull off tests provide the most promising approach for assessing the insitu equivalent cube strength.

Table 6.1:Regression analysis for the relationship between rebound number and compressive strength

Type of Concrete	Mix No.	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	L-C	$f_c=0.017\ R^{2.15}$	0.993
Semi Lytag Concrete	L-3	$f_c=0.024\ R^{2.10}$	0.994
Leca Concrete	Le-4	$f_c=2.01\ R^{0.82}$	0.938
Pellite Concrete	P-5	$f_c=0.007\ R^{2.53}$	0.994
Normal Weight Concrete	N-4	$f_c=0.031\ R^{2.04}$	0.995

Table 6.2:Accuracy of strength measurements from rebound hammer tests

Concrete Type	95 % Confidence Limit on Estimated Strength
Fully Lytag Concrete	±18 %
Semi Lytag Concrete	±15 %
Leca Concrete	±23 %
Pellite Concrete	±15 %
Normal Weight Concrete	±12 %

Table 6.3:Regression analysis for the relationship between pulse velocity and compressive strength

Type of Concrete	Mix No.	Curing Condition	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	L-A	Wet	$f_c=2.1x10^{-3}\ e^{2.66V}$	0.976
		Dry	$f_c=2.0x10^{-6}\ e^{4.87V}$	0.574
	L-D	Wet	$f_c=1.6x10^{-3}\ e^{2.72V}$	0.975
		Dry	$f_c=1.2x10^{-5}\ e^{4.10V}$	0.766
Semi Lytag Concrete	L-1	Wet	$f_c=2.8x10^{-3}\ e^{2.42V}$	0.9996
		Dry	$f_c=2.0x10^{-6}\ e^{4.90V}$	0.9999
	L-4	Wet	$f_c=1.3x10^{-2}\ e^{2.02V}$	0.988
		Dry	$f_c=7.2x10^{-3}\ e^{2.21V}$	0.967
Leca Concrete	Le-1	Wet	$f_c=5.9x10^{-2}\ e^{1.67V}$	0.991
		Dry	$f_c=1.5x10^{-2}\ e^{2.22V}$	0.952
	Le-4	Wet	$f_c=1.18\ e^{0.86V}$	0.941
		Dry	$f_c=1.08\ e^{0.92V}$	0.813
Pellite Concrete	P-1	Wet	$f_c=1.5x10^{-3}\ e^{2.33V}$	0.986
		Dry	$f_c=1.6x10^{-5}\ e^{3.70V}$	0.947
	P-5	Wet	$f_c=2.1x10^{-2}\ e^{1.72V}$	0.997
		Dry	$f_c=9.6x10^{-4}\ e^{2.54V}$	0.991

Table 6.4:Pulse velocity test results under dry curing as used in beam

Type of Concrete	Mix No.	Age Days	Pulse Velocity Km/sec		Cube Compressive Strength N/mm^2	
			V	c.v (%)	f_c	c.v (%)
Fully Lytag Concrete	L-C	1	3.23	0.18	11.1	4.1
		2	3.38	0.17	17.0	2.7
		3	3.44	0.73	20.7	2.7
		5	3.50	0.66	24.0	3.5
		7	3.52	0.16	27.4	2.6
		14	3.58	0.56	34.0	2.7
		21	3.60	0.73	42.1	2.4
		28	3.60	0.89	46.1	1.5
Semi Lytag Concrete	L-3	1	3.53	2.14	13.1	1.2
		2	3.63	0.64	16.6	6.1
		3	3.69	0.81	20.4	2.7
		5	3.82	1.29	27.0	8.5
		7	3.85	0.98	31.5	3.1
		14	3.89	0.90	39.3	2.5
		21	3.91	0.77	41.3	2.7
		28	3.88	1.72	42.9	3.6
Leca Concrete	Le-4	1	3.10	3.07	15.9	5.8
		2	3.18	0.75	20.8	4.9
		3	3.31	4.60	21.3	13.3
		5	3.44	2.30	23.5	5.3
		7	3.58	2.44	28.0	8.0
		14	3.65	5.71	29.7	17.9
		21	3.58	3.06	29.2	5.0
		28	3.48	0.59	31.4	13.4
Pellite Concrete	P-5	1	3.71	0.31	12.4	2.0
		2	4.03	0.38	24.0	1.1
		3	4.09	0.65	27.3	0.2
		5	4.18	0.41	33.2	1.7
		7	4.22	0.47	35.9	2.4
		14	4.23	0.27	42.3	1.3
		21	4.29	0.47	44.5	0.8
		28	4.30	0.27	47.9	0.3
Normal Weight Concrete	N-4	1	4.22	1.90	12.9	3.4
		2	4.32	1.86	18.2	0.6
		3	4.46	2.11	21.3	3.3
		5	4.50	2.49	27.9	2.5
		7	4.59	1.06	30.6	0.9
		14	4.63	0.57	38.0	5.3
		21	4.70	1.30	39.1	1.2
		28	4.69	0.54	40.0	4.8

Table 6.5:Regression analysis for the relationship between pulse velocity and compressive strength under dry curing as used in beam

Type of Concrete	Mix No.	Regression Equation	Coefficient of Correlation	95% Confidence Limit on Estimated Strength
Fully Lytag Concrete	L-C	$f_c=8.0x10^{-5}\ e^{3.64V}$	0.978	±32%
Semi Lytag Concrete	L-3	$f_c=2.1x10^{-4}\ e^{3.12V}$	0.983	±26%
Leca Concrete	Le-4	$f_c=0.62\ e^{1.07V}$	0.919	±31%
Pellite Concrete	P-5	$f_c=2.7x10^{-3}\ e^{2.26V}$	0.993	±15%
Normal Weight Concrete	N-4	$f_c=7.4x10^{-4}\ e^{2.32V}$	0.985	±22%

Table 6.6:Regression analysis for the relationship between exposed probe length and compressive strength for fully Lytag concrete

Age Days	Curing Condition	Regression Equation	Coefficient of Correlation
7	Wet	$f_c=-64.95+1.76\ W$	0.985
	Dry	$f_c=-51.56+1.52\ W$	0.899
	Wet+Dry	$f_c=-58.79+1.65\ W$	0.921
28	Dry	$f_c=-56.80+1.60\ W$	0.715
	Wet+Dry	$f_c=-41.86+1.36\ W$	0.824
7+28	Wet+Dry	$f_c=-55.98+1.60\ W$	0.925

Table 6.7:Regression analysis for the relationship between Lok force and compressive strength

Type of Concrete	Age Days	Curing Condition	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	7	Wet	f_c=-1.89+1.99 L	0.988
		Dry	f_c=-2.78+2.23 L	0.984
	28	Wet	f_c= 2.56+1.70 L	0.918
		Dry	f_c=-7.19+2.22 L	0.943
	7+28	Wet	f_c=-0.01+1.84 L	0.972
		Dry	f_c= 1.47+1.83 L	0.968
		Wet+Dry	f_c= 0.38+1.86 L	0.968
	180	Wet	f_c=-1.22+1.95 L	0.989
		Dry	f_c=-24.88+2.85 L	0.987
		Wet+Dry	f_c=-16.21+2.50 L	0.975
	360	Wet	f_c=20.23+1.20 L	0.984
		Dry	f_c=-50.81+4.07 L	0.996
		Wet+Dry	f_c=-11.39+2.33 L	0.920
	180 + 360	Wet	f_c= 6.41+1.68 L	0.966
		Dry	f_c=-34.06+3.29 L	0.969
		Wet+Dry	f_c=-13.76+2.42 L	0.944
	7-360	Wet+Dry	f_c=-1.40+1.94 L	0.972
Semi Lytag Concrete	7+28	Dry	f_c=-1.99+1.94 L	0.991
Leca Concrete	7+28	Wet	f_c=-0.73+1.81 L	0.936
Pellite Concrete	7+28	Dry	f_c=-4.42+1.97 L	0.982

Table 6.8: Regression analysis for the relationship between B.R.E. internal fracture torque and compressive strength

Type of Concrete	Age Days	Curing Condition	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	7	Wet	$f_c=2.79\ T^{2.17}$	0.996
		Dry	$f_c=4.13\ T^{1.78}$	0.984
	28	Wet	$f_c=10.03\ T^{1.02}$	0.894
		Dry	$f_c=11.90\ T^{0.95}$	0.915
	7+28	Wet+Dry	$f_c=4.67\ T^{1.69}$	0.978
	180	Wet+Dry	$f_c=13.21\ T^{0.88}$	0.924
	360	Wet+Dry	$f_c=15.44\ T^{0.82}$	0.735
	180 + 360	Wet+Dry	$f_c=14.08\ T^{0.86}$	0.804
	7-360	Wet+Dry	$f_c=5.75\ T^{1.50}$	0.941
Semi Lytag Concrete	7+28	Wet	$f_c=2.89\ T^{1.63}$	0.955
		Wet+Dry	$f_c=3.14\ T^{1.57}$	0.963
Pellite Concrete	7-28	Wet	$f_c=3.45\ T^{1.68}$	0.980

Table 6.9:Regression analysis for the relationship between direct pull internal fracture force and compressive strength

Type of Concrete	Age Days	Curing Condition	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	7	Wet	$f_c=2.34\ F^{1.48}$	0.991
		Dry	$f_c=1.51\ F^{1.82}$	0.998
	28	Wet	$f_c=3.77\ F^{1.23}$	0.976
		Dry	$f_c=2.98\ F^{1.42}$	0.974
	7+28	Wet+Dry	$f_c=1.99\ F^{1.62}$	0.987
	180	Wet+Dry	$f_c=7.29\ F^{0.95}$	0.787
	360	Wet+Dry	$f_c=4.26\ F^{1.24}$	0.816
	180 + 360	Wet+Dry	$f_c=5.53\ F^{1.10}$	0.808
	7-360	Wet+Dry	$f_c=2.09\ F^{1.60}$	0.973
Semi Lytag Concrete	7+28	Wet	$f_c=1.56\ F^{1.69}$	0.978
		Wet+Dry	$f_c=1.56\ F^{1.70}$	0.970
Leca Concrete	7+28	Wet	$f_c=9.98\ F^{0.66}$	0.795
Pellite Concrete	7-28	Wet	$f_c=2.39\ F^{1.51}$	0.978
		Dry	$f_c=2.46\ F^{1.52}$	0.996
		Wet+Dry	$f_c=2.44\ F^{1.51}$	0.984

Table 6.10:Accuracy of strength measurements from internal fracture tests

Type of Concrete	95 % Confidence Limit on Estimated Strength	
	B.R.E.	Direct Pull
Fully Lytag Concrete	±34 %	±16 %
Semi Lytag Concrete	±39 %	±32 %
Leca Concrete	-	±77 %
Pellite Concrete	±26 %	±18 %

Table 6.11: Regression analysis for the relationship between pull off and compressive strengths in surface pull off tests

Type of Concrete	Age Days	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	7	$f_c=2.73\ p^{1.95}$	0.987
	28	$f_c=5.92\ p^{1.41}$	0.970
	7+28	$f_c=3.02\ p^{1.90}$	0.986
	180+360	$f_c=2.44\ p^{2.04}$	0.951
Semi Lytag Concrete	7	$f_c=5.98\ p^{1.38}$	0.999
	28	$f_c=8.37\ p^{1.12}$	0.995
	7+28	$f_c=6.72\ p^{1.28}$	0.993
Leca Concrete	14+28	$f_c=8.32\ p^{0.91}$	0.976
	360	$f_c=6.74\ p^{1.06}$	0.979
Pellite Concrete	7+14+28	$f_c=6.99\ p^{1.30}$	0.975
Normal Weight Concrete	7	$f_c=4.15\ p^{1.91}$	0.944
	28	$f_c=3.31\ p^{2.20}$	0.997
	7+28	$f_c=3.66\ p^{2.08}$	0.974
	180+360	$f_c=3.97\ p^{1.96}$	0.972

Table 6.12: Accuracy of strength measurements from surface pull off tests

Concrete Type	95 % Confidence Limit on Estimated Strength
Fully Lytag Concrete	±24 %
Semi Lytag Concrete	±19 %
Leca Concrete	±15 %
Pellite Concrete	±24 %
Normal Weight Concrete	±25 %

Table 6.13: Regression analysis for the relationship between pull off and compressive strengths in partial cored pull off tests

Concrete Type	Regression Equation	Coefficient of Correlation
Fully Lytag Concrete	$f_c=1.72\ p^{3.27}$	0.891
Semi Lytag Concrete	$f_c=6.98\ p^{1.60}$	0.982
Leca Concrete	$f_c=15.15\ p^{1.04}$	0.973
Pellite Concrete	$f_c=9.37\ p^{1.36}$	0.983

Table 6.14:Summary of correction factors for different types of 28 day old lightweight concrete

Concrete Type	Curing Regime	Cube Strength N/mm^2	ℓ/d ratio 1.0	1.4	1.6	2.0
Fully Lytag Concrete	Wet	24.8	0.83	0.92	0.95	1.00
		29.1	0.85	0.93	0.95	1.00
		33.3	0.87	0.94	0.99	1.00
		41.6	0.89	0.96	0.99	1.00
	Dry	30.0	0.92	0.93	0.93	1.00
		31.9	0.87	0.98	0.97	1.00
		40.5	0.89	0.96	1.02	1.00
		47.2	0.91	0.96	0.99	1.00
Semi Lytag Concrete	Wet	20.7	0.83	-	-	1.00
		47.8	0.88	-	-	1.00
Leca Concrete	Wet	15.1	0.83	-	-	1.00
		25.8	0.91	-	-	1.00
Pellite Concrete	Wet	21.8	0.79	-	-	1.00
		46.4	0.85	-	-	1.00
	Dry	20.8	0.83	-	-	1.00
		45.4	0.87	-	-	1.00

Table 6.15:Comparison of core correction factors

Type of Concrete	Curing Regime	ℓ/d Ratio 1.0	1.4	1.6	2.0
Fully Lytag Concrete	Wet	0.86	0.94	0.97	1.00
	Dry	0.90	0.96	0.98	1.00
Semi Lytag Concrete	Wet	0.86	-	-	1.00
Leca Concrete	Wet	0.87	-	-	1.00
Pellite Concrete	Wet	0.82	-	-	1.00
	Dry	0.85	-	-	1.00
Normal weight Concrete(Bungey,1979)		0.77	0.86	0.91	1.00
A.S.T.M C42(1984)		0.87	0.95	0.97	1.00
B.S. 1881;Part120(1983)		0.80	0.90	0.94	1.00

Table 6.16:Regression analysis for the relationship between core and cube strengths at the age of 28 days

Type of Concrete	Curing Regime	Regression Equation	Correlation Coefficient
Fully Lytag Concrete	Wet	$f_c=0.96\ f_{cc}^{1.04}$	0.954
	Dry	$f_c=0.77\ f_{cc}^{1.09}$	0.999
	Wet+Dry	$f_c=0.90\ f_{cc}^{1.05}$	0.978
Semi Lytag Concrete	Wet	$f_c=1.11\ f_{cc}^{1.00}$	0.990
Leca Concrete	Wet	$f_c=1.05\ f_{cc}^{1.20}$	0.962
Pellite Concrete	Wet	$f_c=1.93\ f_{cc}^{0.87}$	0.991
	Dry	$f_c=2.01\ f_{cc}^{0.86}$	0.997
	Wet+Dry	$f_c=2.00\ f_{cc}^{0.86}$	0.993

Table 6.17:Comparison of reliabilty of partially destructive tests on different types of concrete

Concrete Type	Test Method	Coefficient of Variation %	95 % Confidence Limit on Estimated Strength (%)
Fully Lytag Concrete	Pull Out	5.6	±17
	B.R.E. Internal Fracture	9.0	±34
	Direct Pull Internal Frature	9.8	±16
	Pull Off	5.7	±24
Semi Lytag Concrete	Pull Out	7.0	±13
	B.R.E. Internal Fracture	13.4	±39
	Direct Pull Internal Fracture	8.3	±35
	Pull Off	8.6	±19
Leca Concrete	Pull Out	12.0	±23
	Direct Pull Internal Fracture	34.0	±77
	Pull Off	23.8	±15
Pellite Concrete	Pull Out	7.4	±18
	B.R.E. Internal Fracture	13.6	±26
	Direct Pull Internal Fracture	8.7	±18
	Pull Off	9.0	±24
Normal Weight Concrete	Pull Out	7.0^{+}	$\pm 20^{x}$
	B.R.E. Internal Fracture	15.9^{+}	$\pm 28^{*}$
	Direct Pull Internal Fracture	15.6^{+}	$\pm 20^{*}$
	Pull Off	8.0^{+}	±25

+;Bungey and Madandoust(1989) x;Bungey(1987) *;Bungey(1989)

Figure 6.1: Floating of Leca aggregates to the top of the specimens during vibration

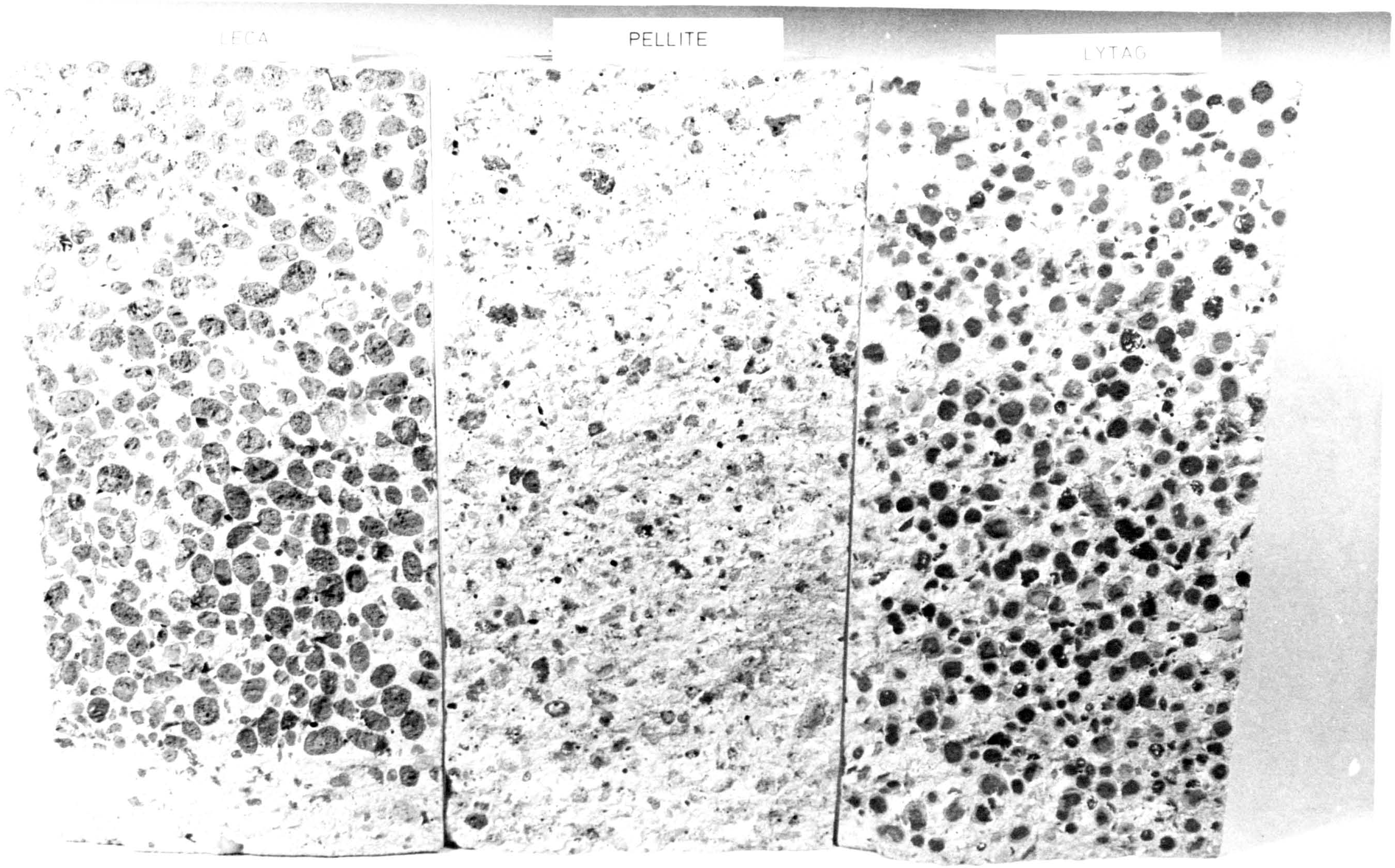


Figure 6.2: Distribution of lightweight aggregates in the concrete cylinder specimens

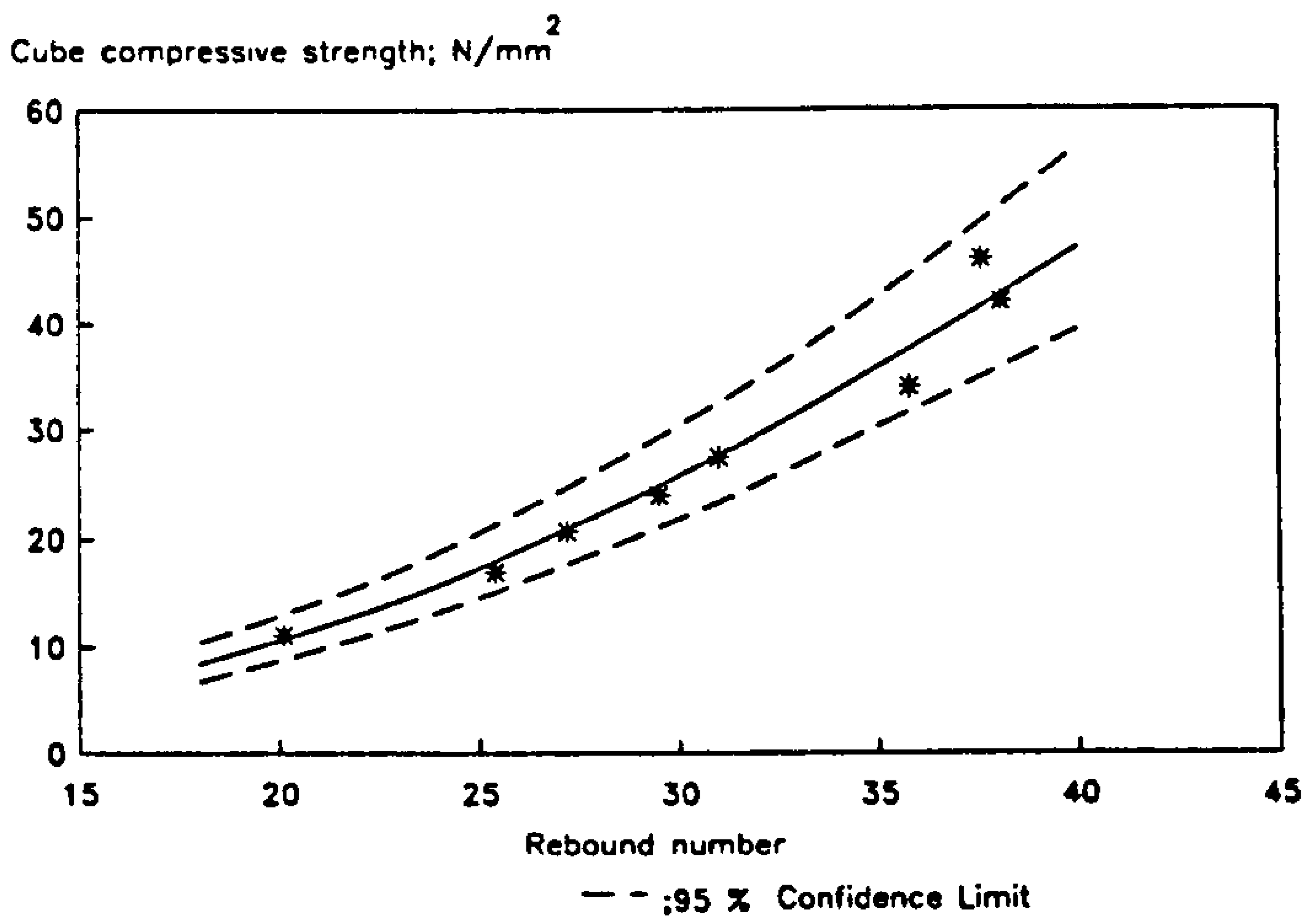


Figure 6.3:Relationship between rebound number and compressive strength for fully Lytag concrete

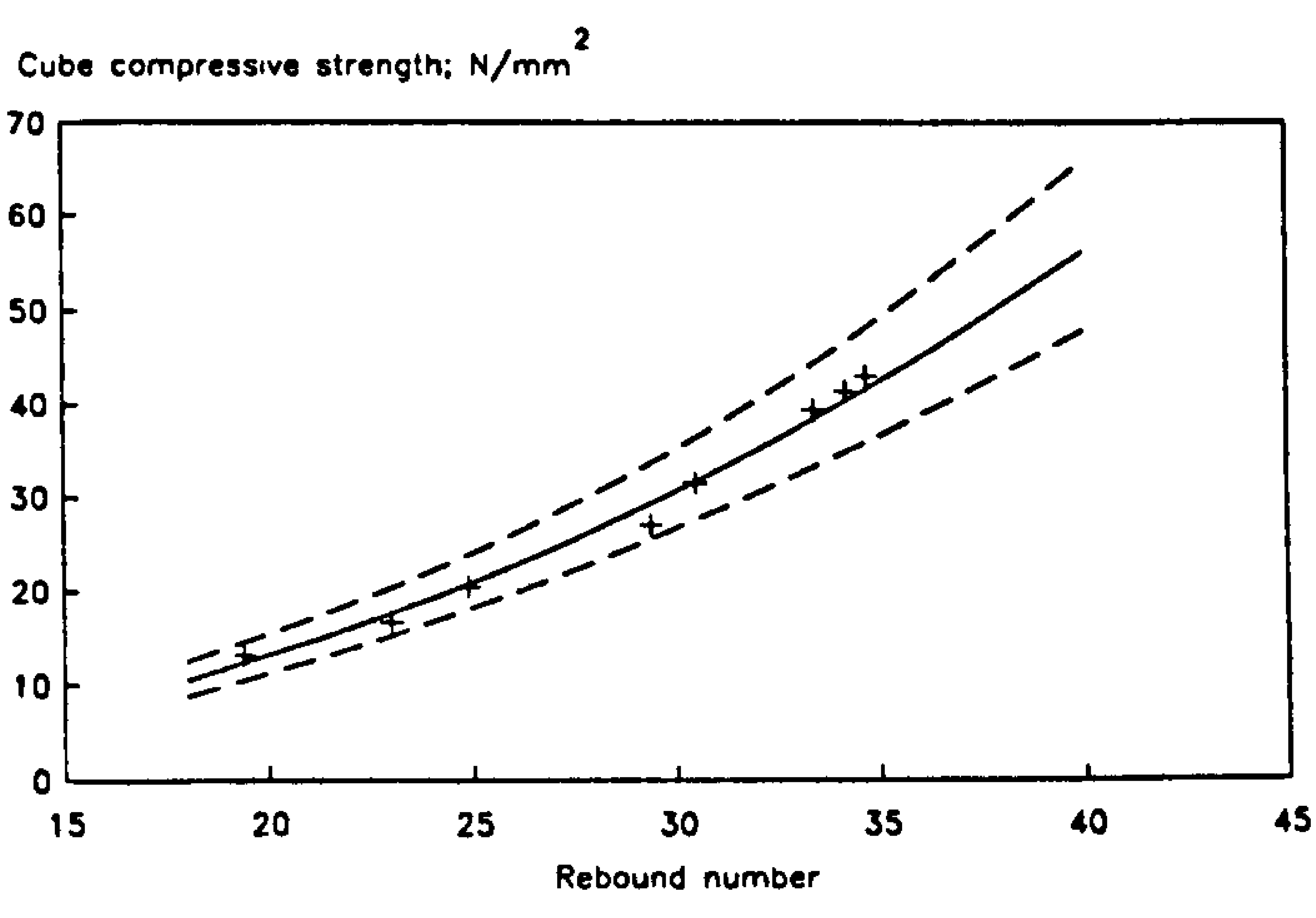


Figure 6.4:Relationship between rebound number and compressive strength for semi Lytag concrete

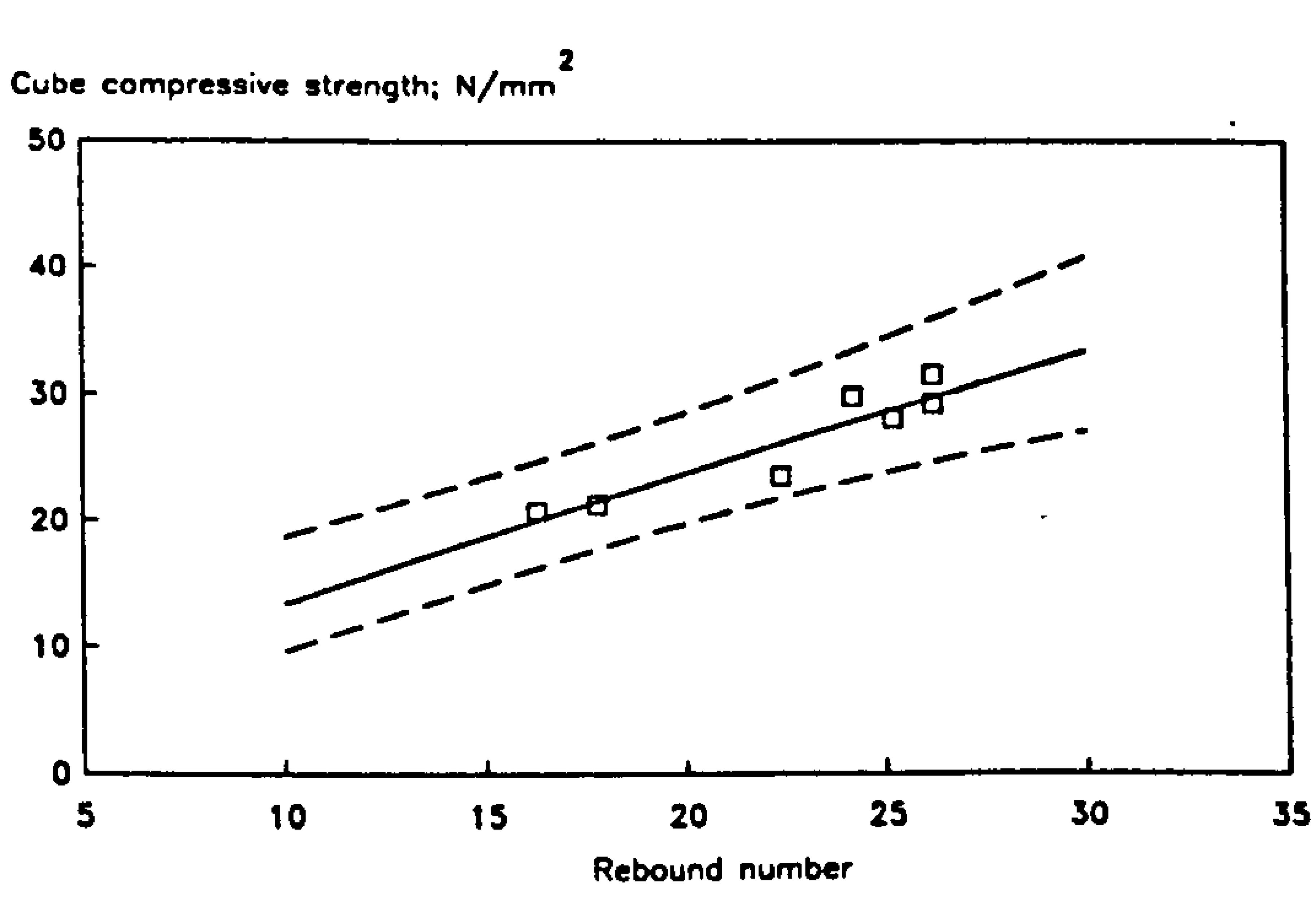


Figure 6.5:Relationship between rebound number and compressive strength for Leca concrete

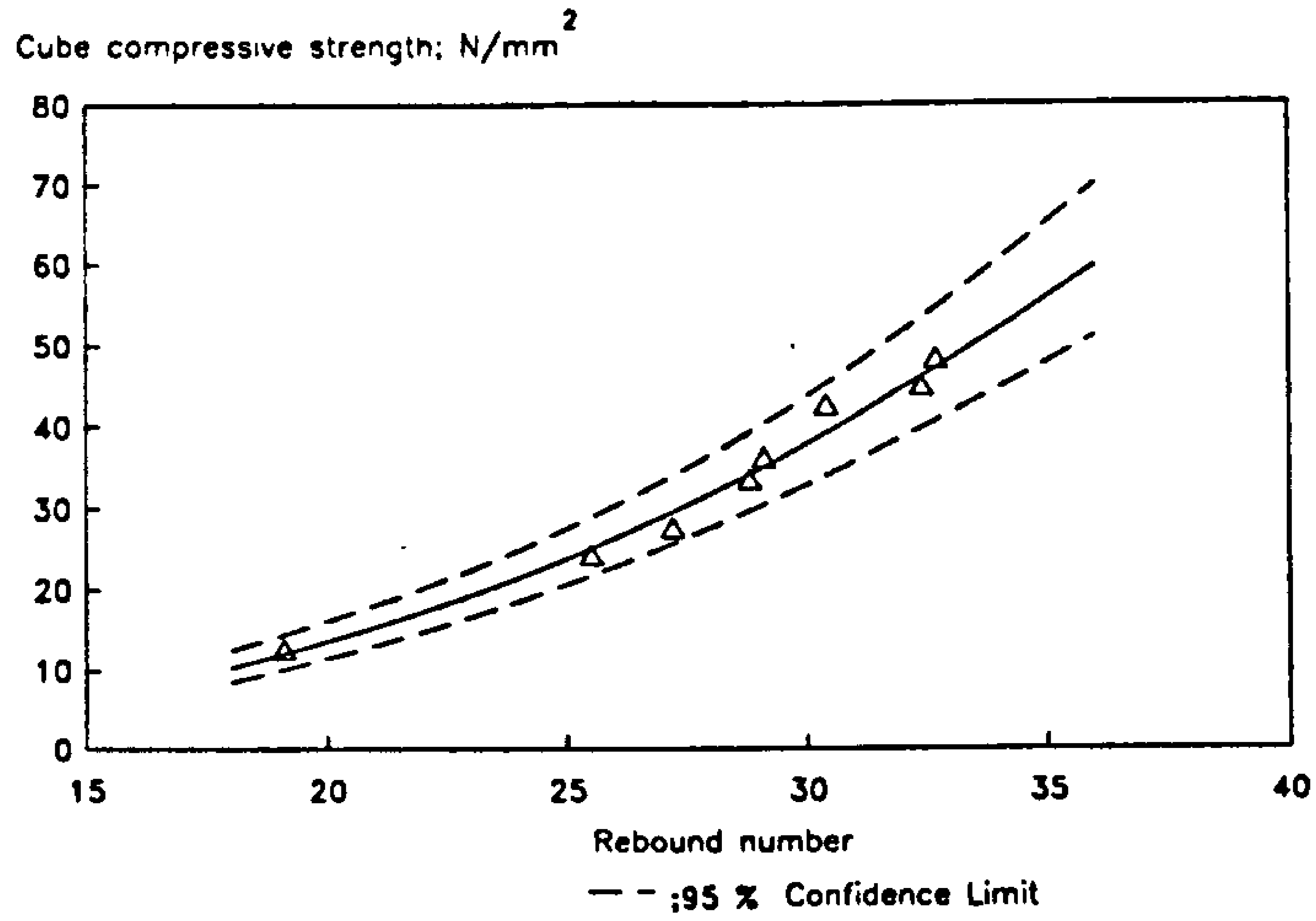


Figure 6.6:Relationship between rebound number and compressive strength for Pellite concrete

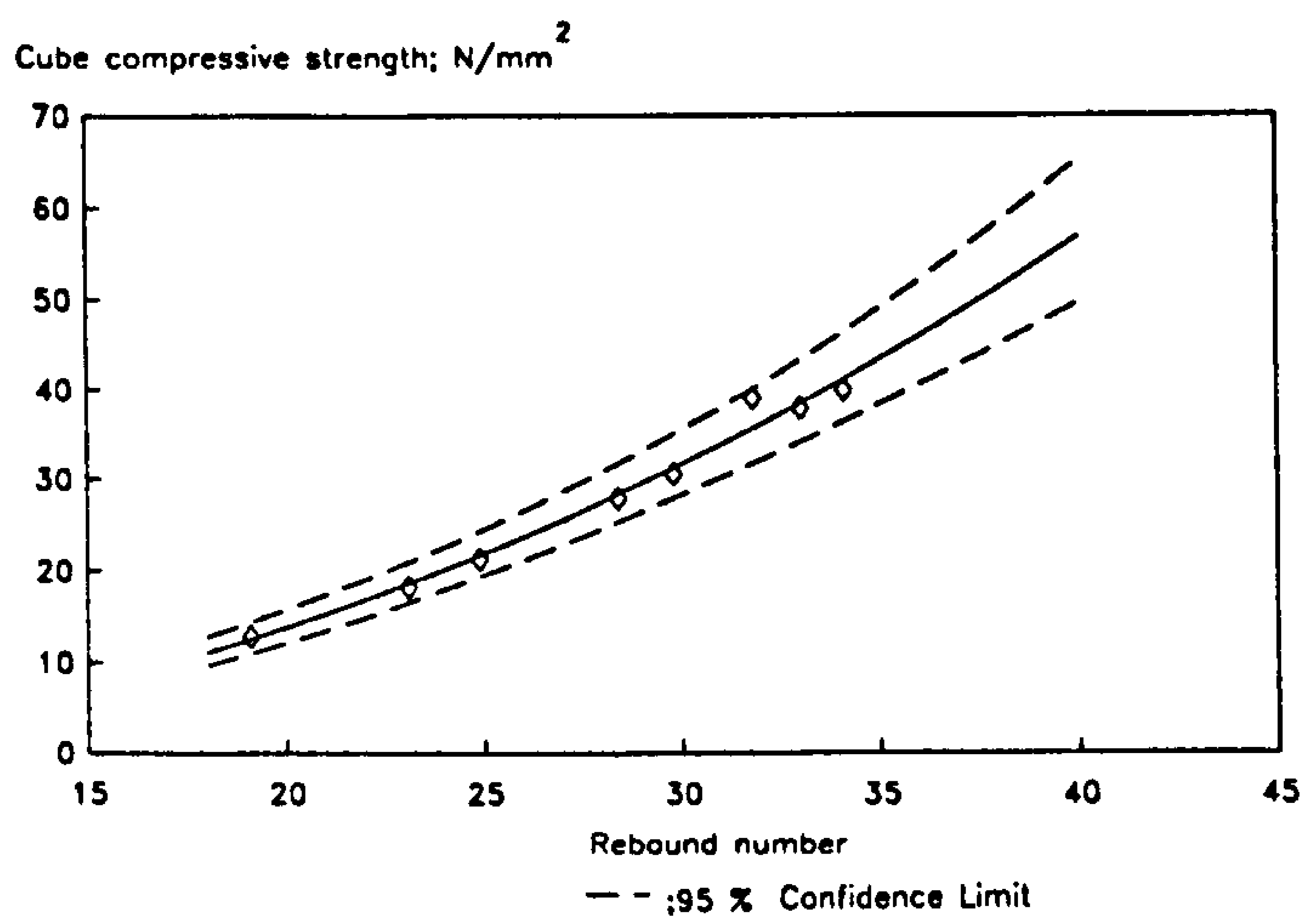


Figure 6.7:Relationship between rebound number and compressive strength for normal weight concrete

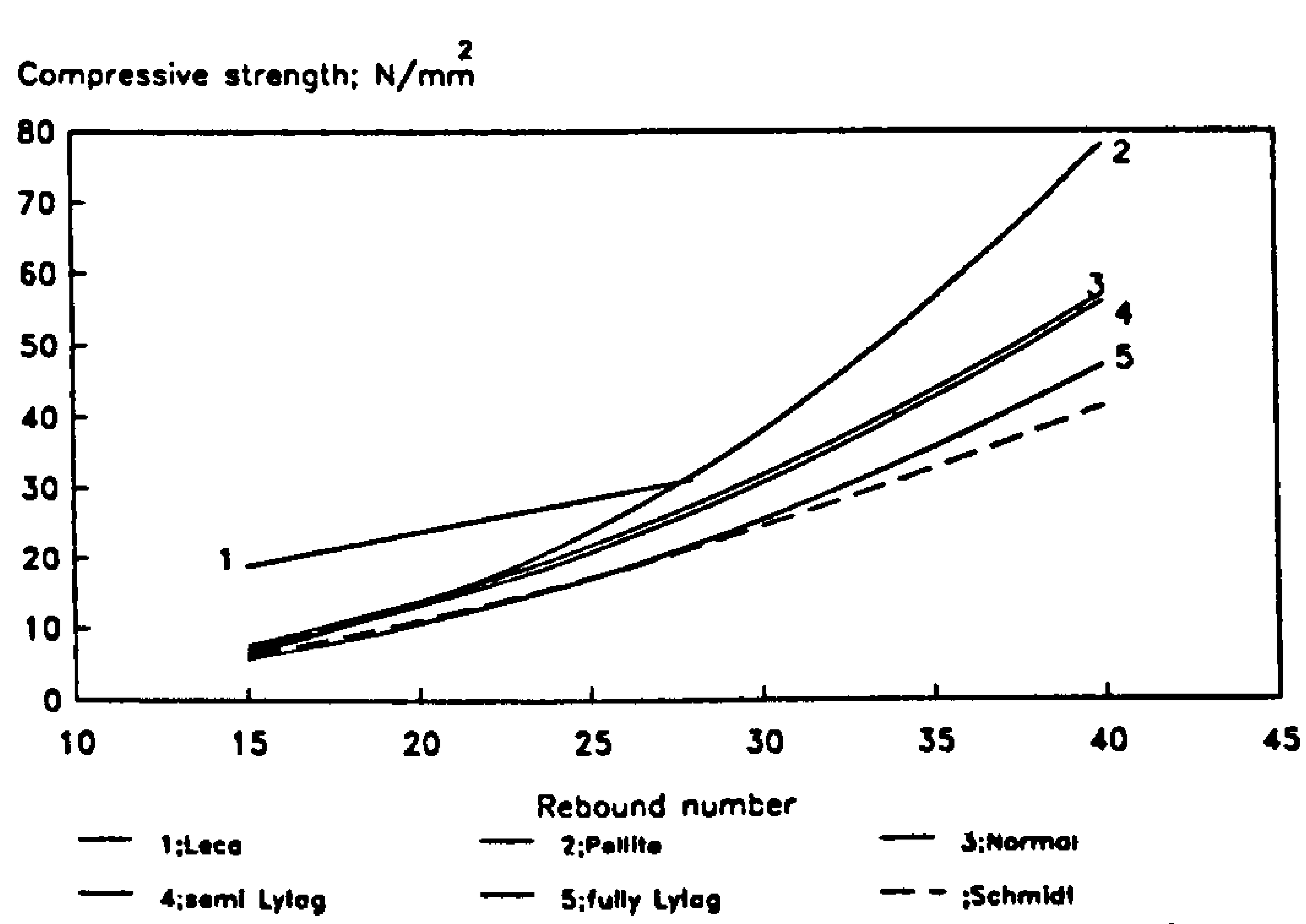


Figure 6.8:Relationship between rebound number and compressive strength for different types of concrete

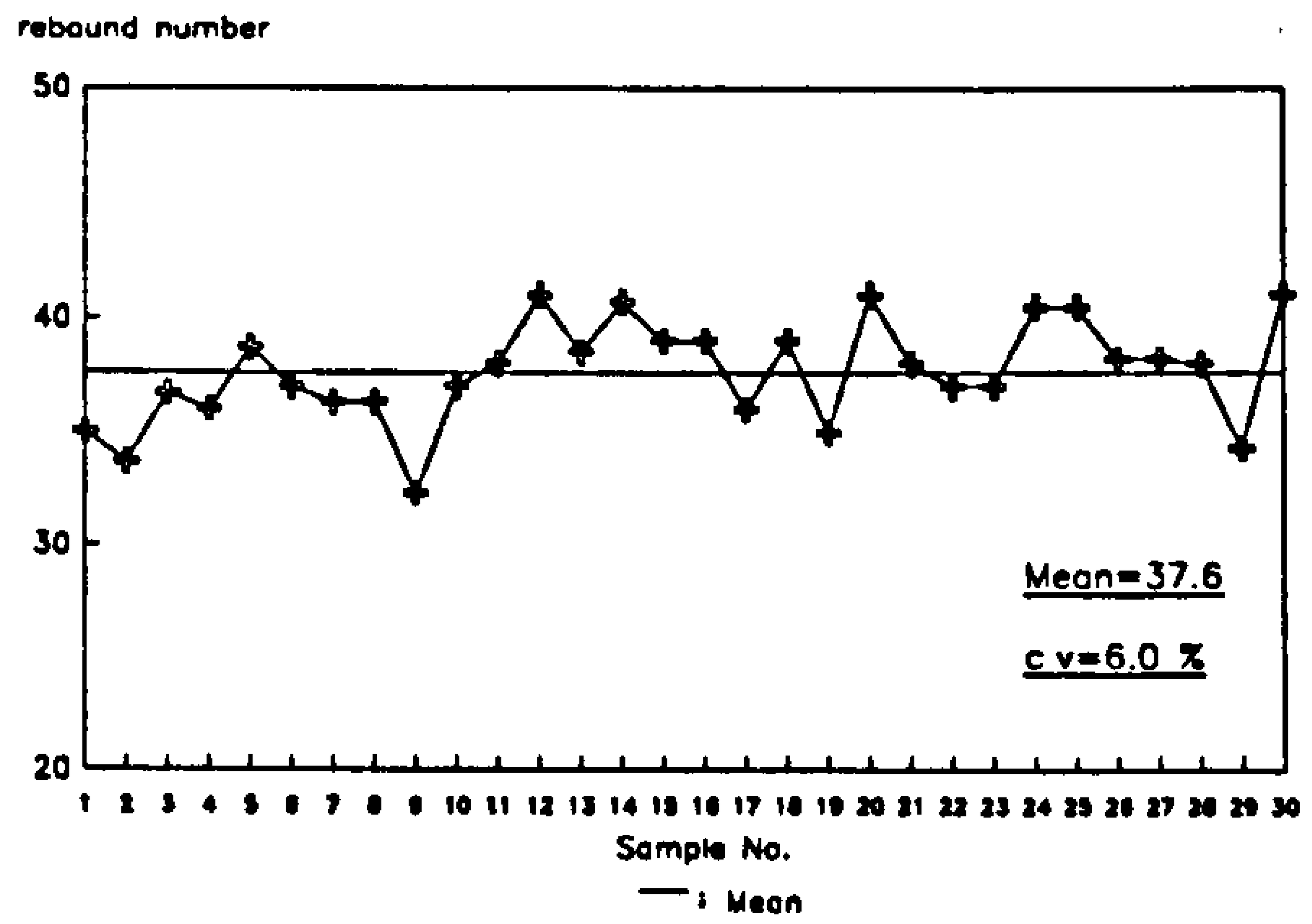


Figure 6.9:Typical variation of rebound number on fully Lytag concrete

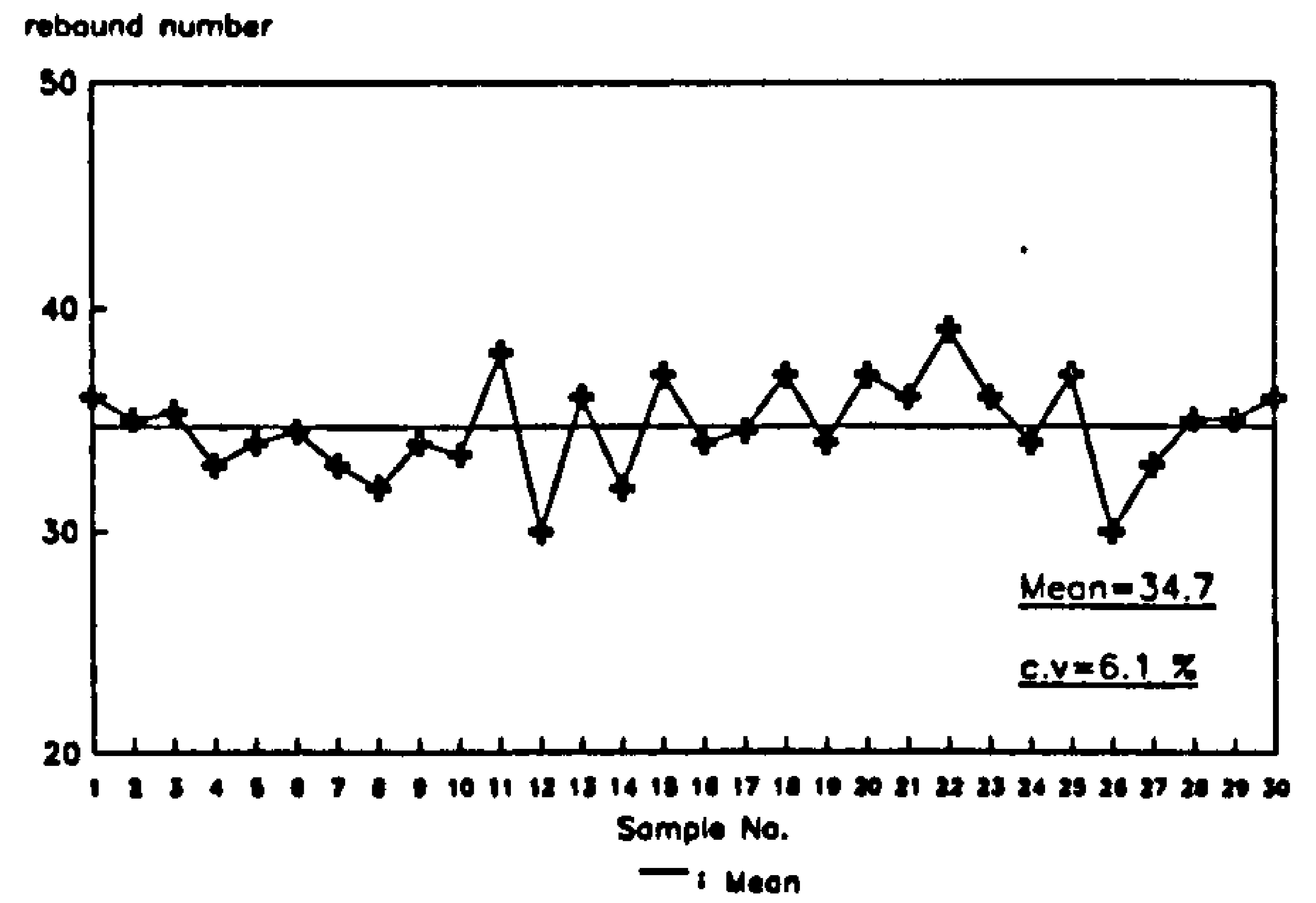


Figure 6.10:Typical variation of rebound number on semi Lytag concrete

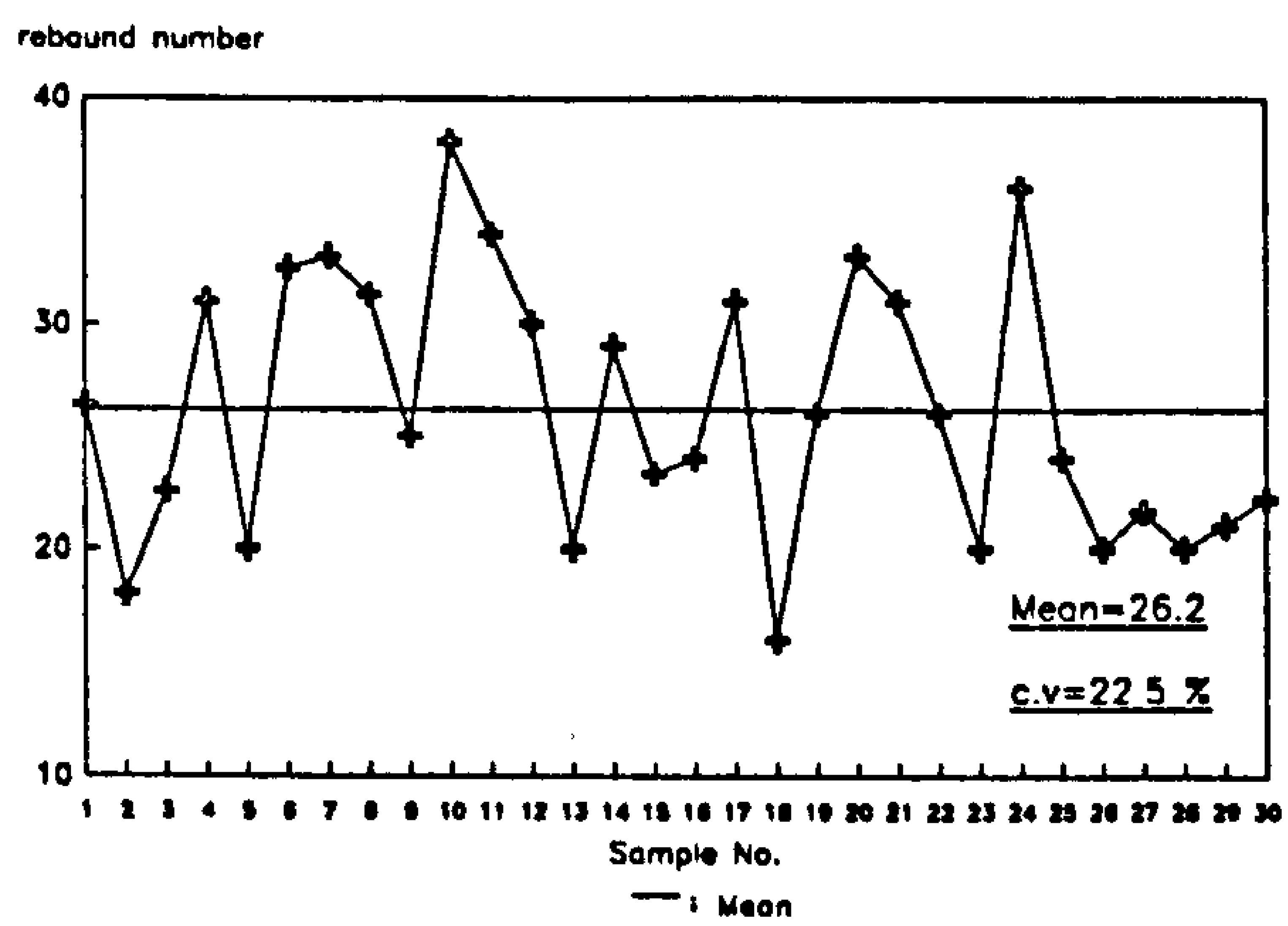


Figure 6.11:Typical variation of rebound number on Leca concrete

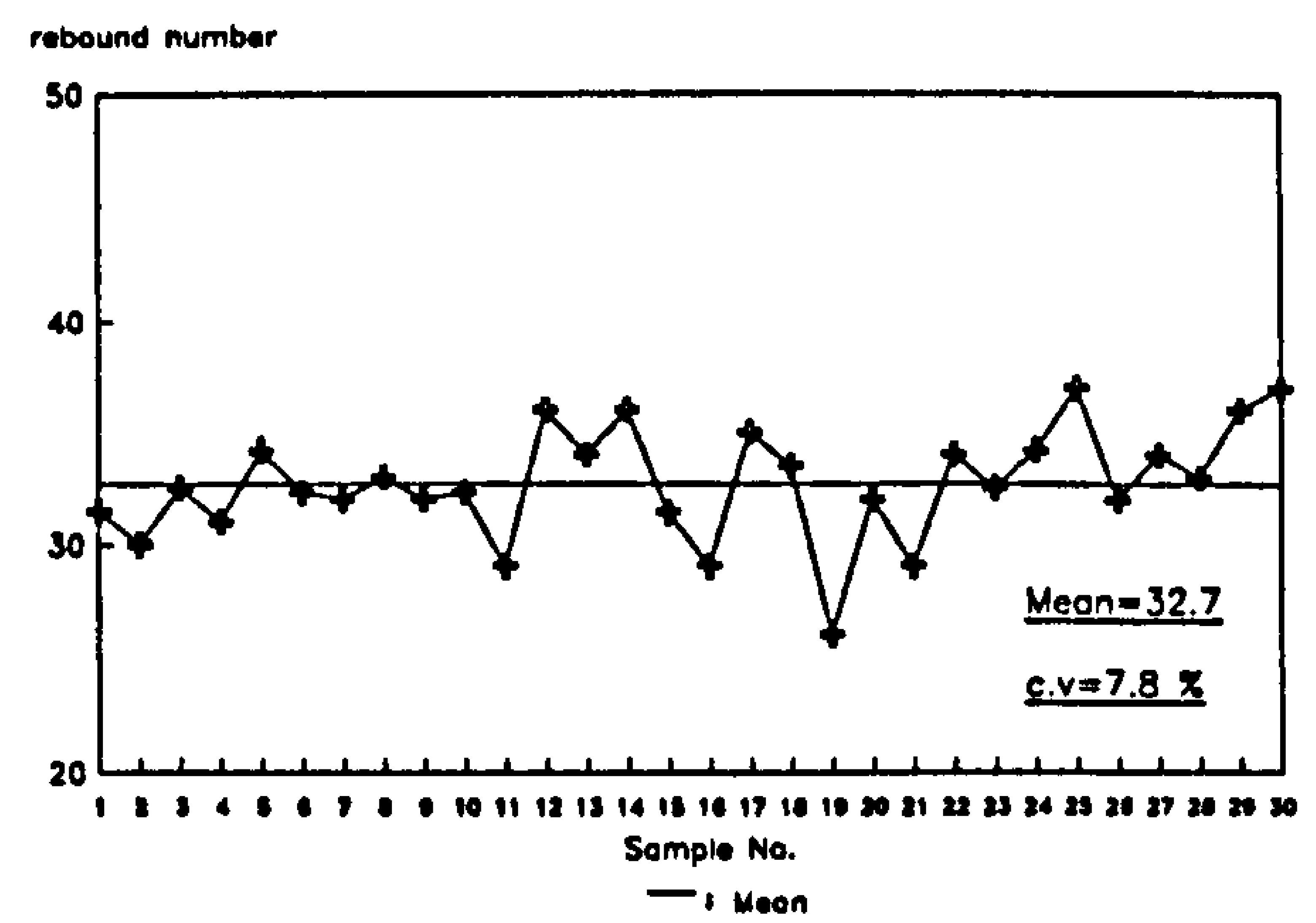


Figure 6.12:Typical variation of rebound number on Pellite concrete

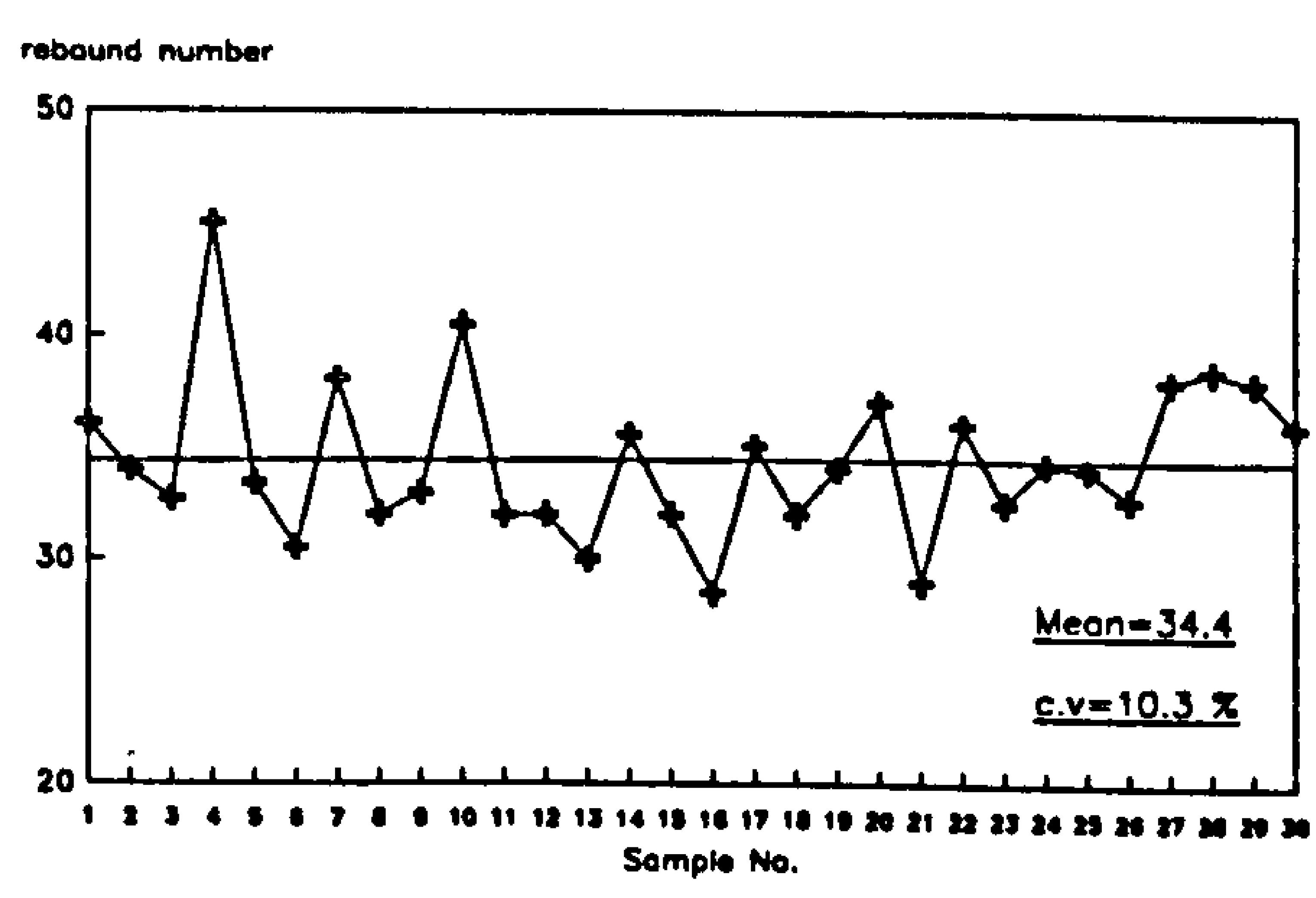


Figure 6.13:Typical variation of rebound number on normal weight concrete

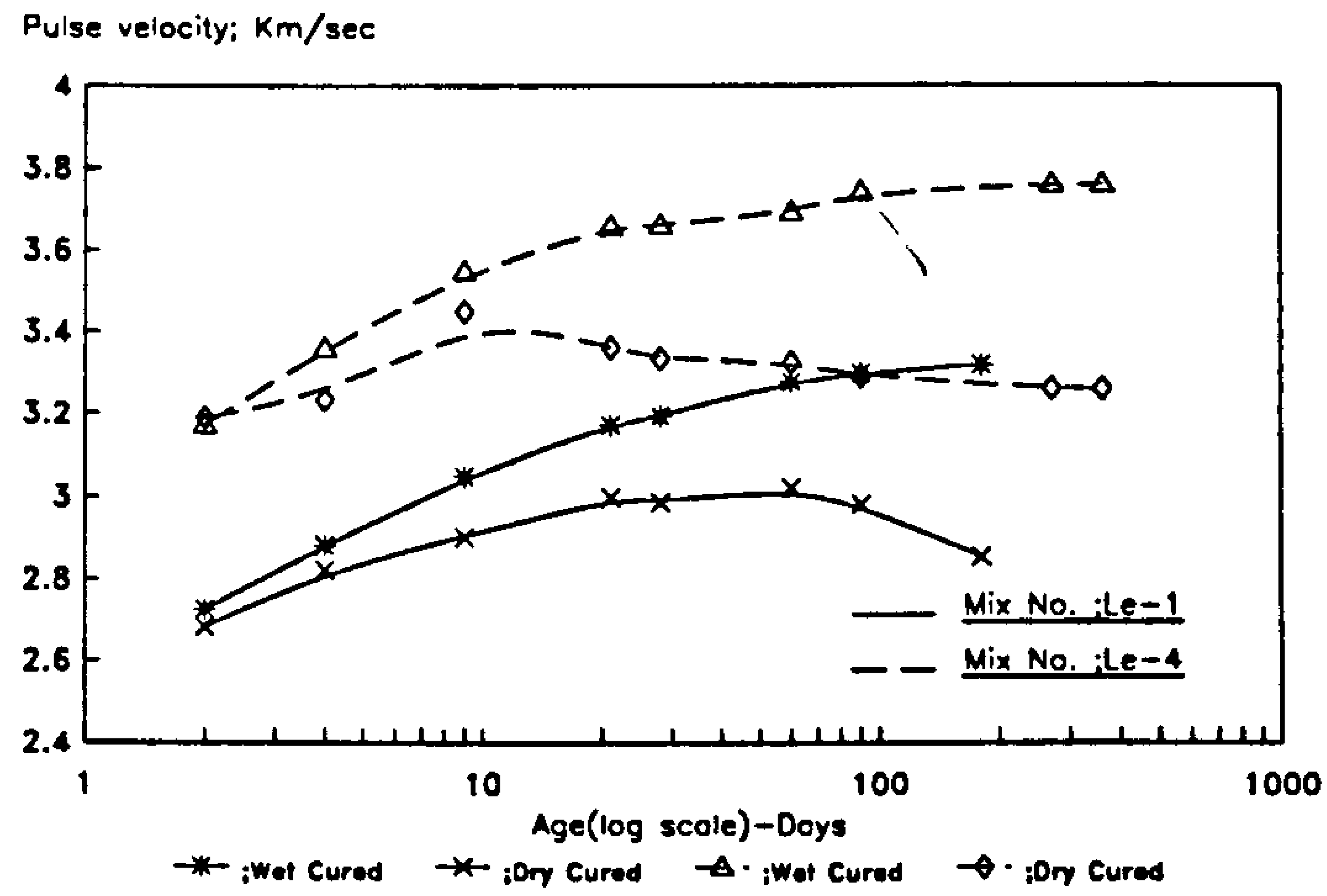


Figure 6.14:Pulse velocity development with age for Leca concrete

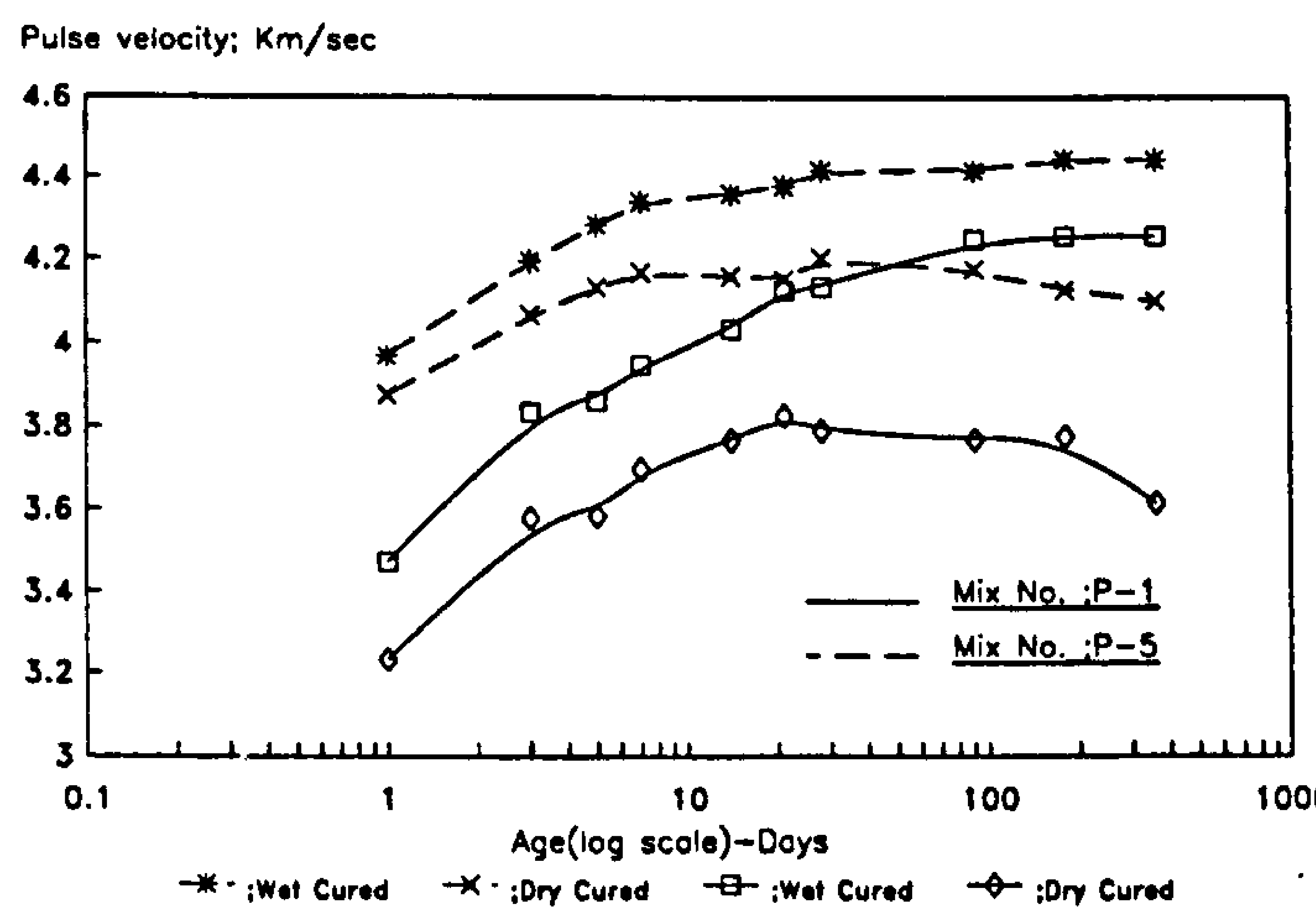


Figure 6.15:Pulse velocity development with age for Pellite concrete

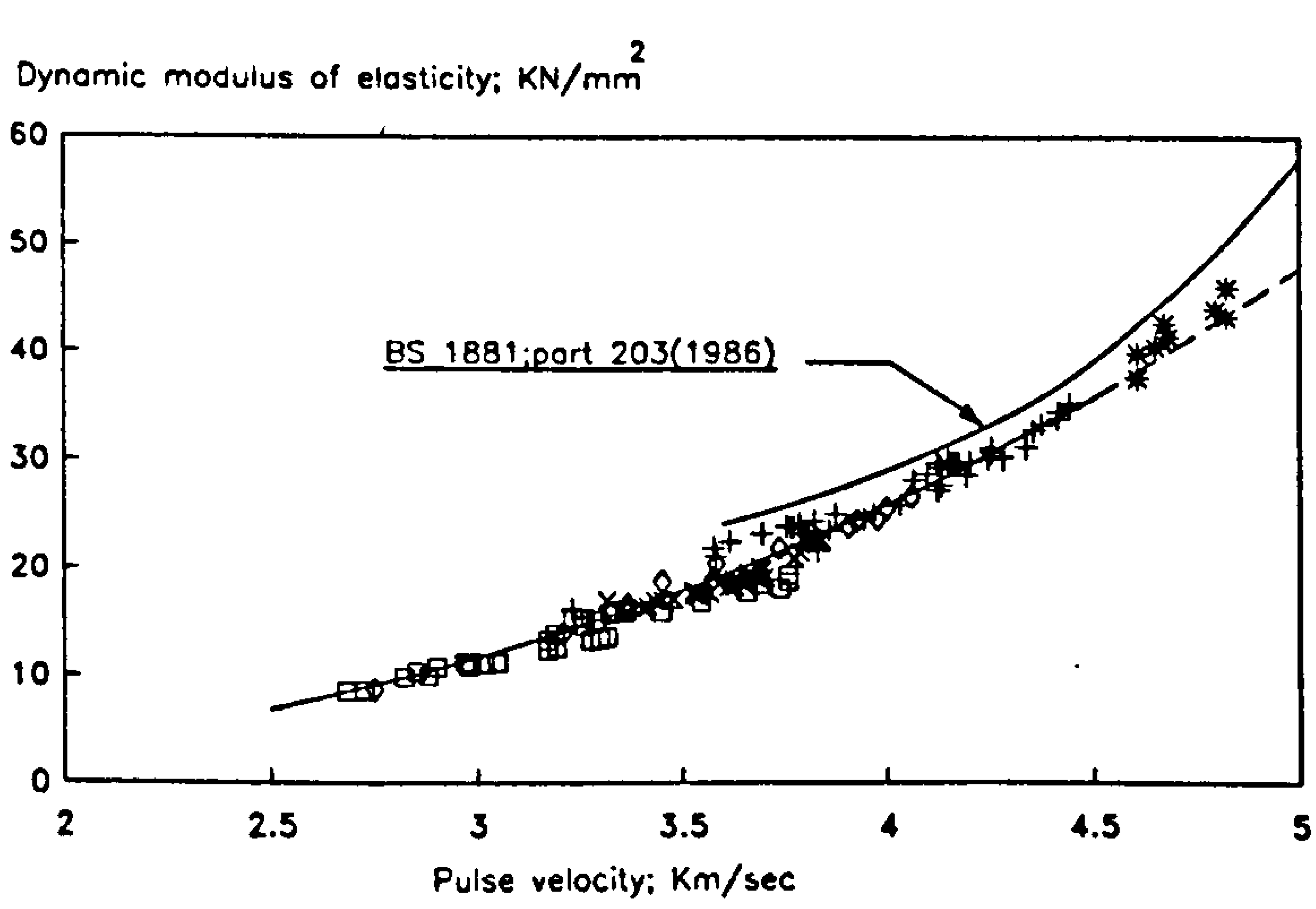


Figure 6.16:Relationship between pulse velocity and dynamic modulus of elasticity of different types of concrete

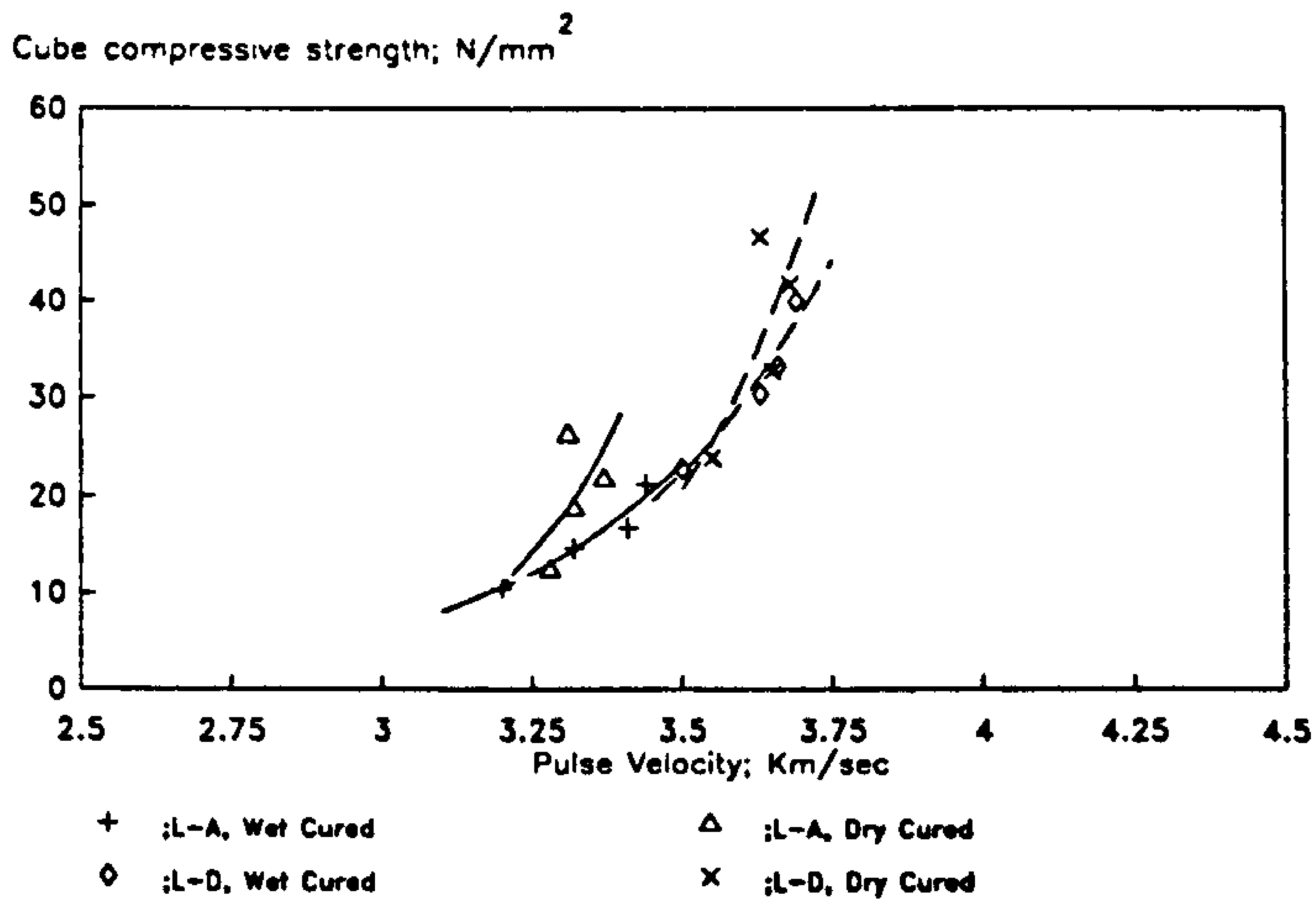


Figure 6.17:Relationship between pulse velocity and compressive strength for fully Lytag concrete

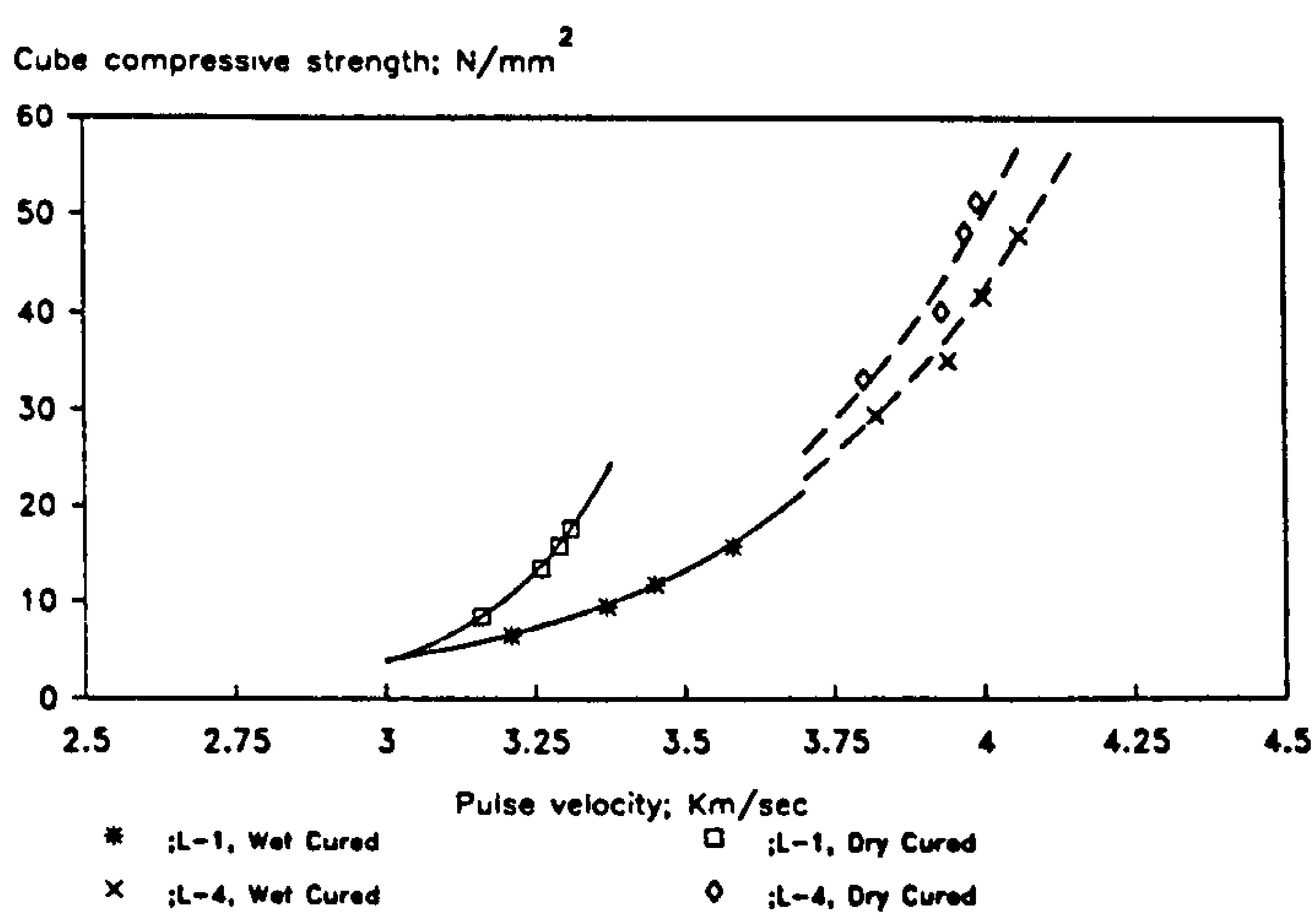


Figure 6.18:Relationship between pulse velocity and compressive strength for semi Lytag concrete

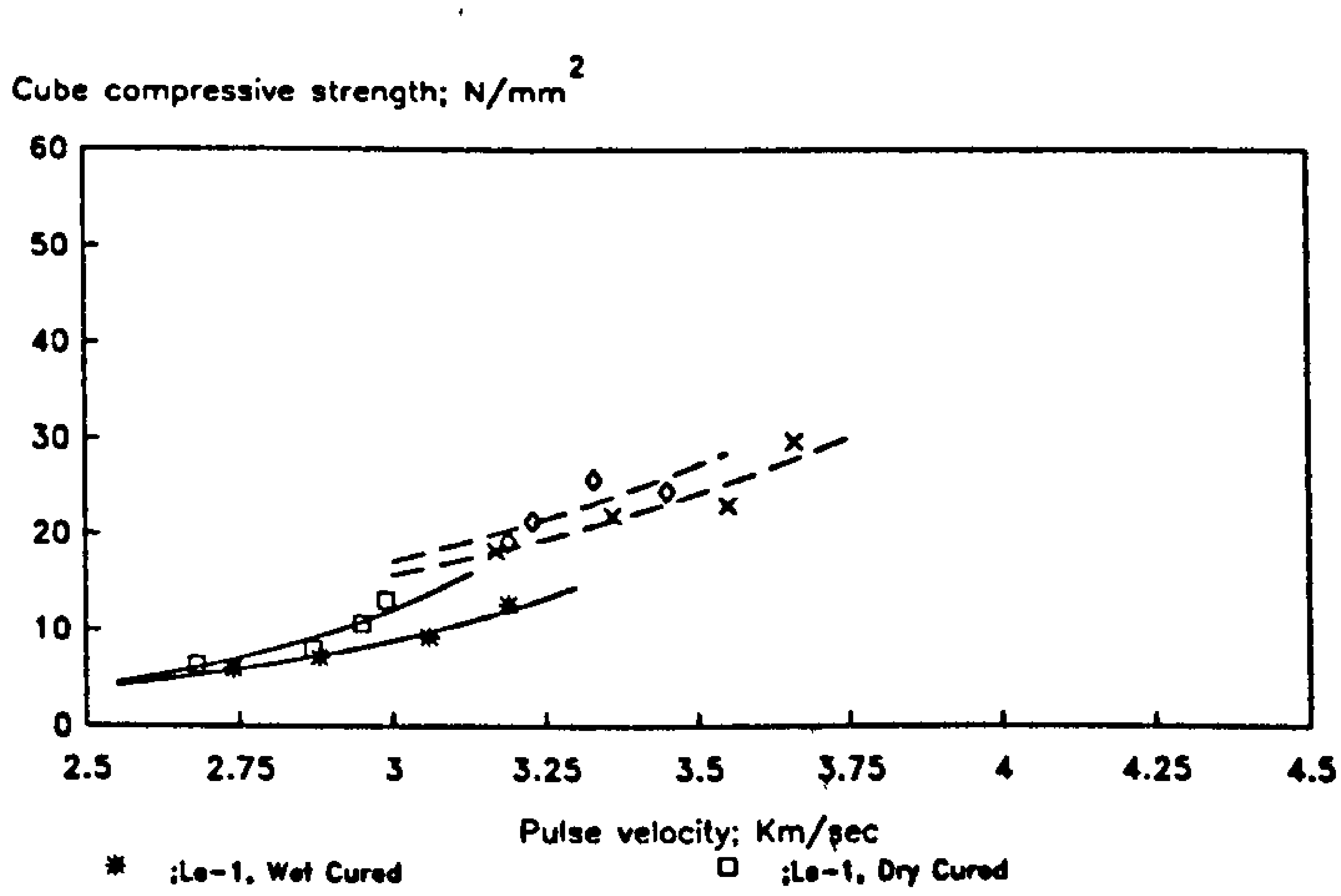


Figure 6.19:Relationship between pulse velocity and compressive strength for Leca concrete

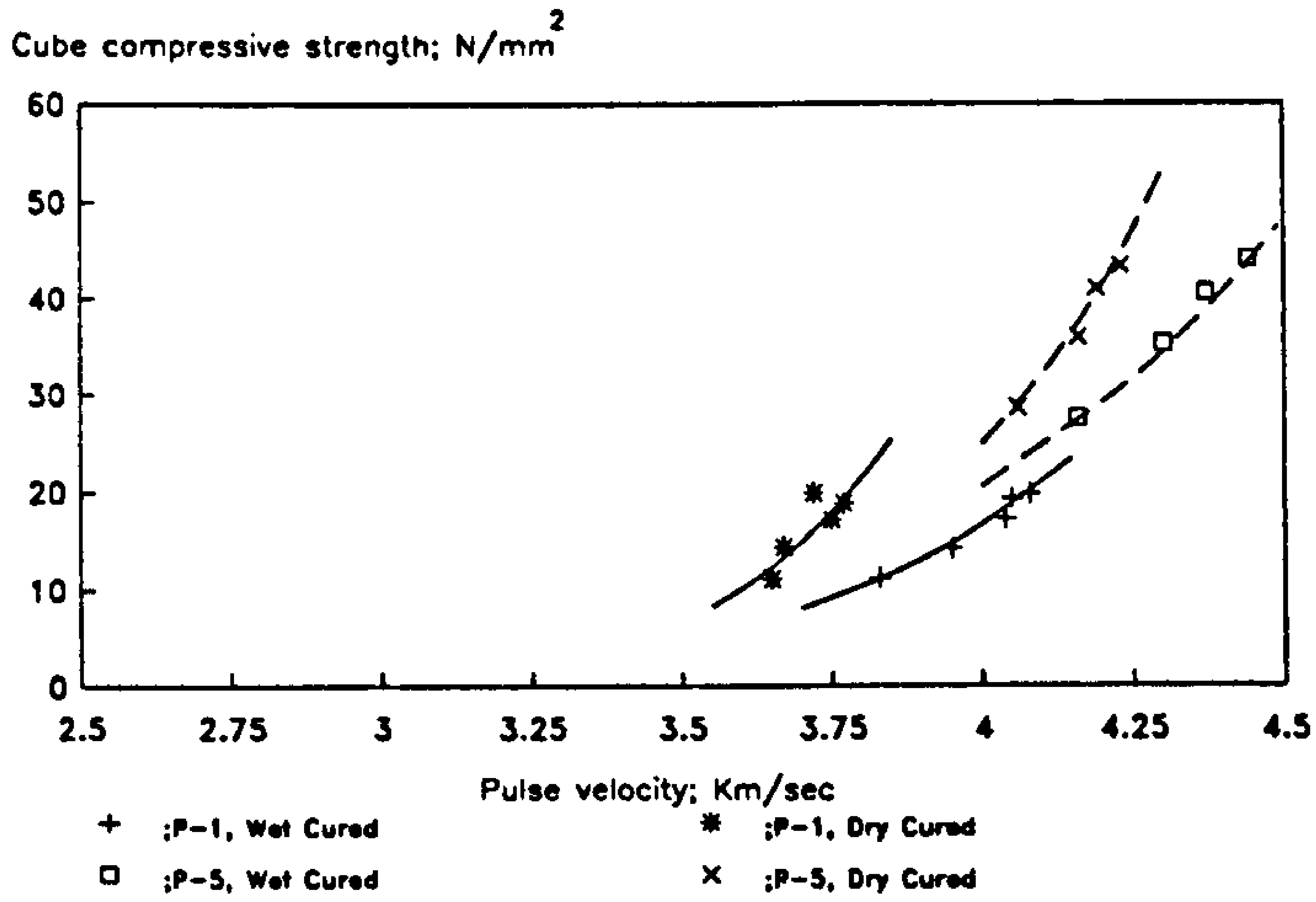


Figure 6.20:Relationship between pulse velocity and compressive strength for Pellite concrete

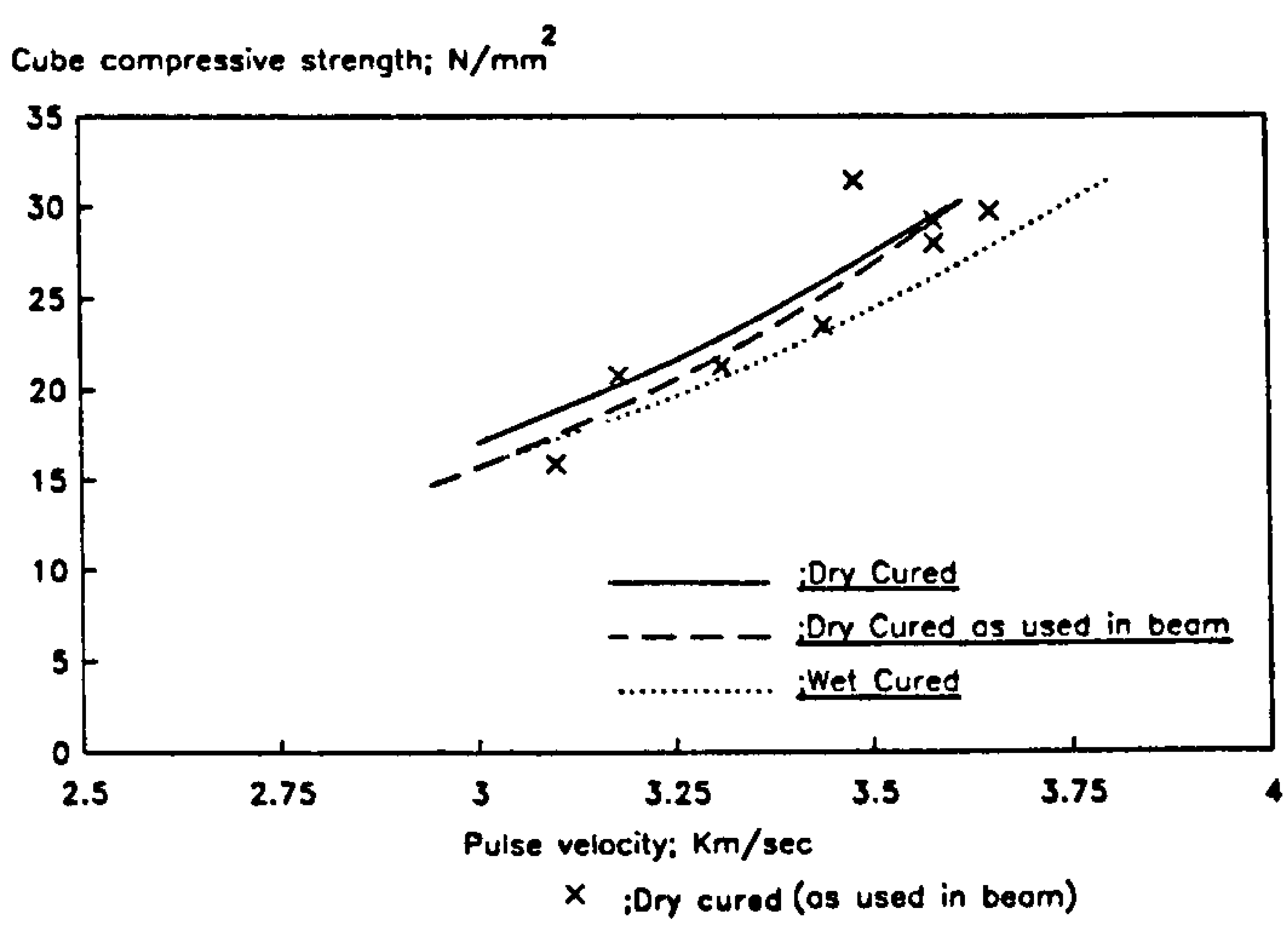


Figure 6.21:Relationship between pulse velocity and compressive strength for richest mix of Leca concrete

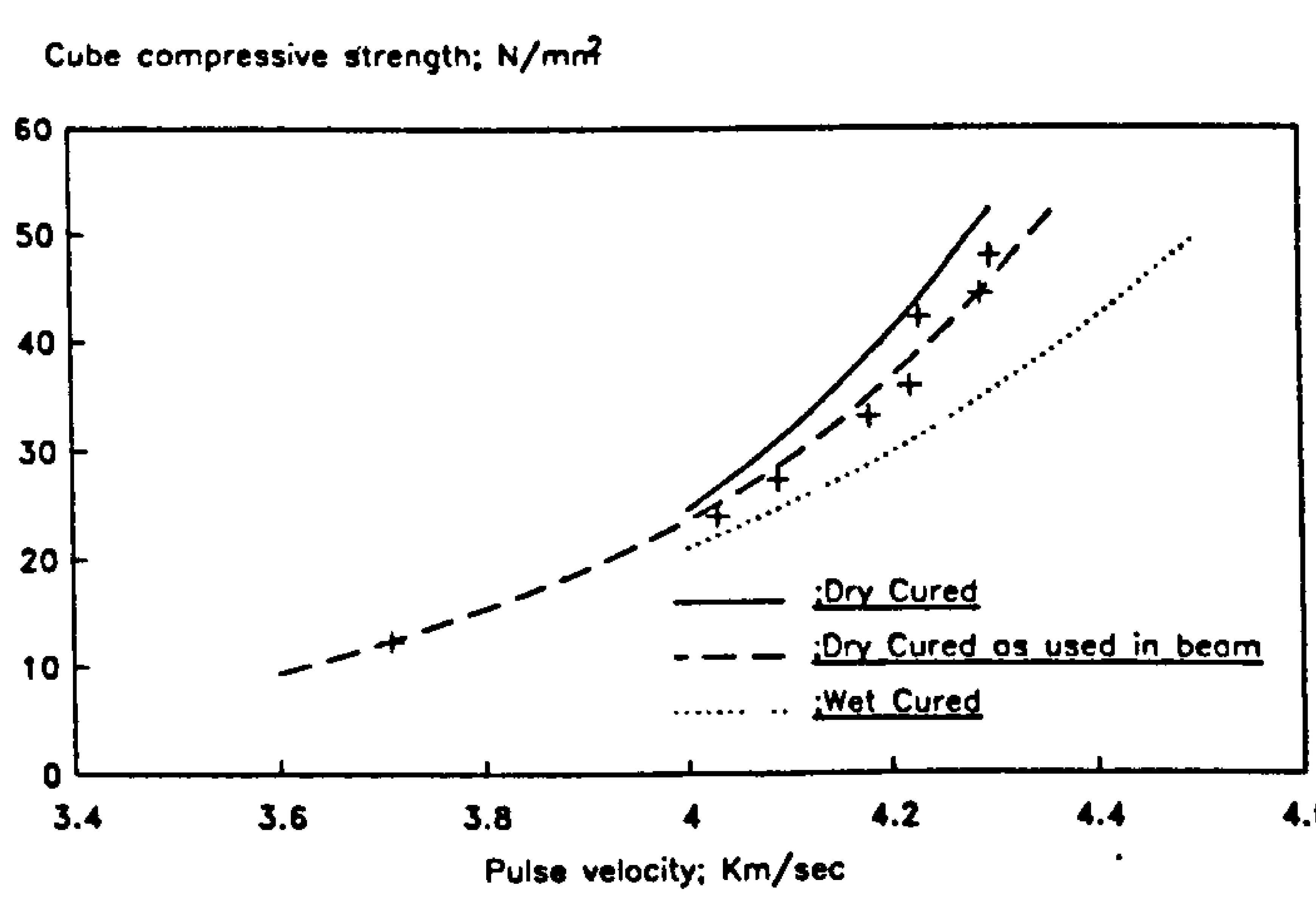


Figure 6.22:Relationship between pulse velocity and compressive strength for richest mix of Pellite concrete

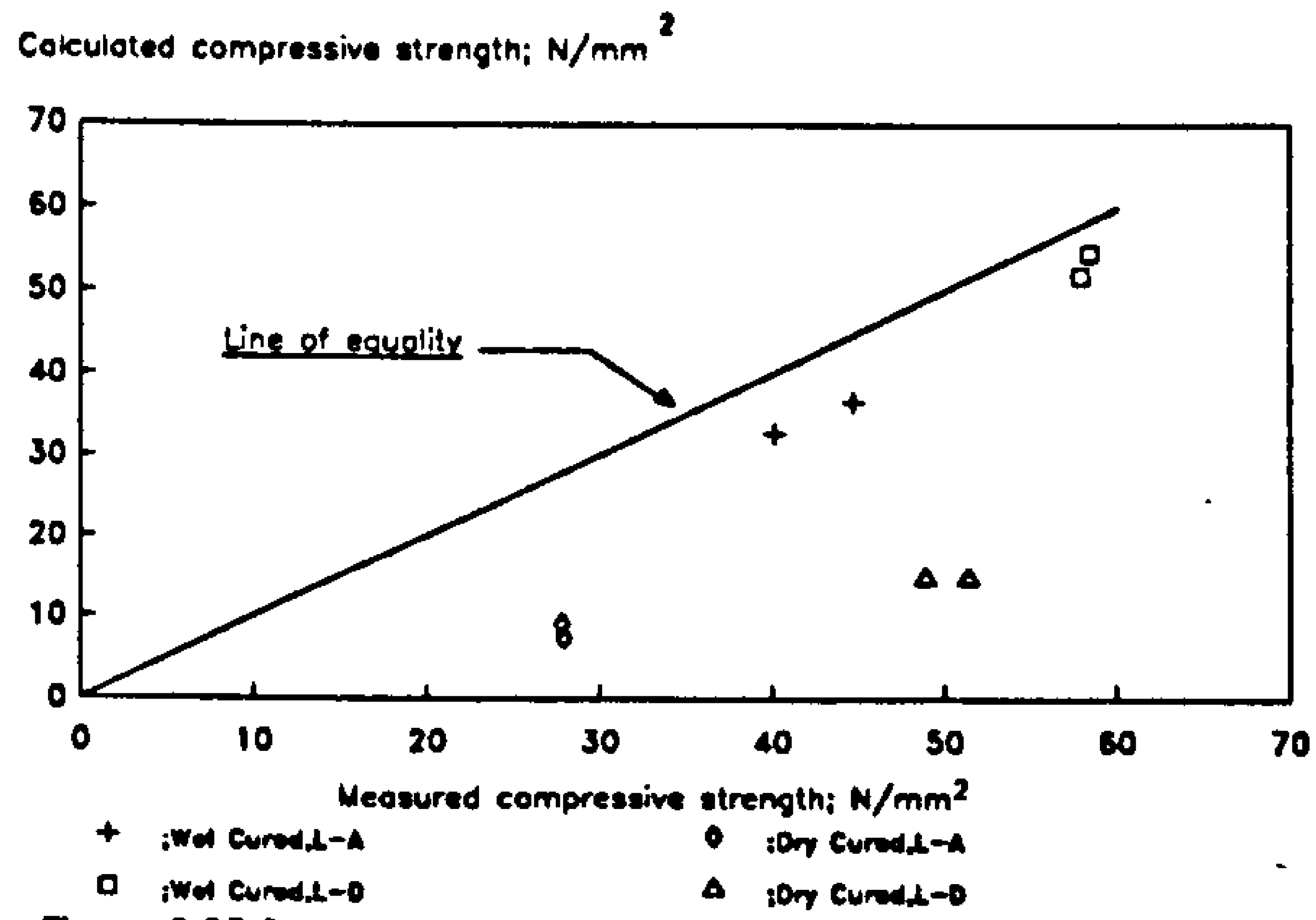


Figure 6.23:Comparison of measured and calculated wet and dry long term compressive strength for fully Lytag concrete

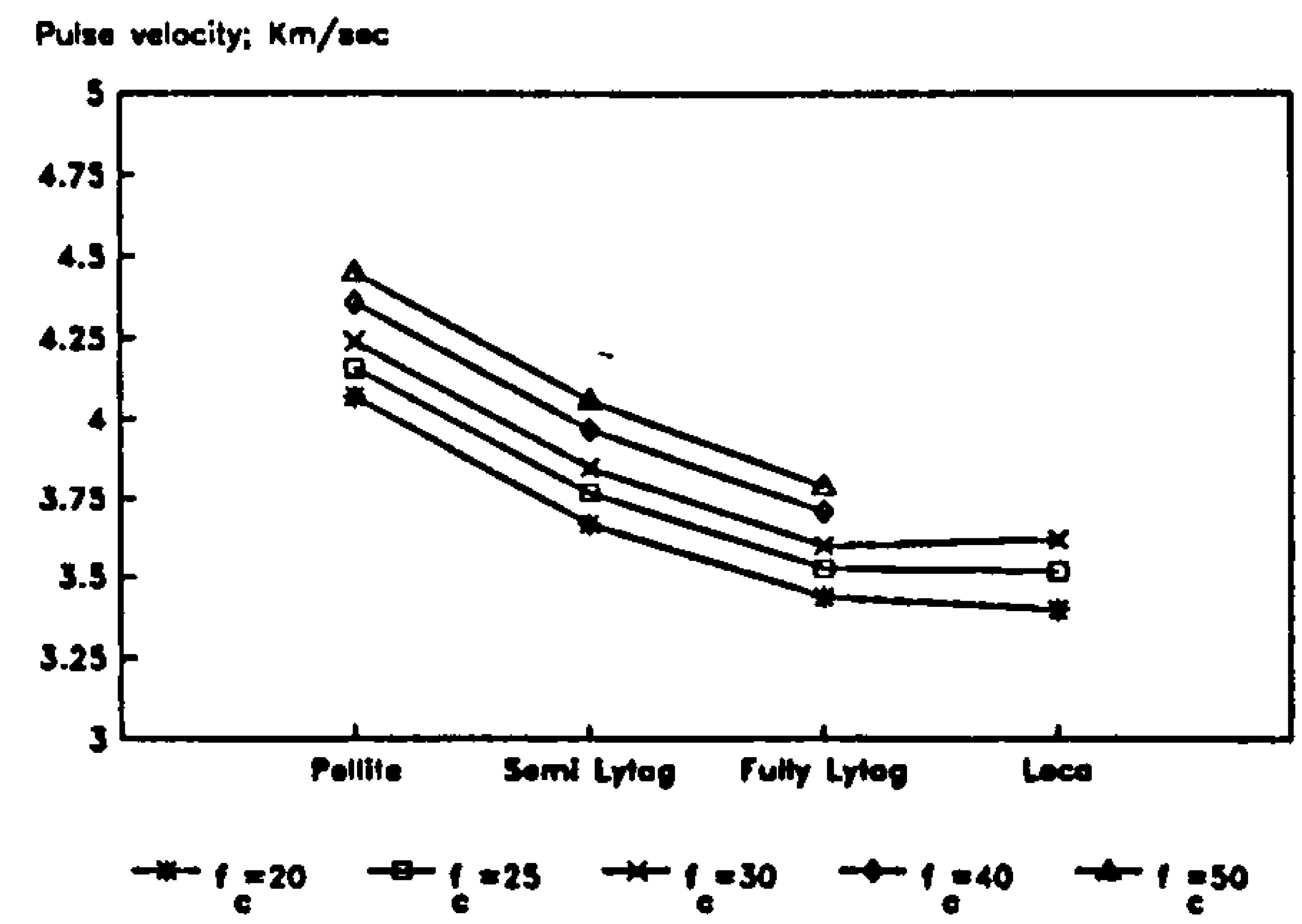


Figure 6.24:Variation in pulse velocity for different types of concrete

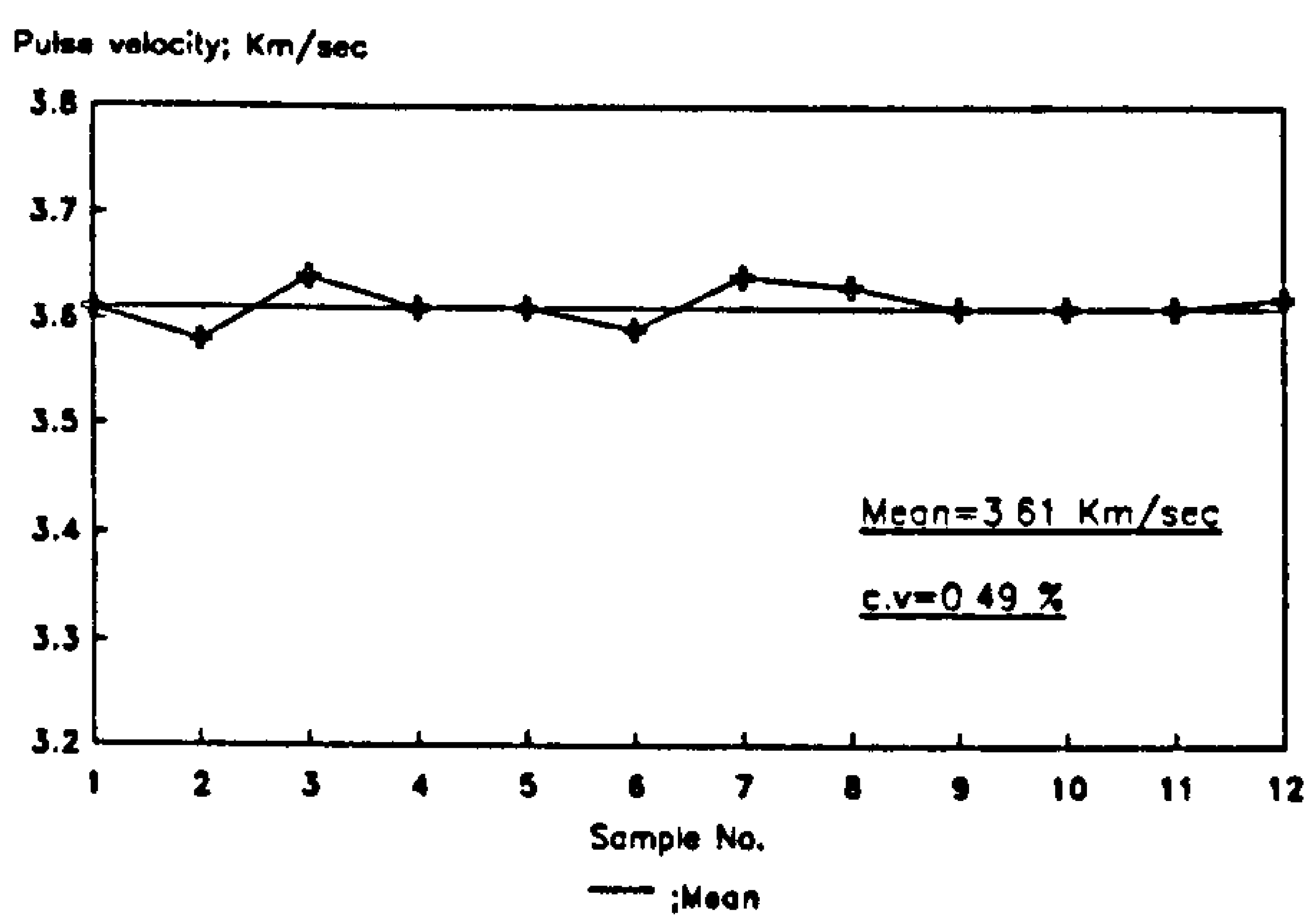


Figure 6.25:Typical variation of pulse velocity on fully Lytag concrete

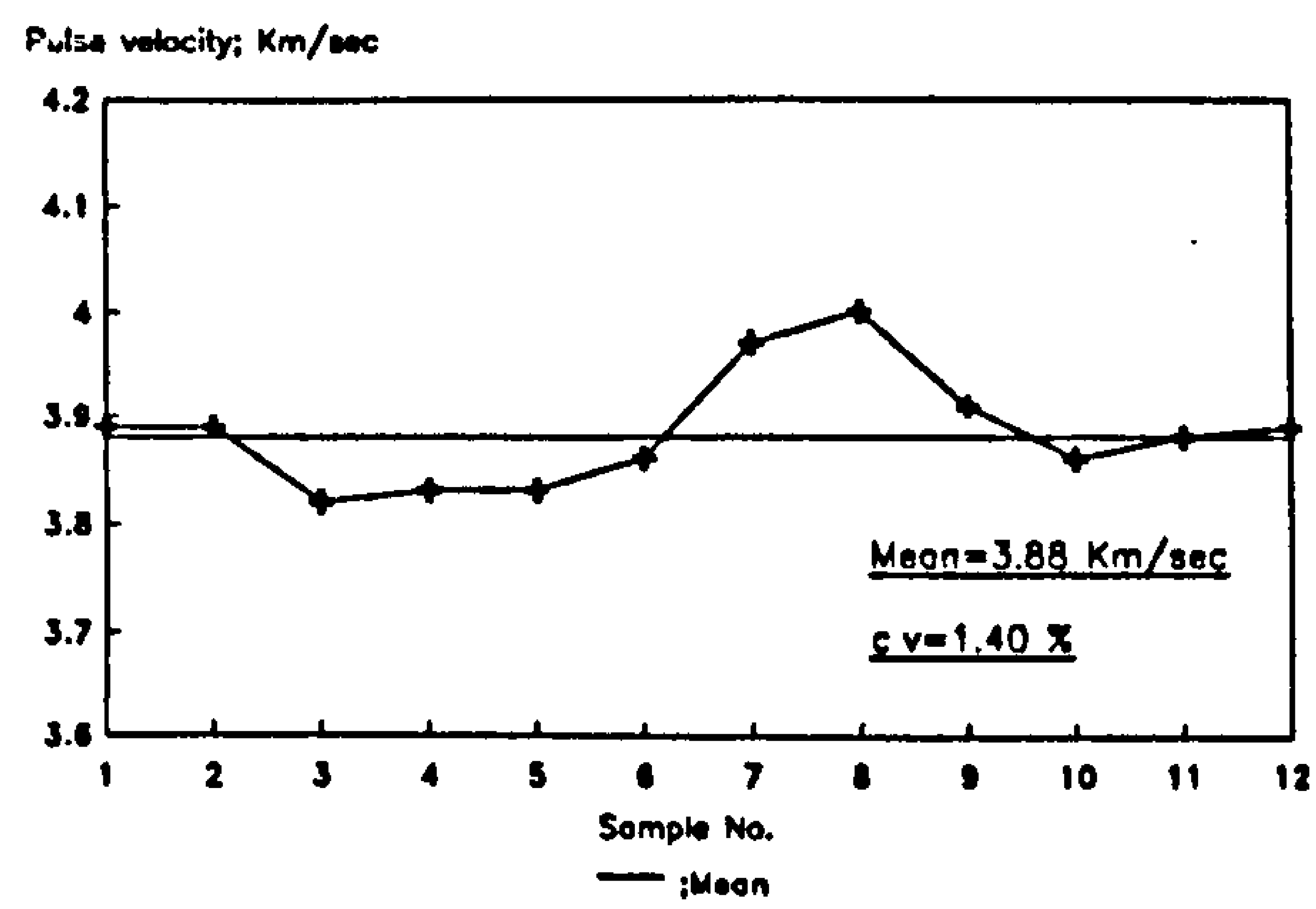


Figure 6.26:Typical variation of pulse velocity on semi Lytag concrete

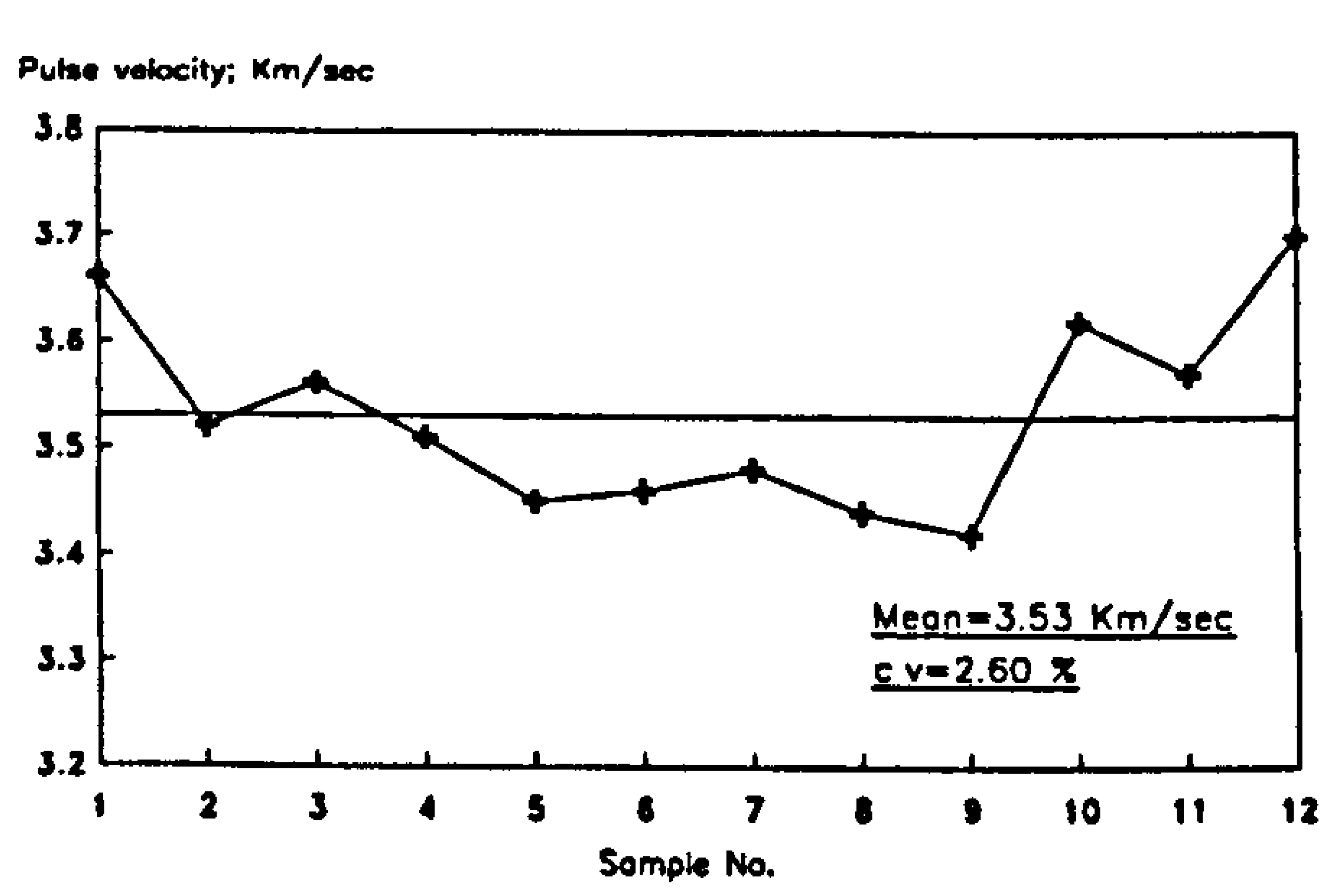


Figure 6.27:Typical variation of pulse velocity on Leca concrete

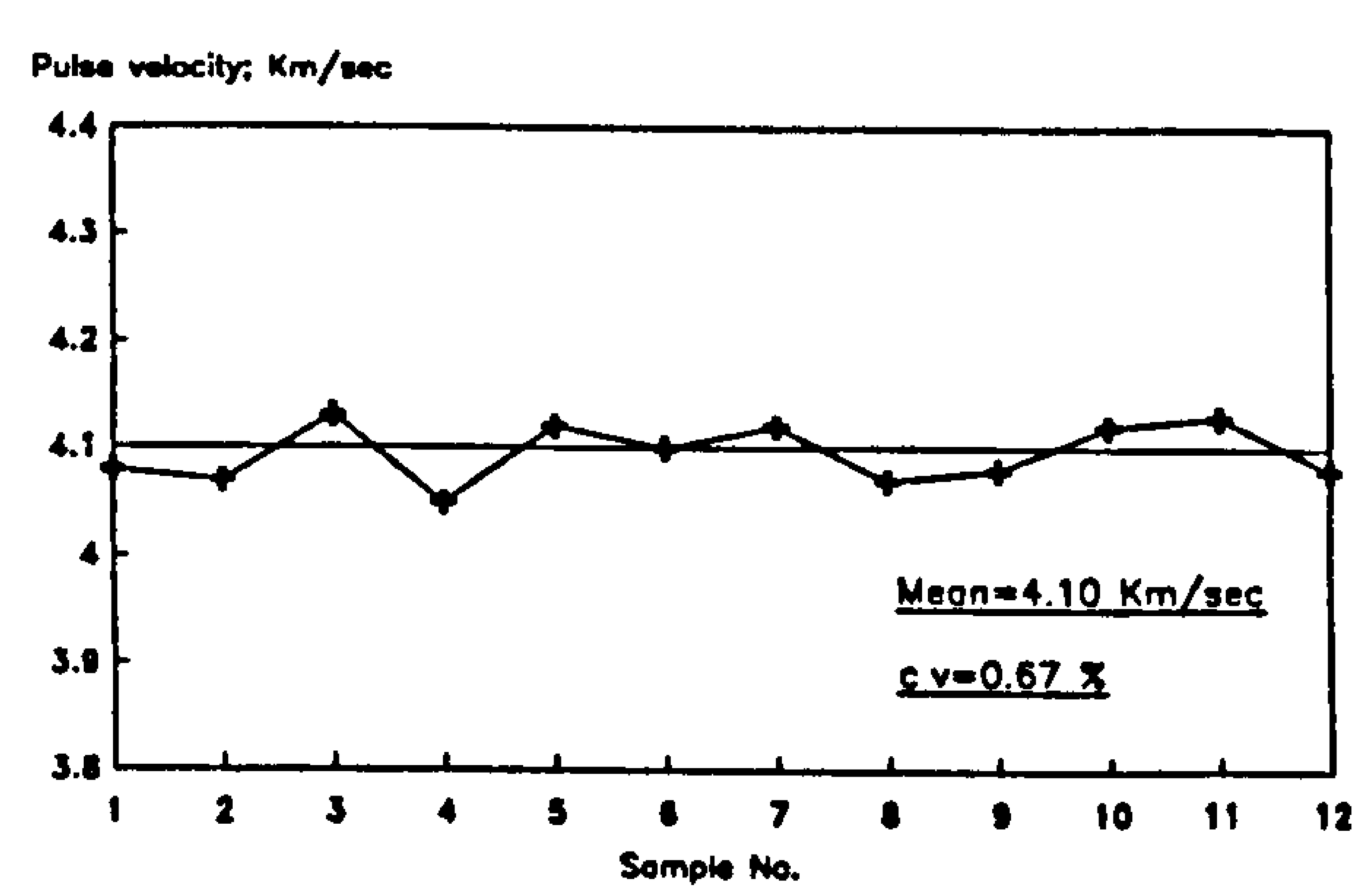


Figure 6.28:Typical variation of pulse velocity on Pellite concrete

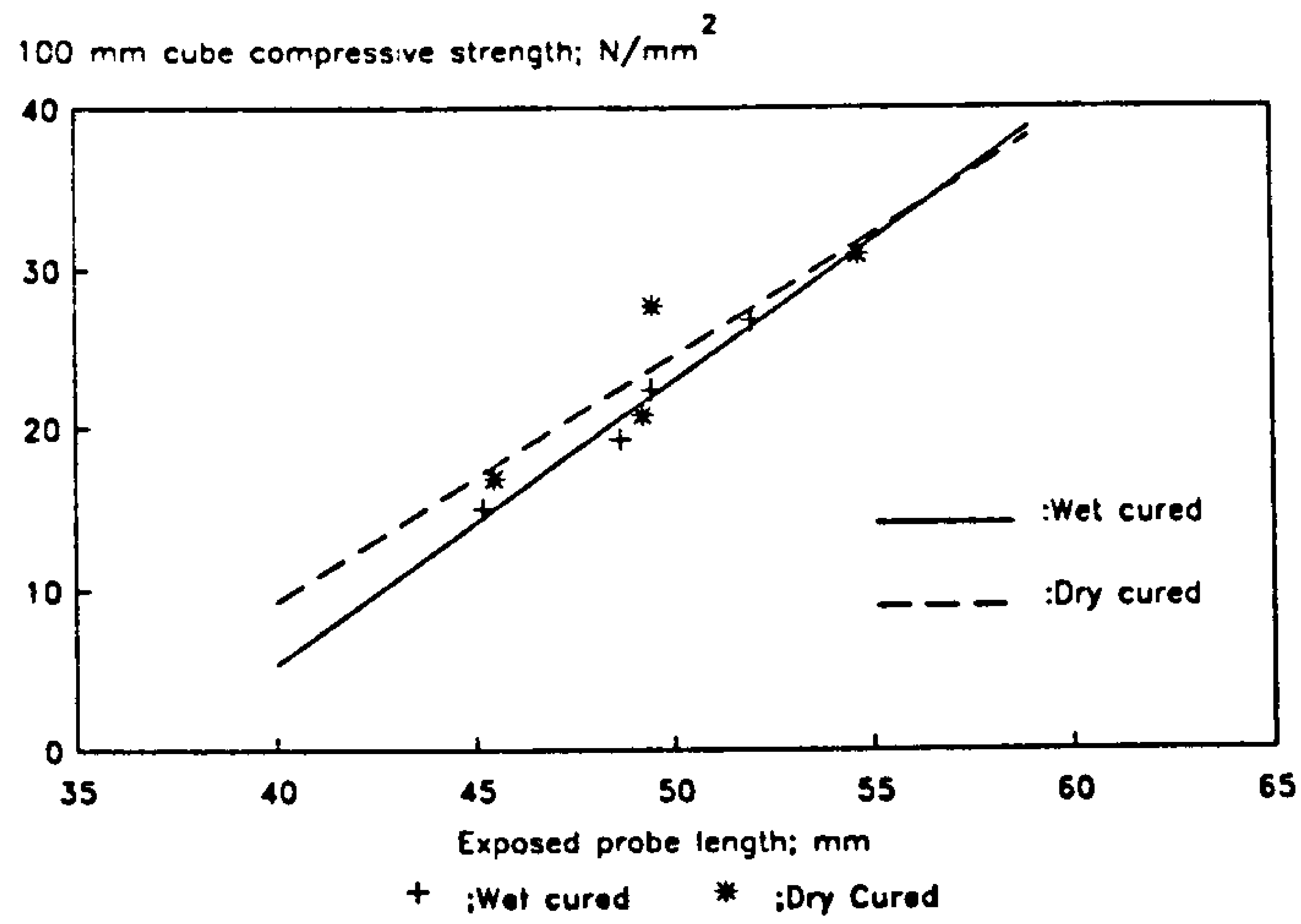


Figure 6.29.Windsor Probe calibrations for 7 day tests of fully Lytag concrete(low power)

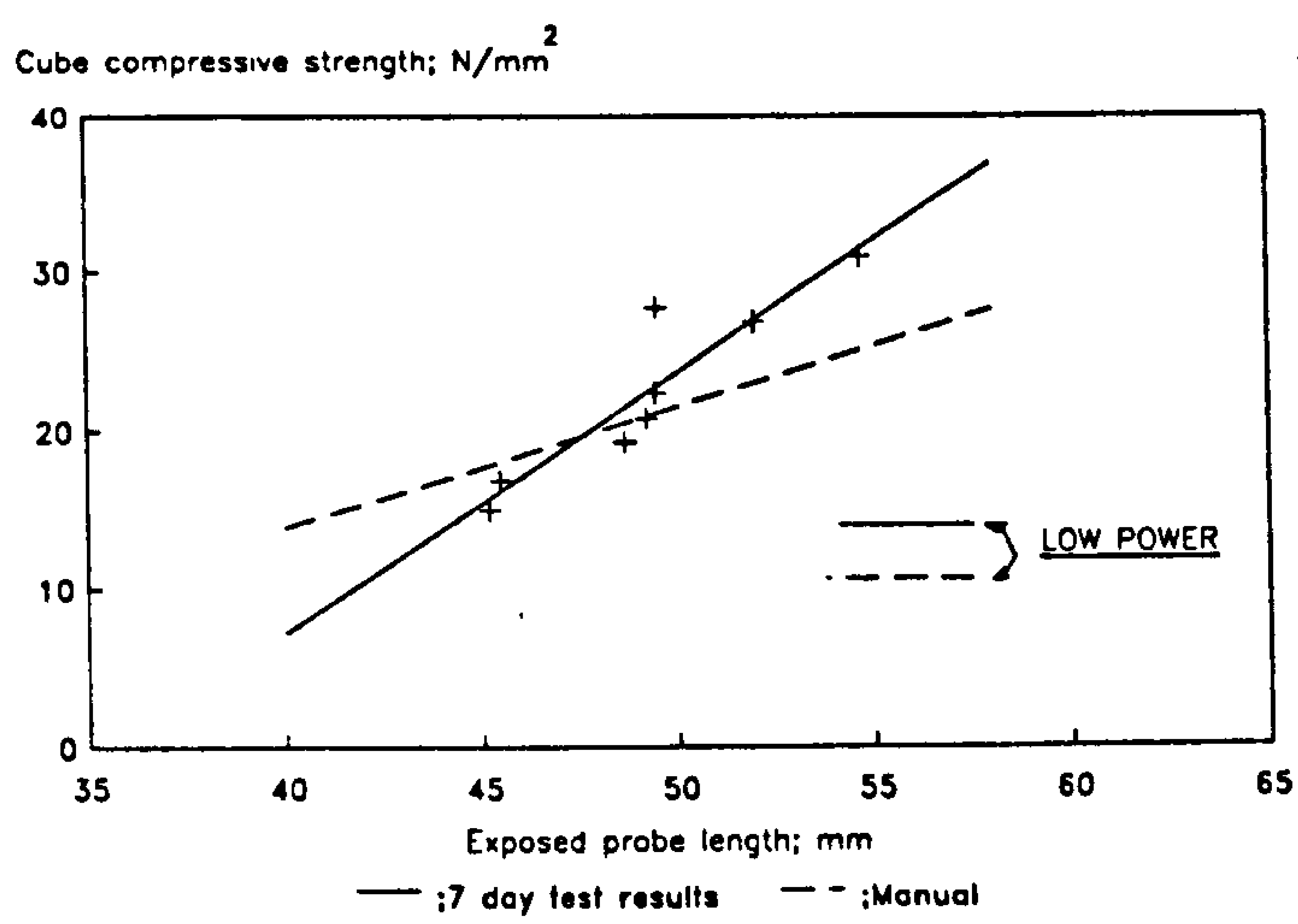


Figure 6.30:Comparison between Windsor Probe manual estimated strength and actual cube strength for fully Lytag concrete

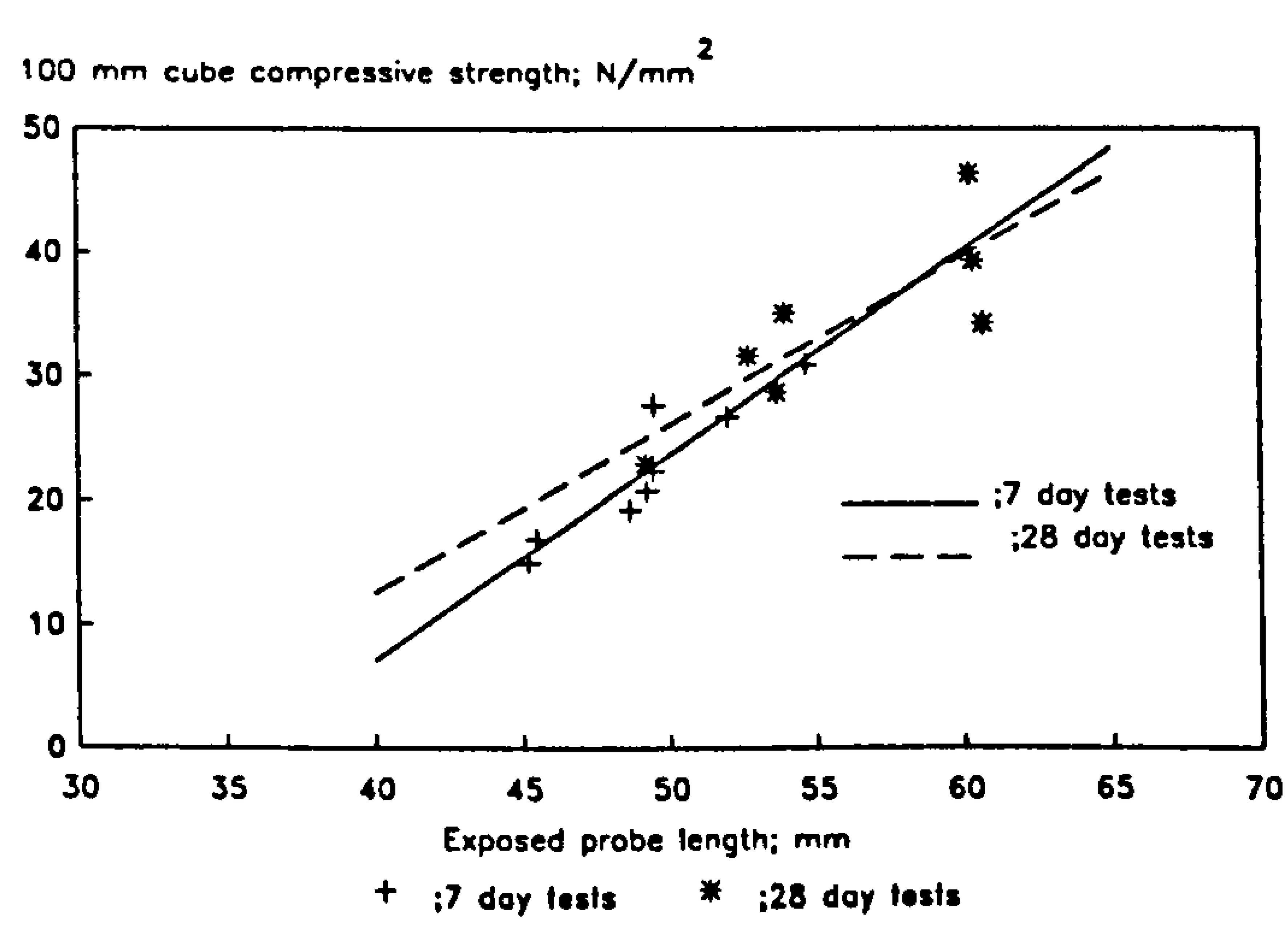


Figure 6.31:Windsor Probe calibrations for 7 and 28 day tests for fully Lytag concrete (low power)

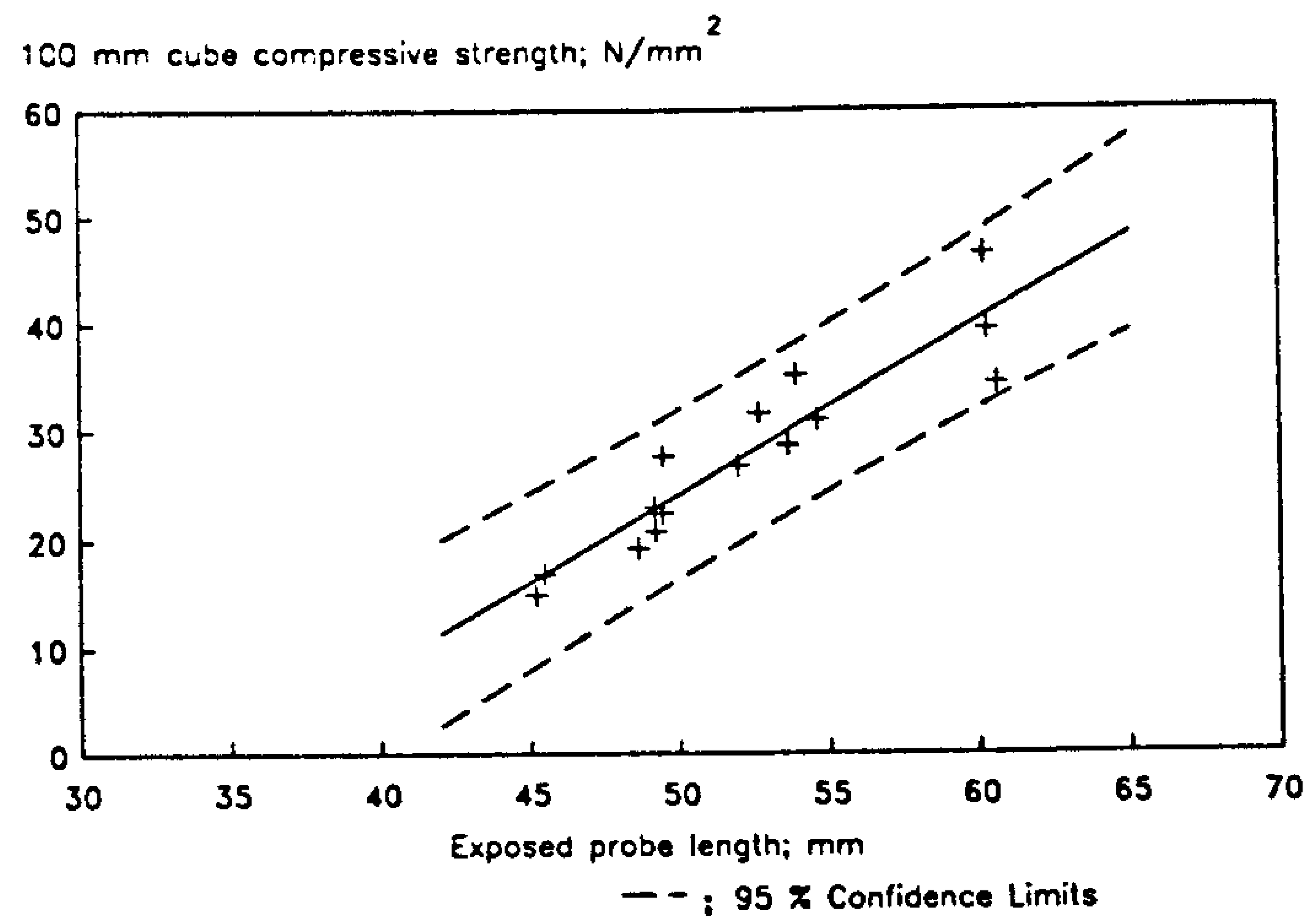


Figure 6.32.Windsor Probe calibration for fully Lytag concrete(low power)

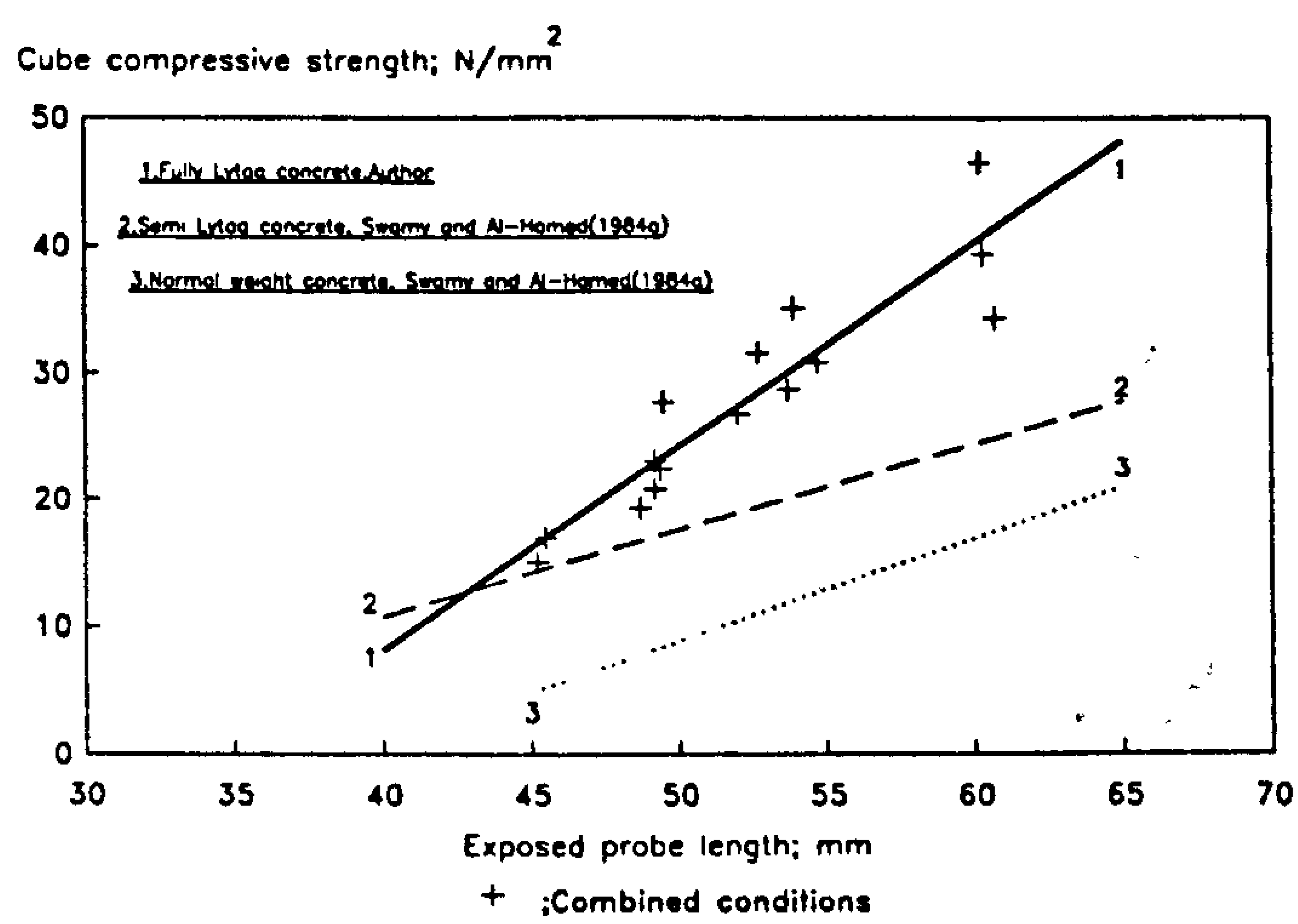


Figure 6.33.Comparison of Windsor Probe calibrations for different types of concrete(low power)

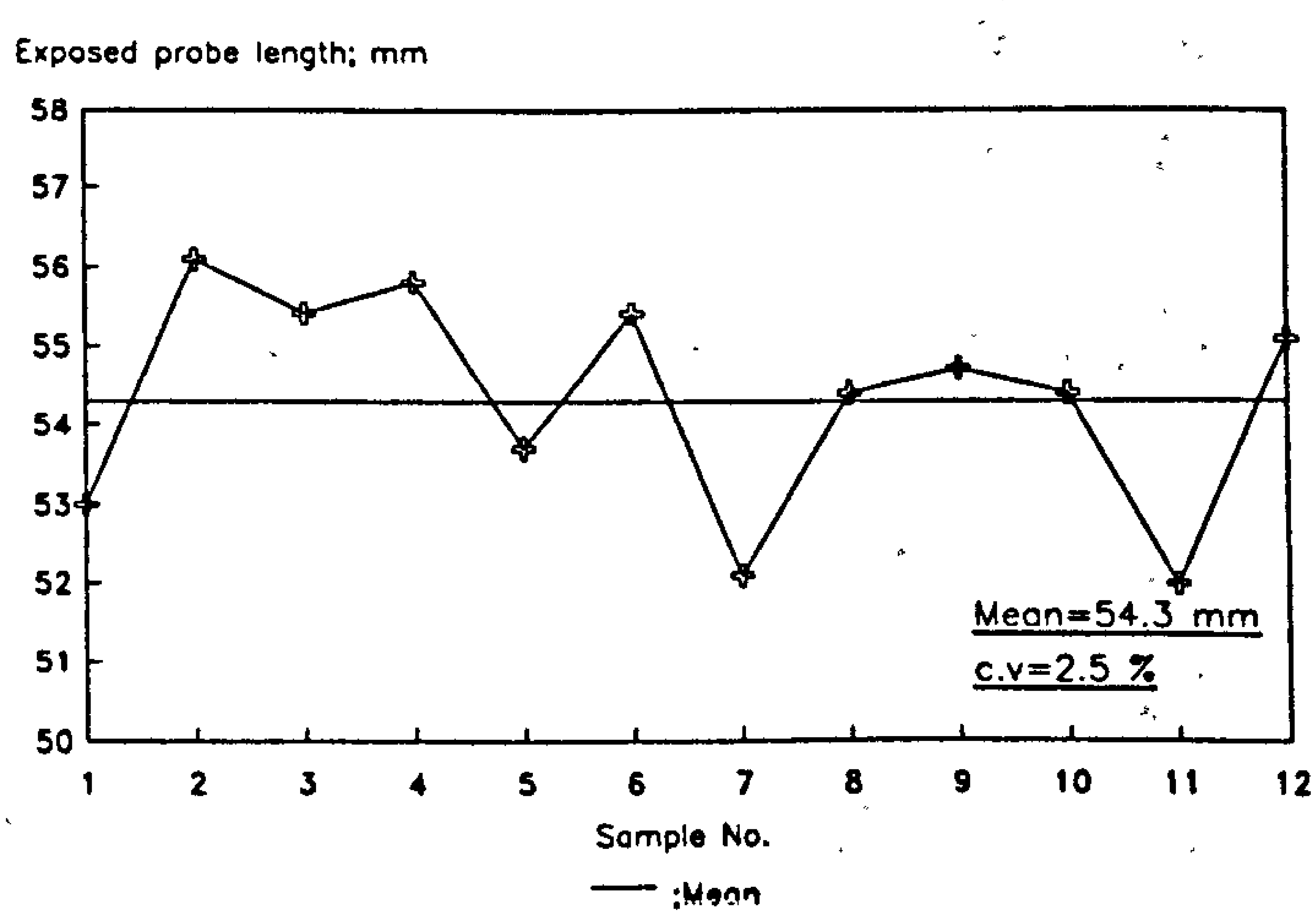


Figure 6.34:Typical variation of Windsor Probe tests on fully Lytag concrete

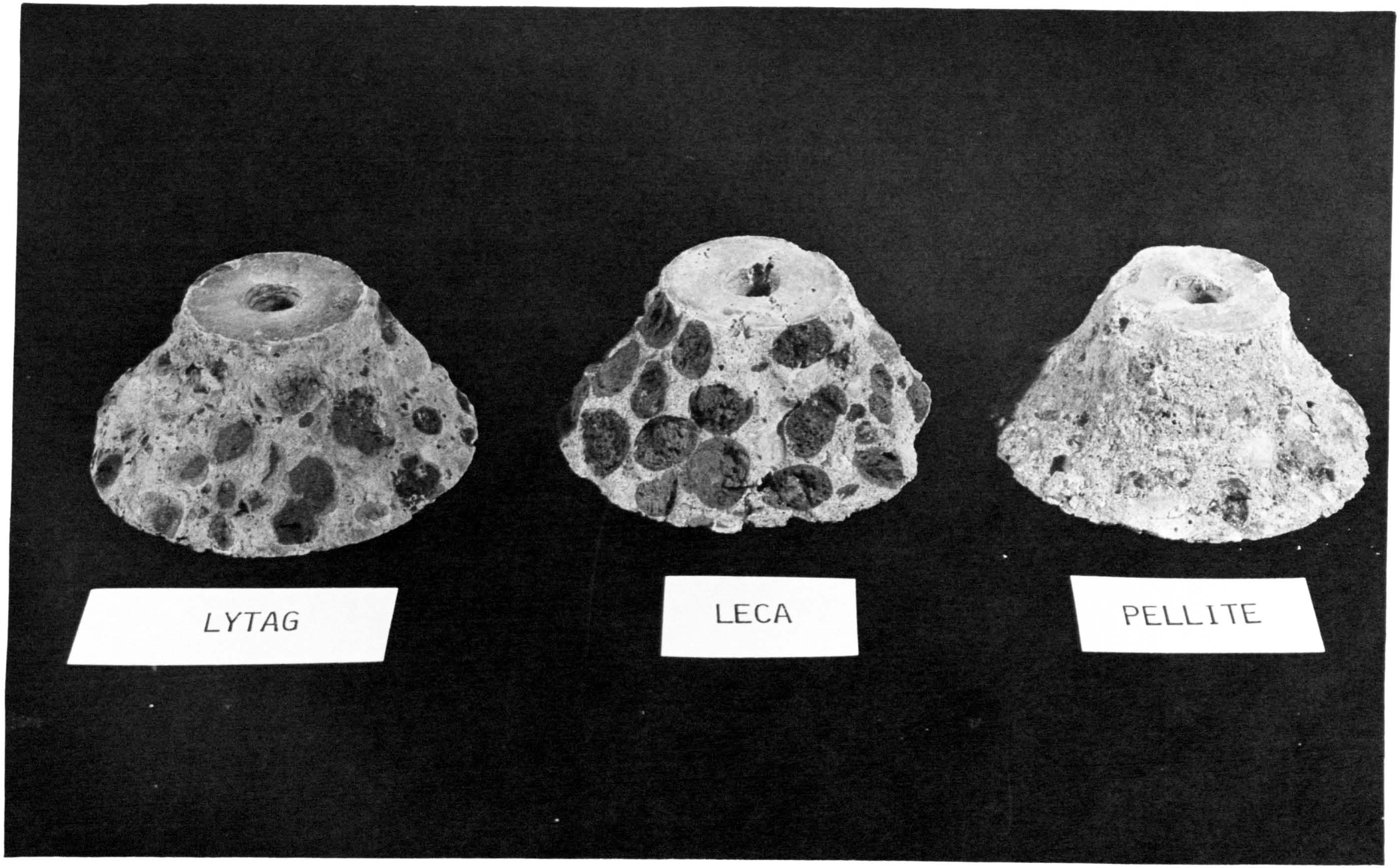


Figure 6.35: Typical truncated cones of lightweight concretes extracted through the pull out tests

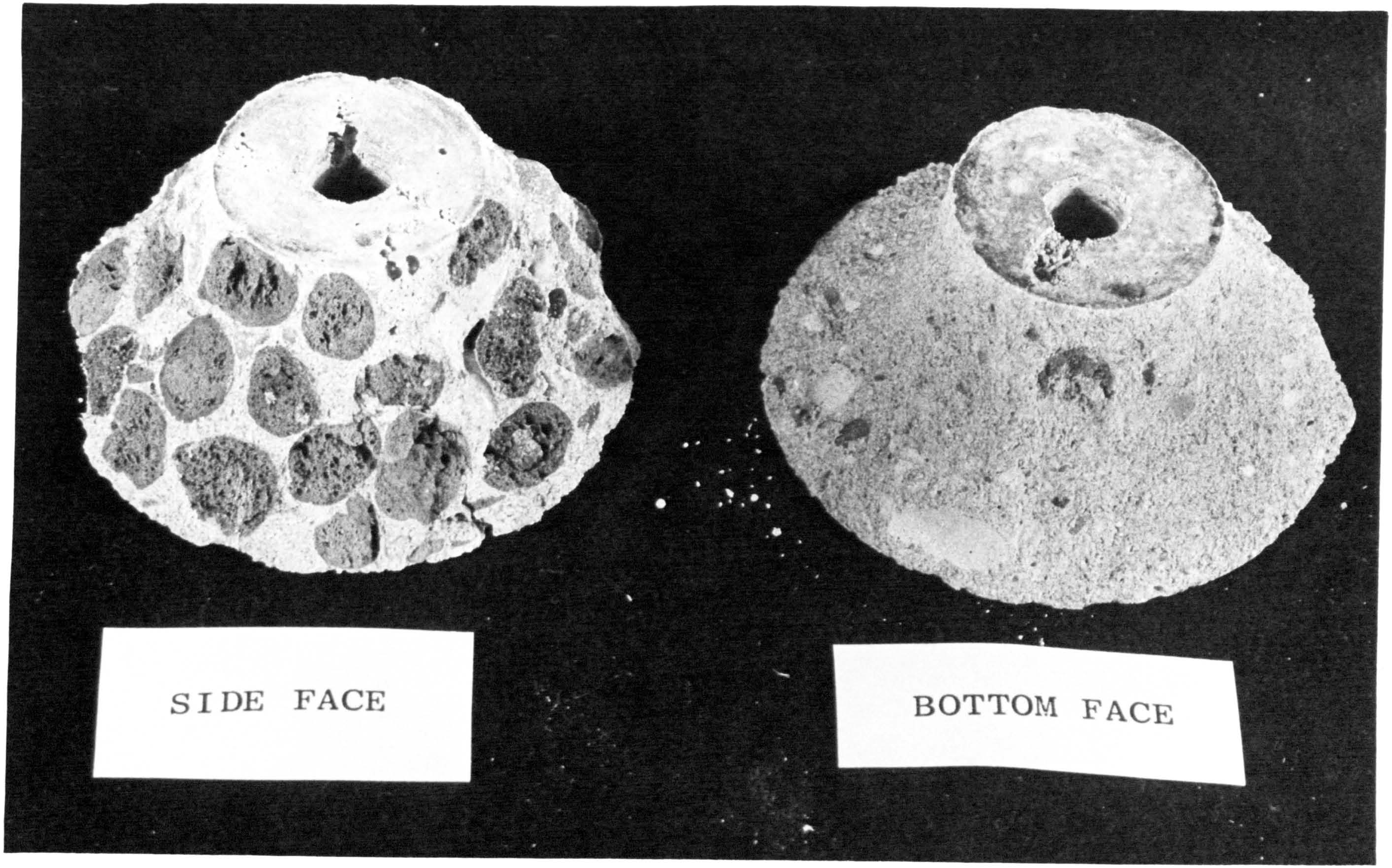


Figure 6.36: Typical pull out truncated cones extracted from side and bottom faces of a Leca concrete specimen

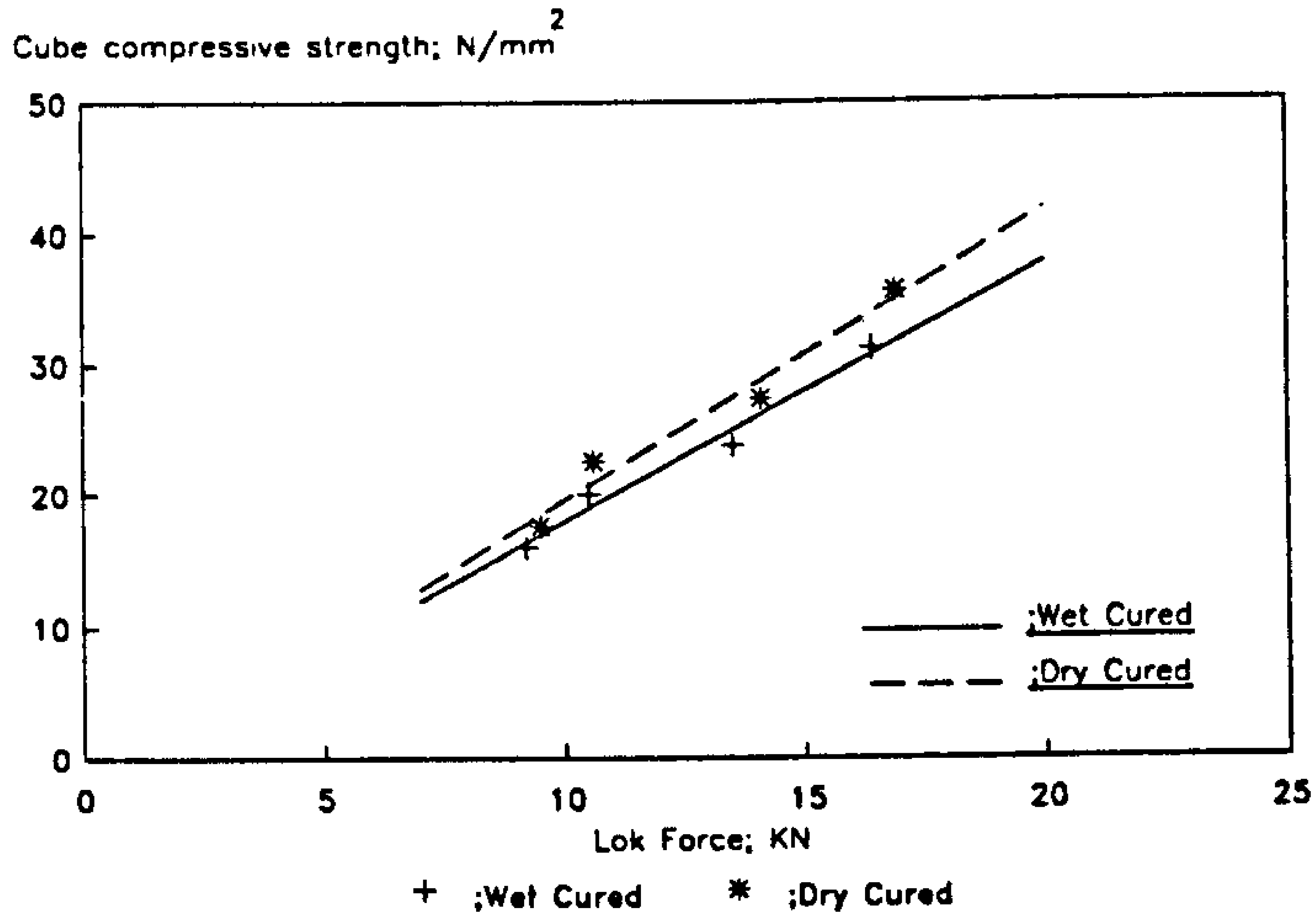


Figure 6.37:Relationship between Lok force and compressive strength for 7 day tests of fully Lytag concrete

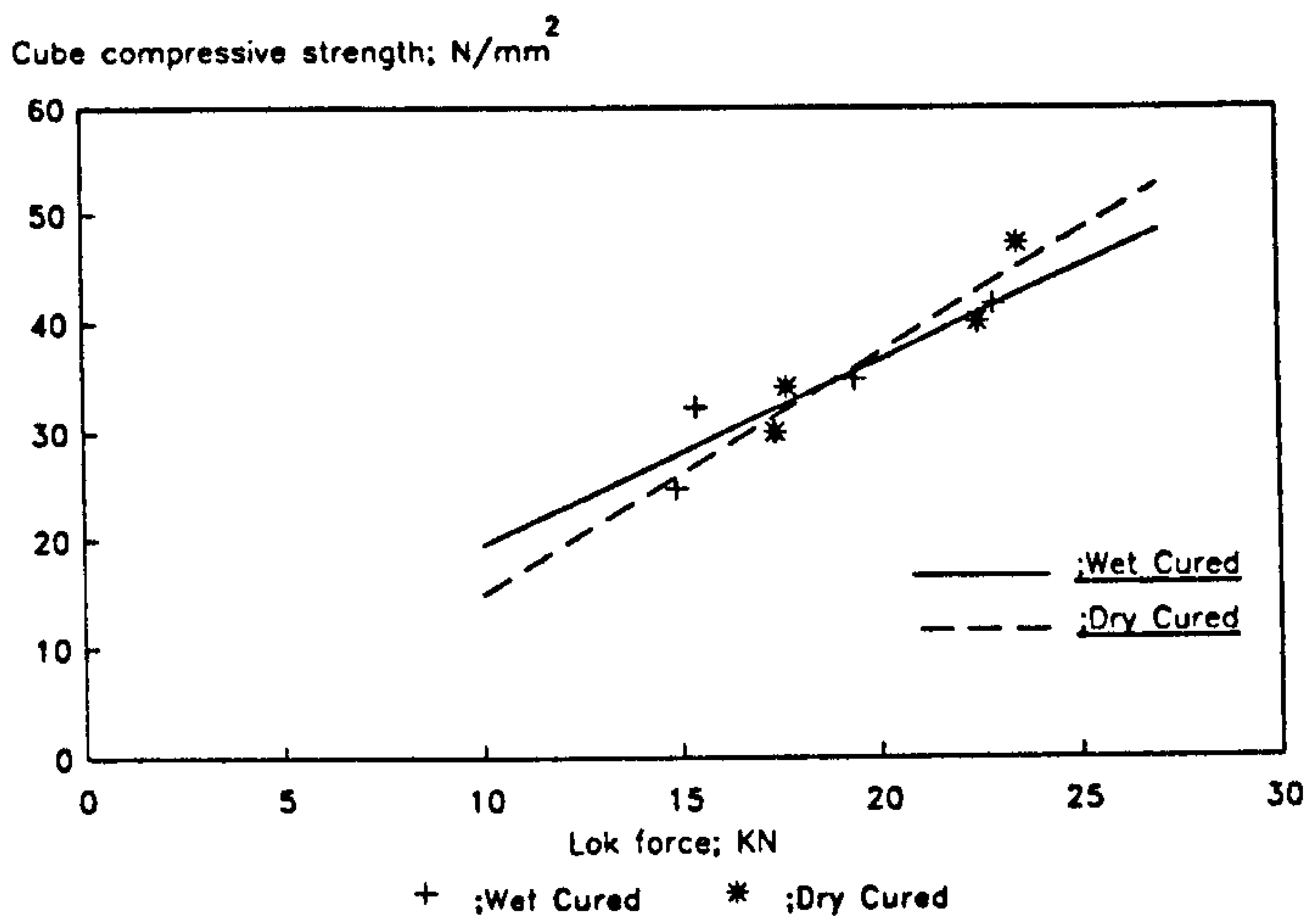


Figure 6.38:Relationship between Lok force and compressive strength for 28 day tests of fully Lytag concrete

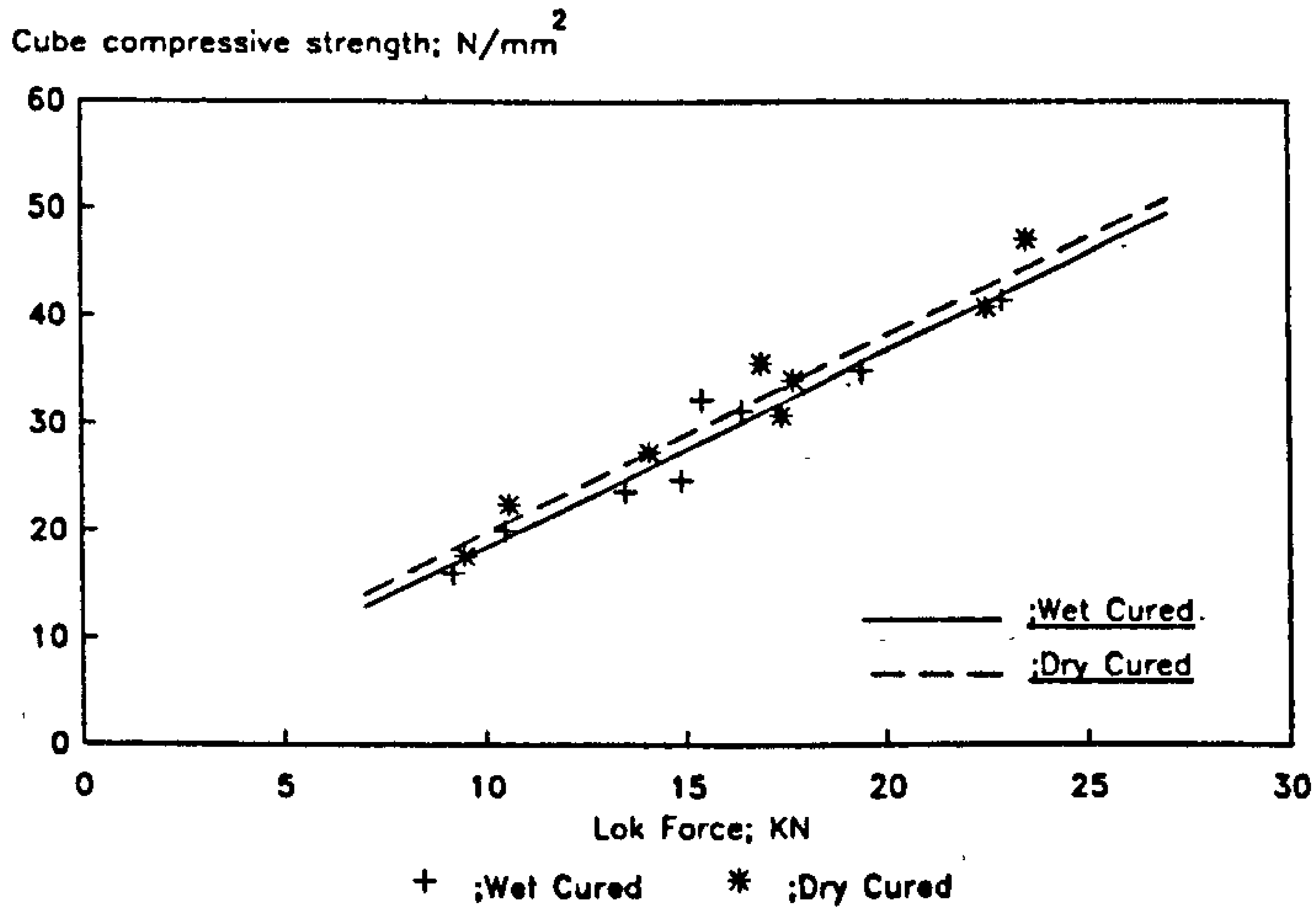


Figure 6.39:Relationship between Lok force and compressive strength for 7 and 28 day tests of fully Lytag concrete

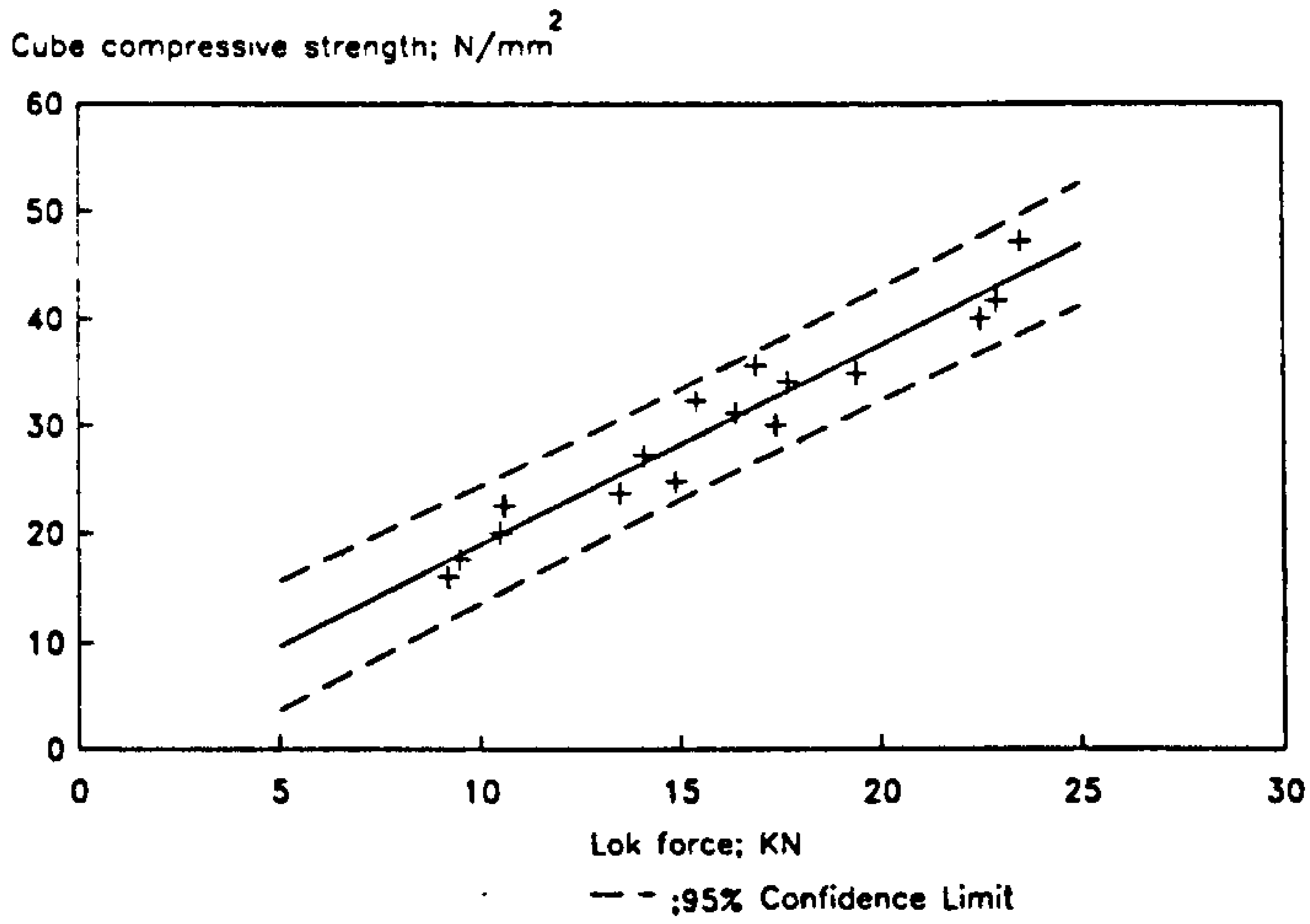


Figure 6.40:Short term relationship between Lok force and compressive strength for fully Lytag concrete

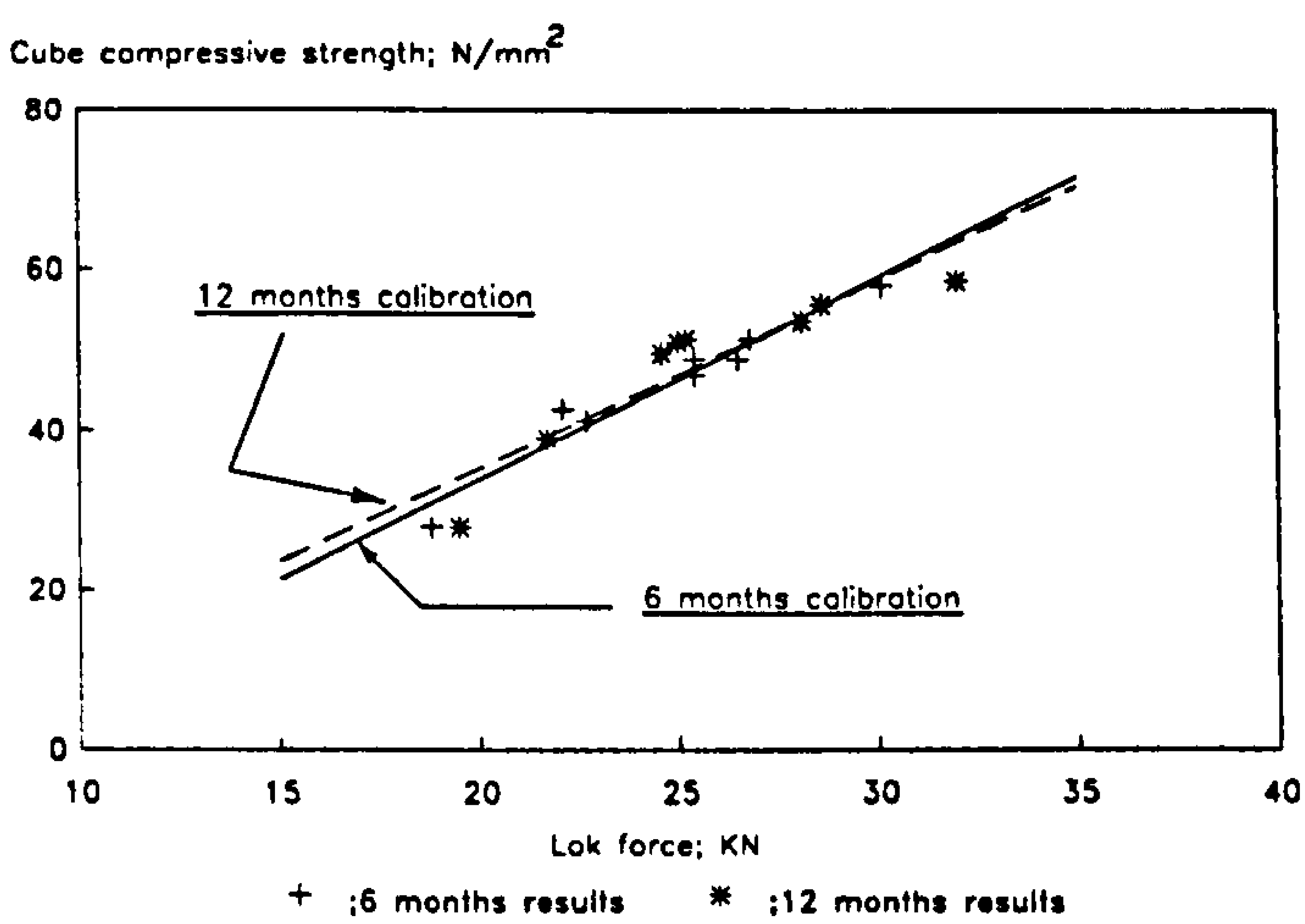


Figure 6.41:Long term relationship between Lok force and compressive strength for fully Lytag concrete

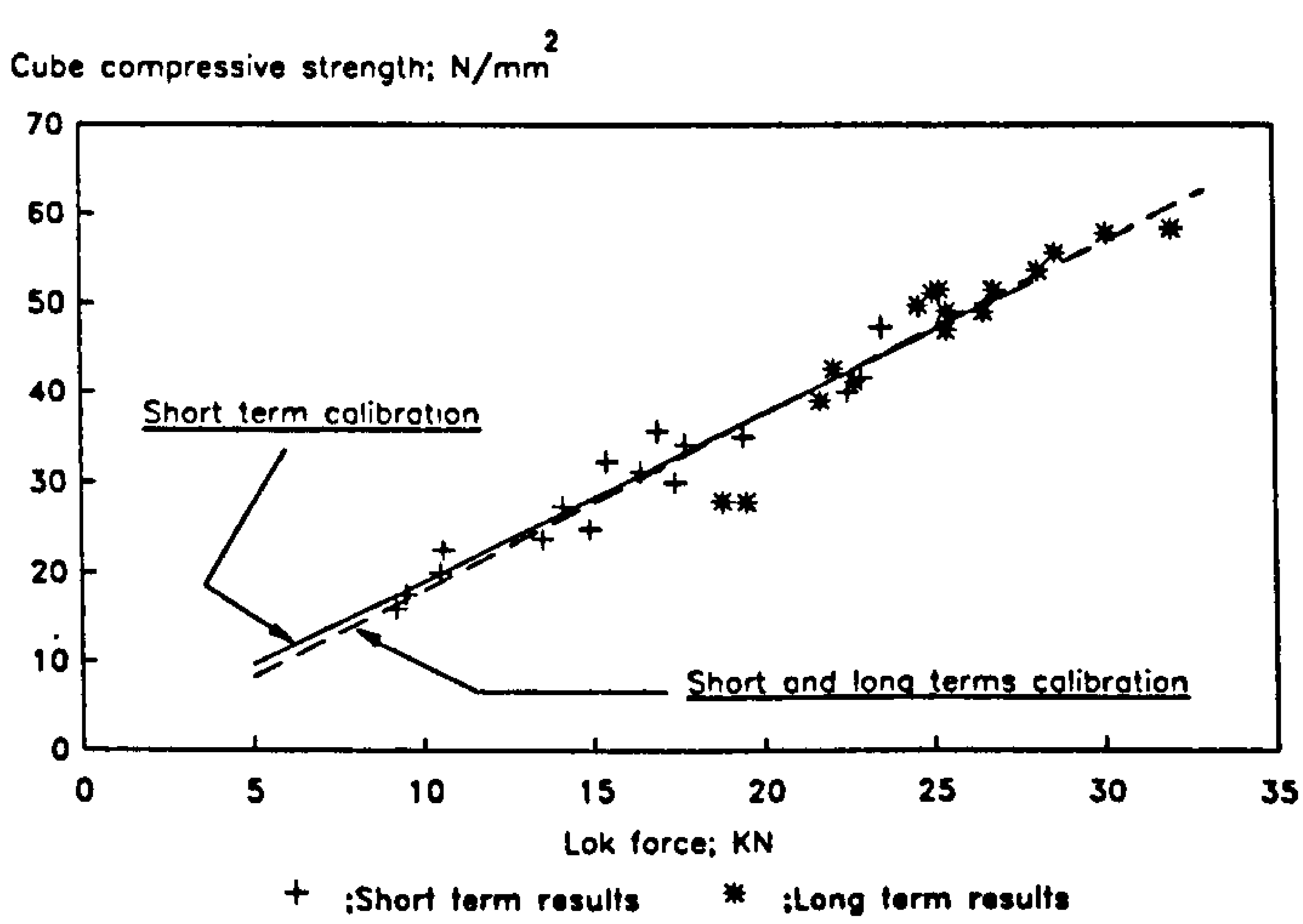


Figure 6.42:Comparison of given calibrations for fully Lytag concrete

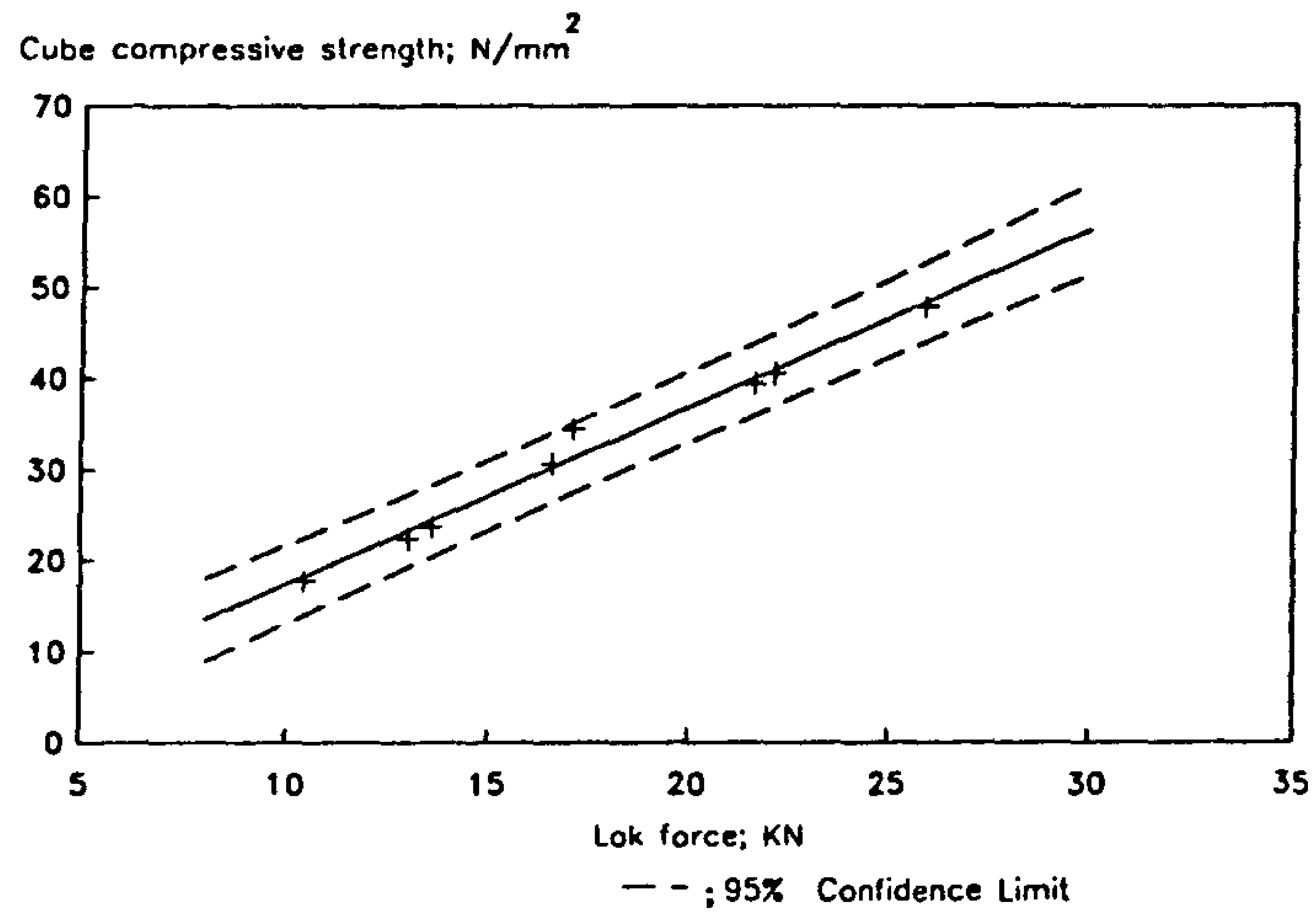


Figure 6.43.Relationship between Lok force and compressive strength for semi Lytag concrete

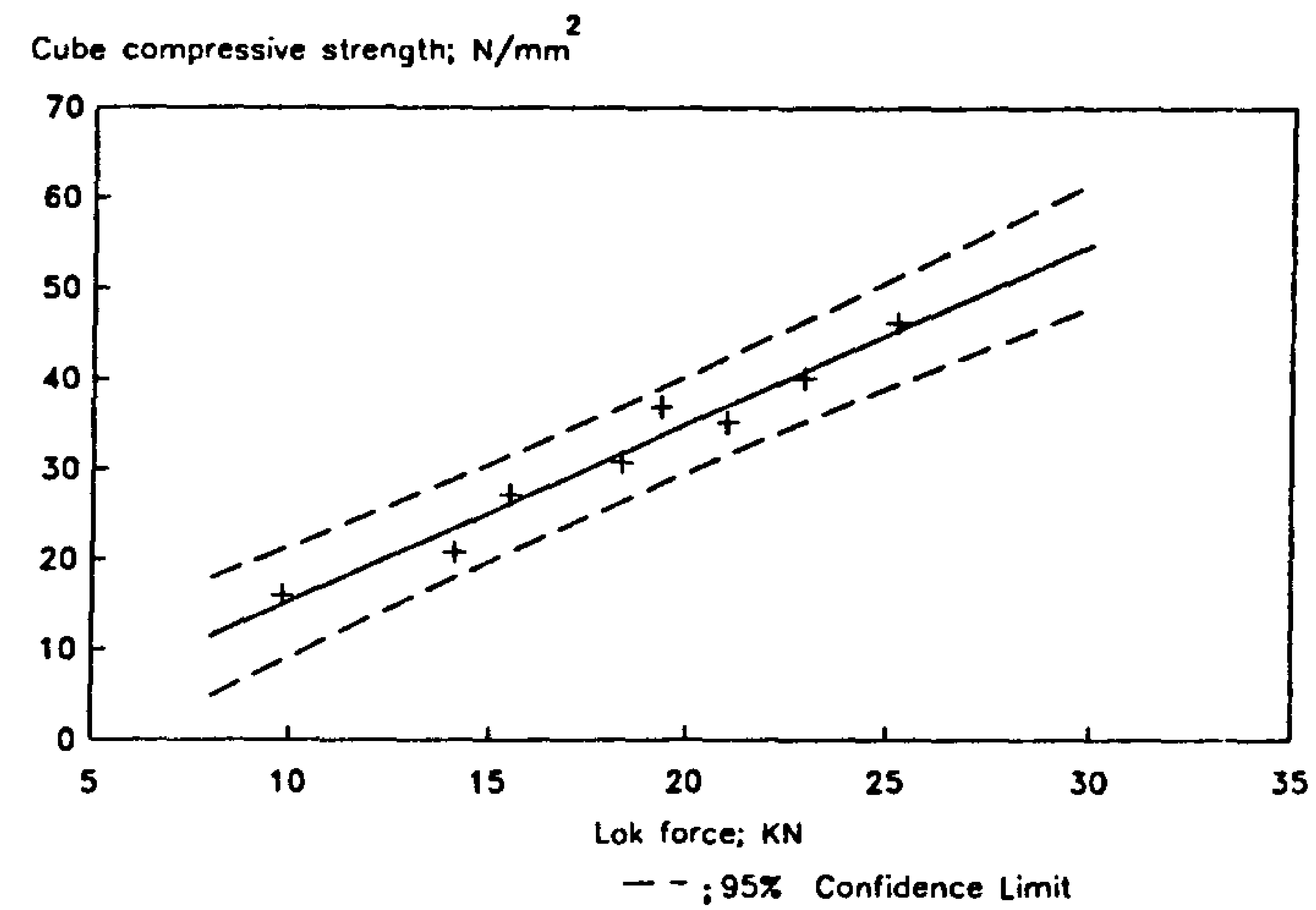


Figure 6.45:Relationship between Lok force and compressive strength for Pellite concrete

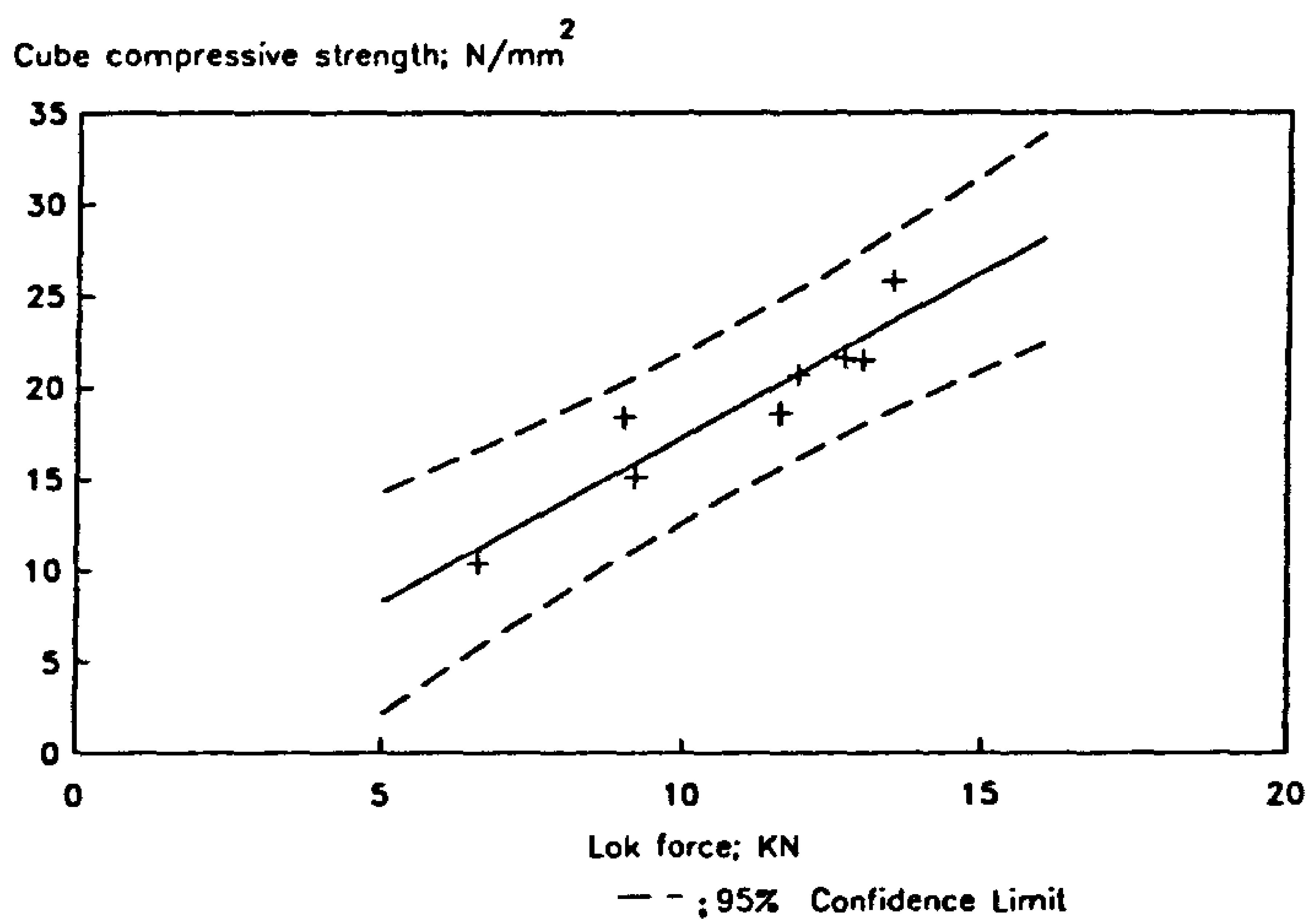


Figure 6.44:Relationship between Lok force and compressive strength for Leca concrete

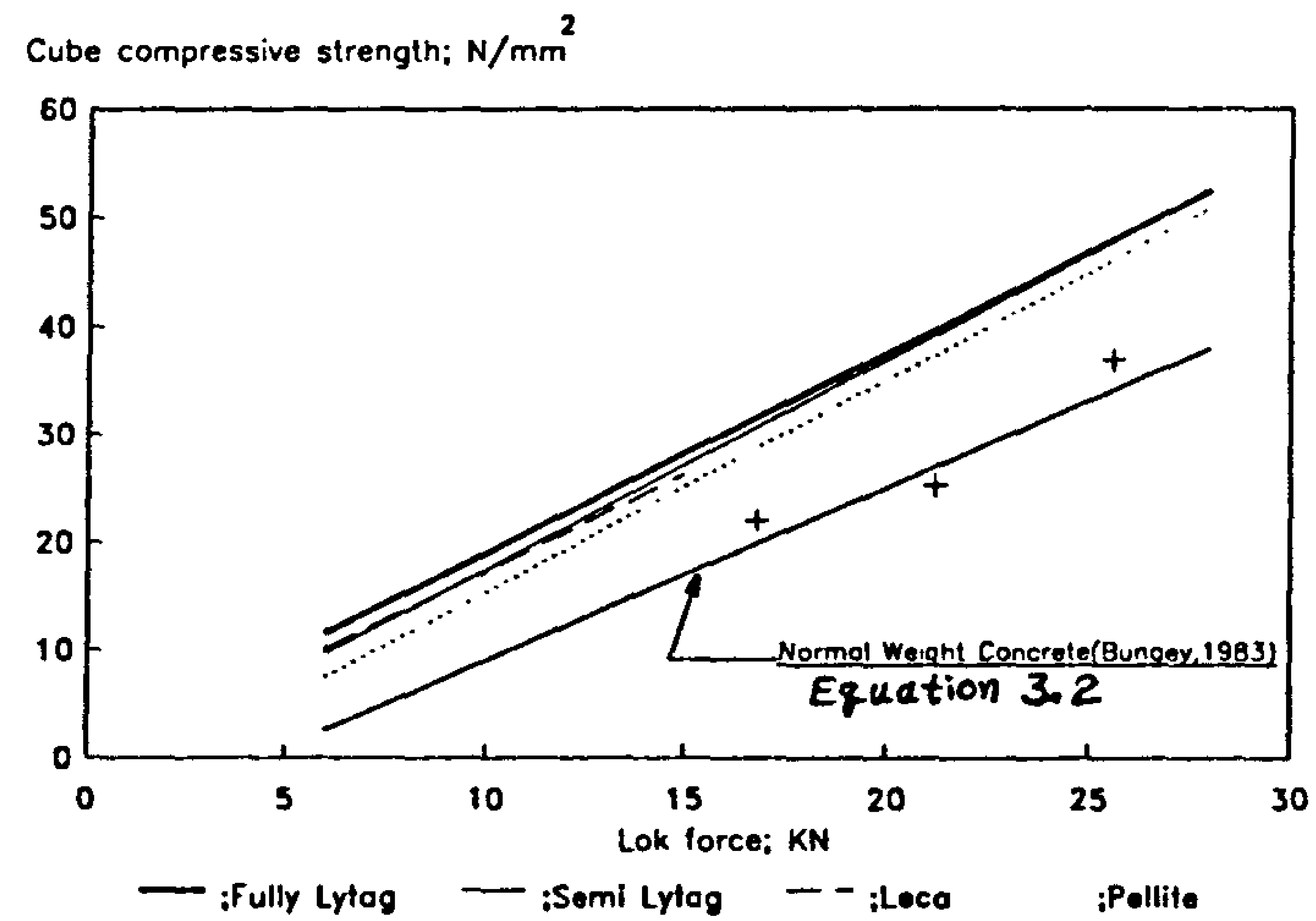


Figure 6.46:Comparison of relationships between Lok force and compressive strength for different types of concrete

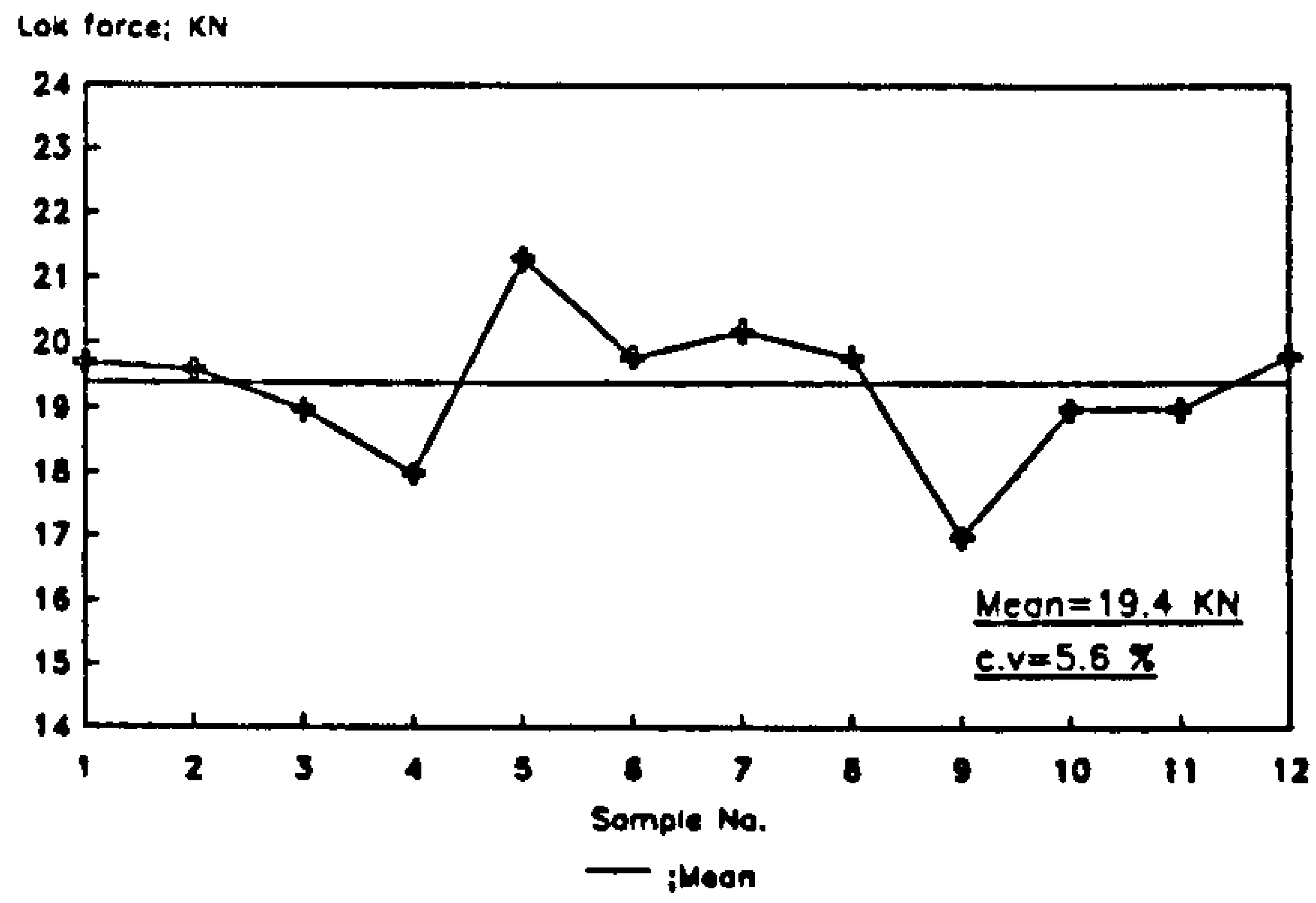


Figure 6.47:Typical variation of Lok force on fully Lytag concrete

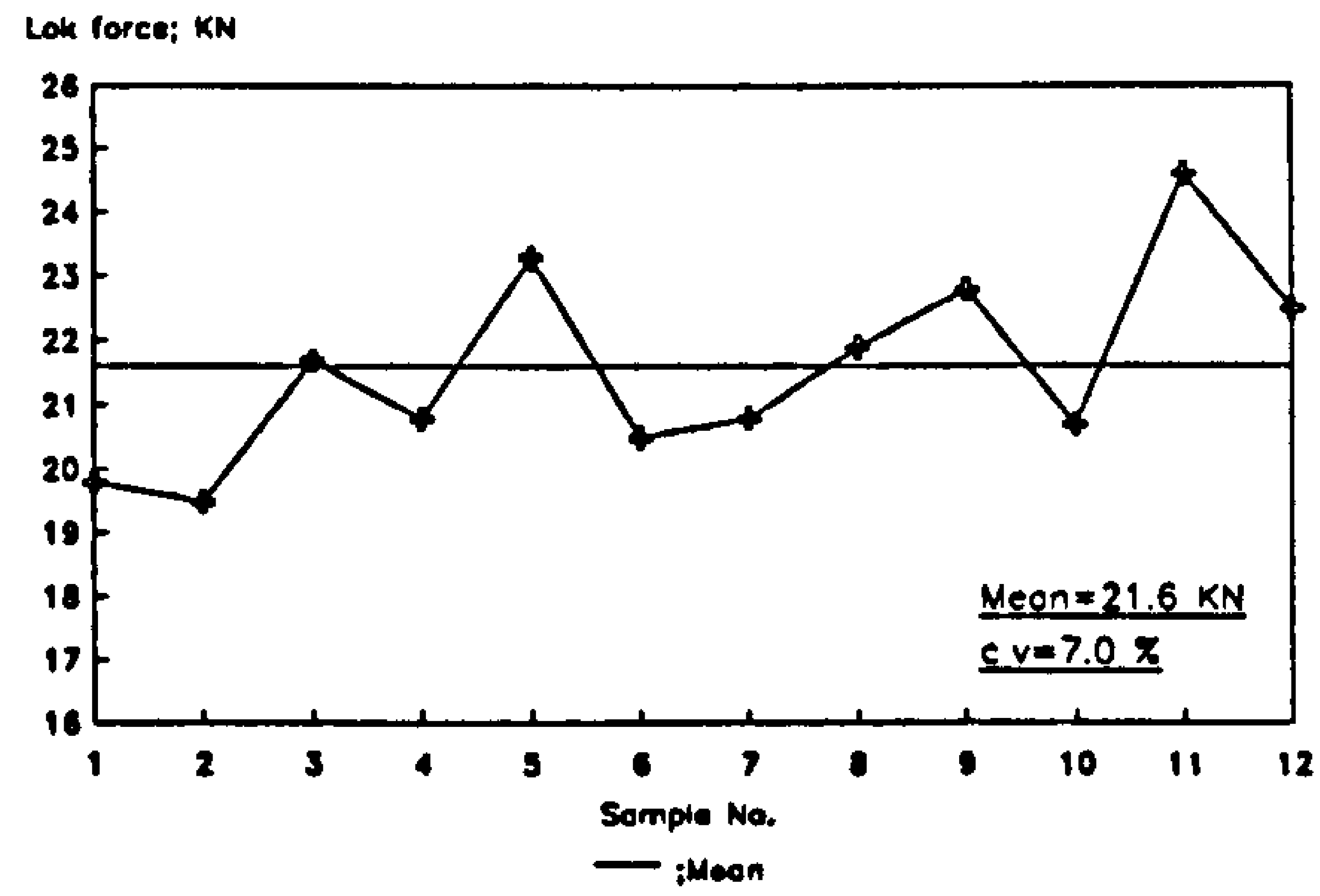


Figure 6.48.Typical variation of Lok force on semi Lytag concrete

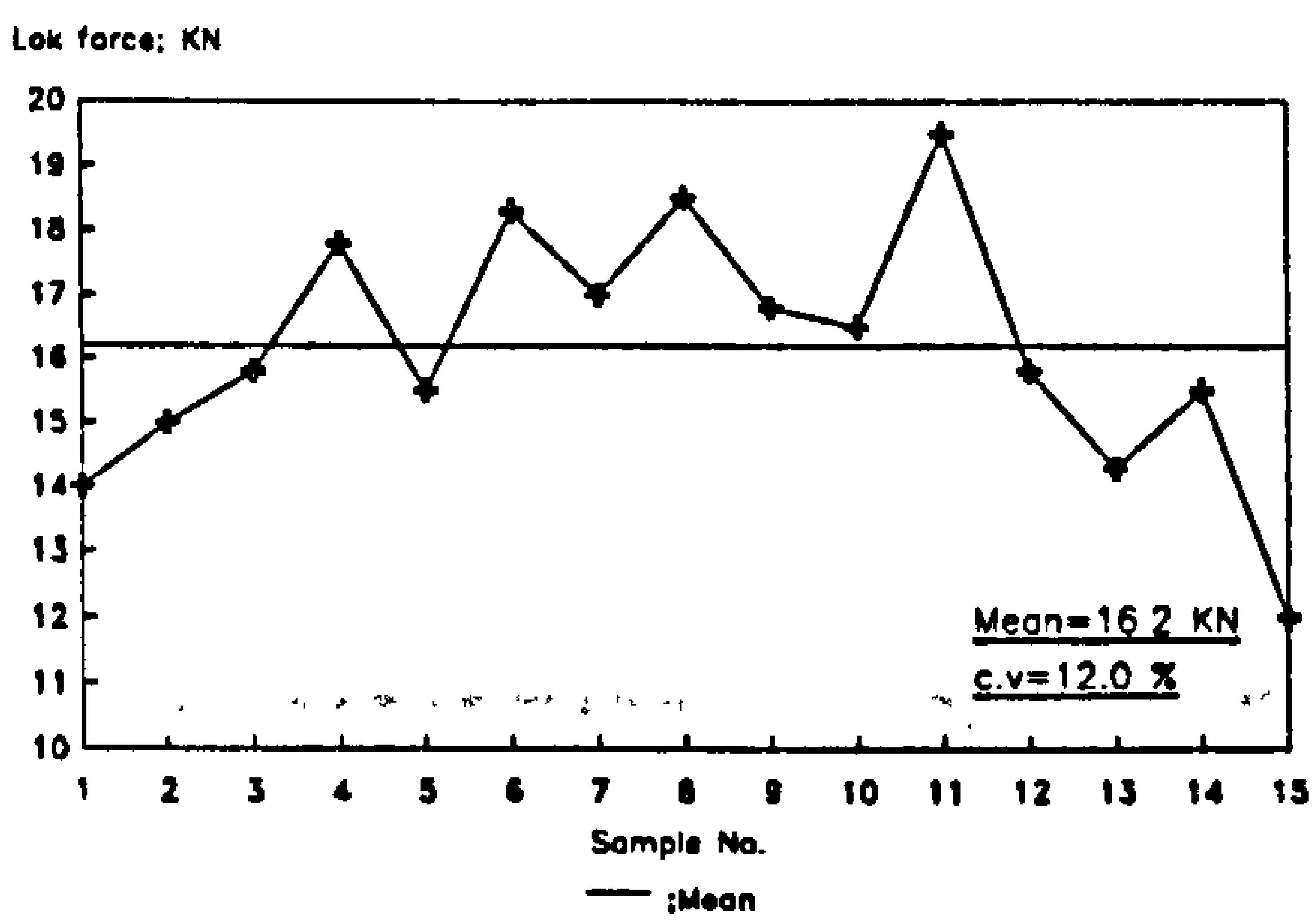


Figure 6.49:Typical variation of Lok force on Leca concrete

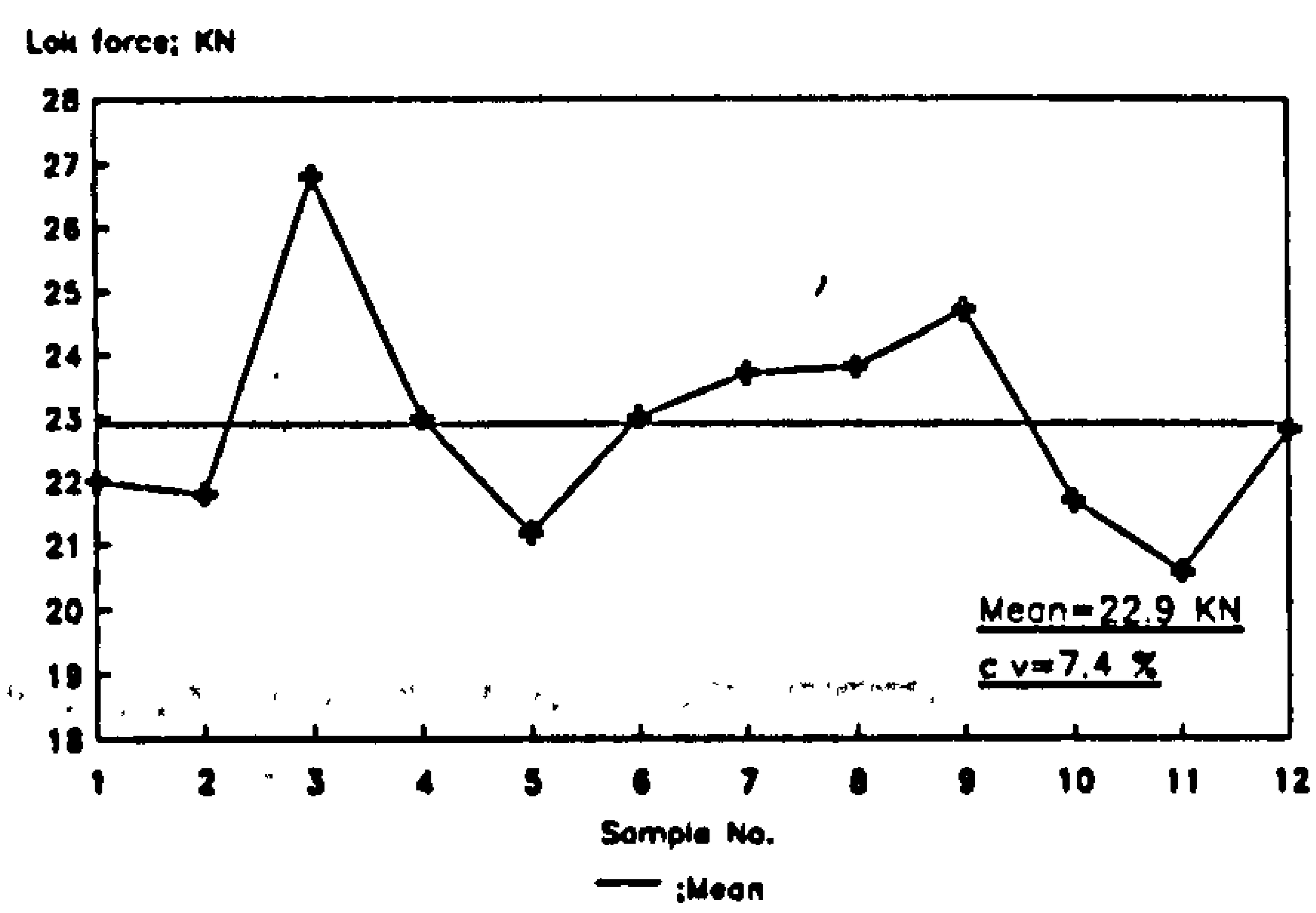


Figure 6.50:Typical variation of Lok force on Pellite concrete

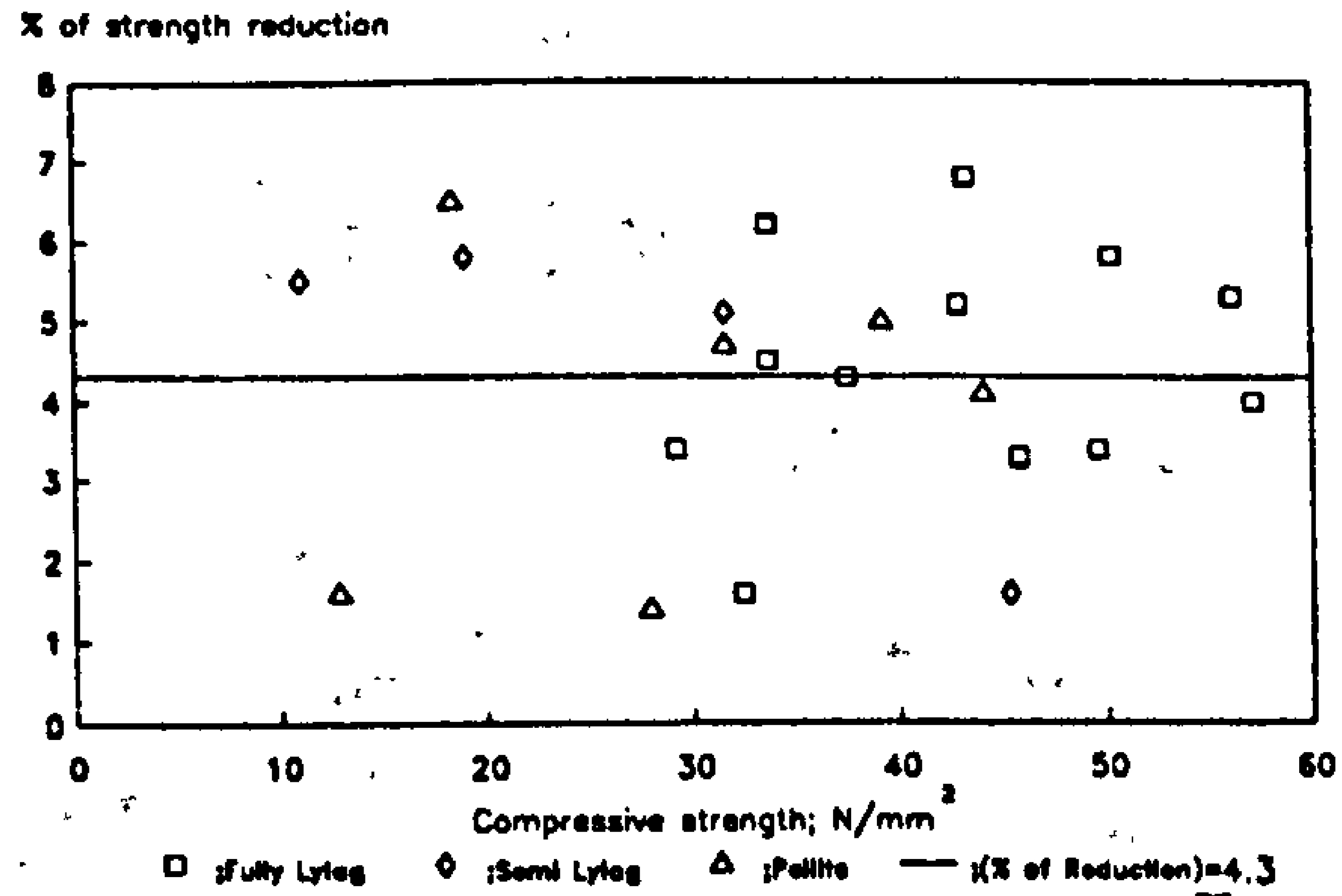
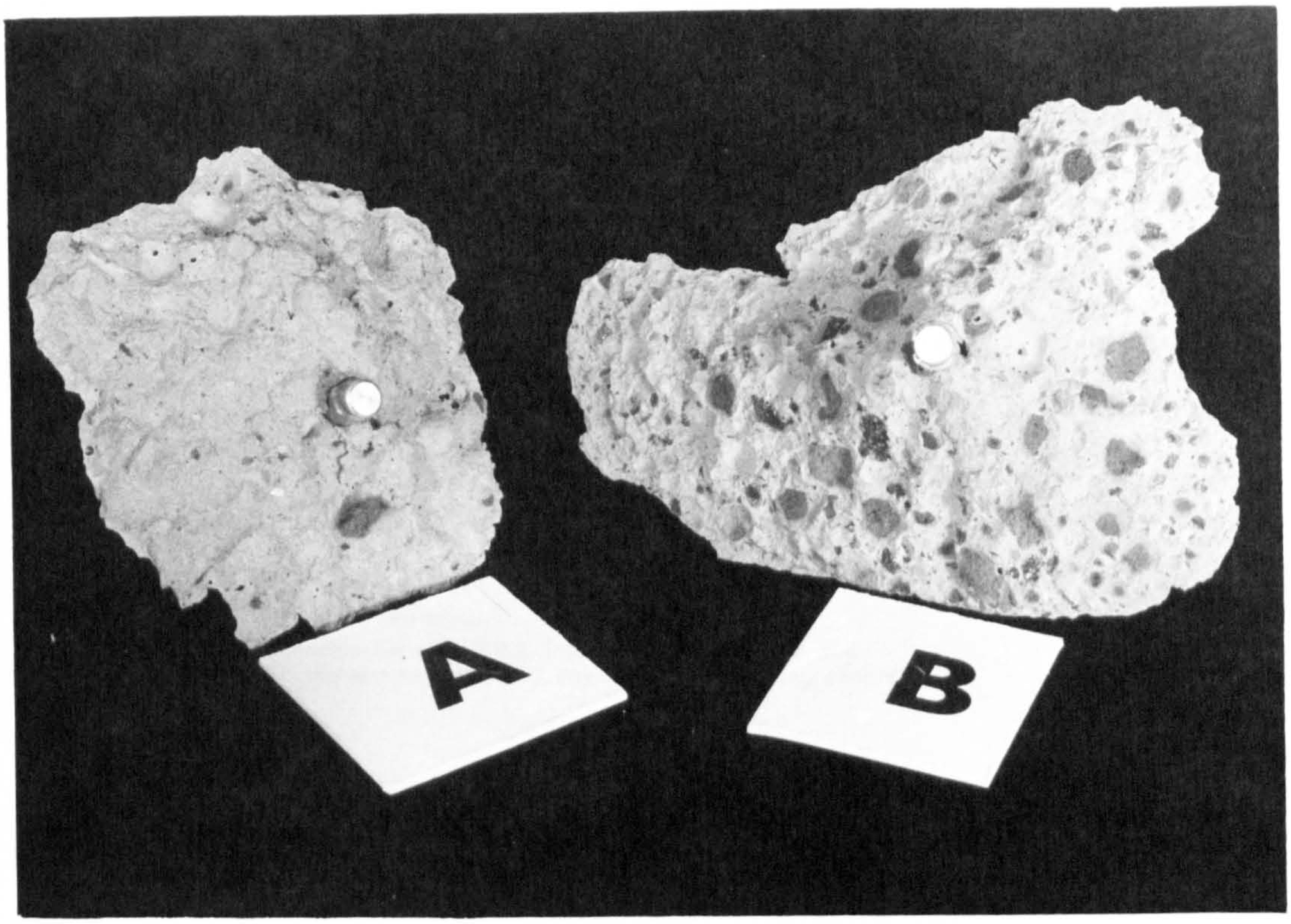
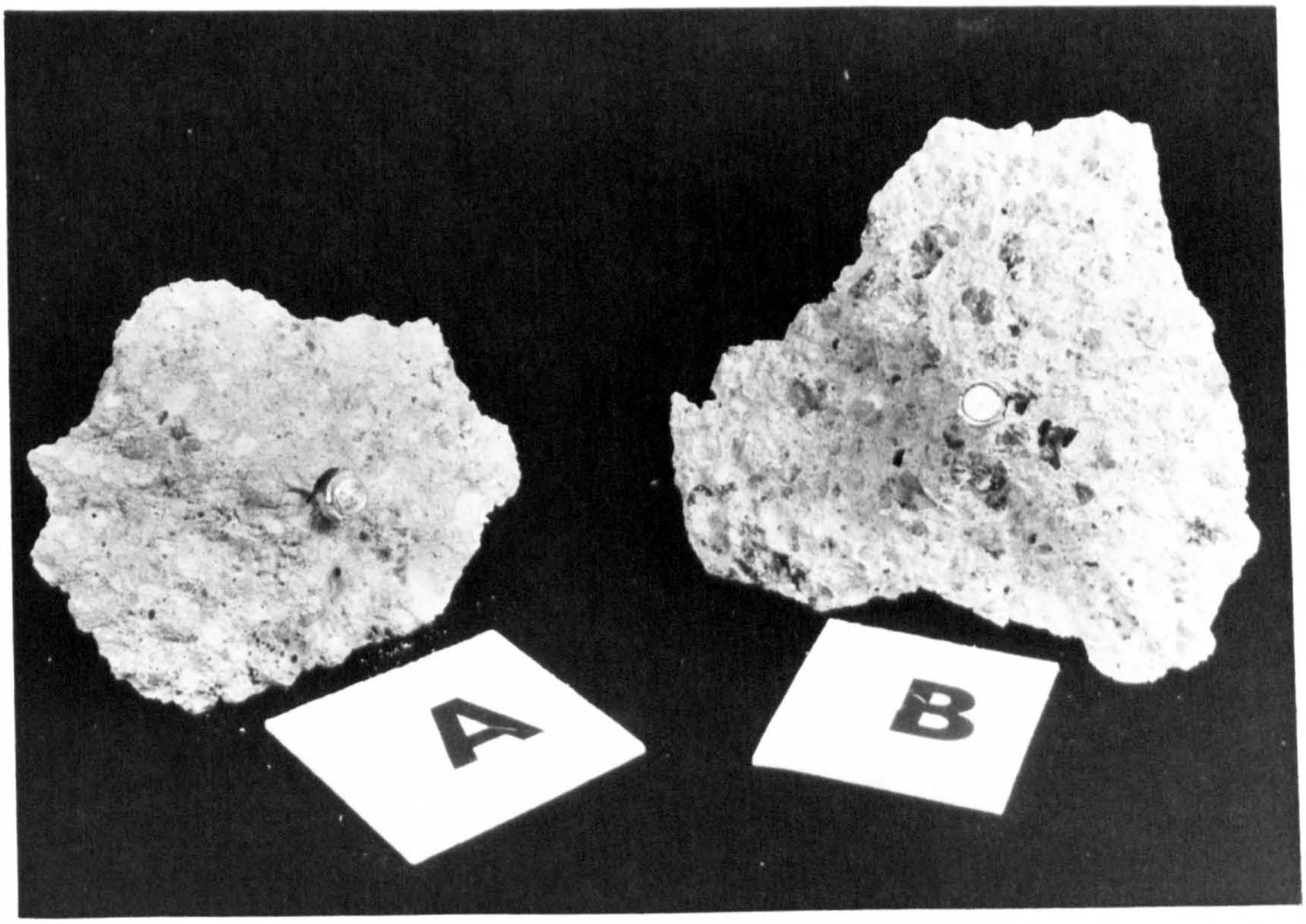


Figure 6.51:Percentage of compressive strength reduction on damaged cube by internal fracture test



A : low strength B : high strength

Figure 6.52: Mode of failure of fully Lytag concrete cones caused by the internal fracture test



A : low strength B : high strength

Figure 6.53: Mode of failure of Pellite concrete cones caused by the internal fracture test

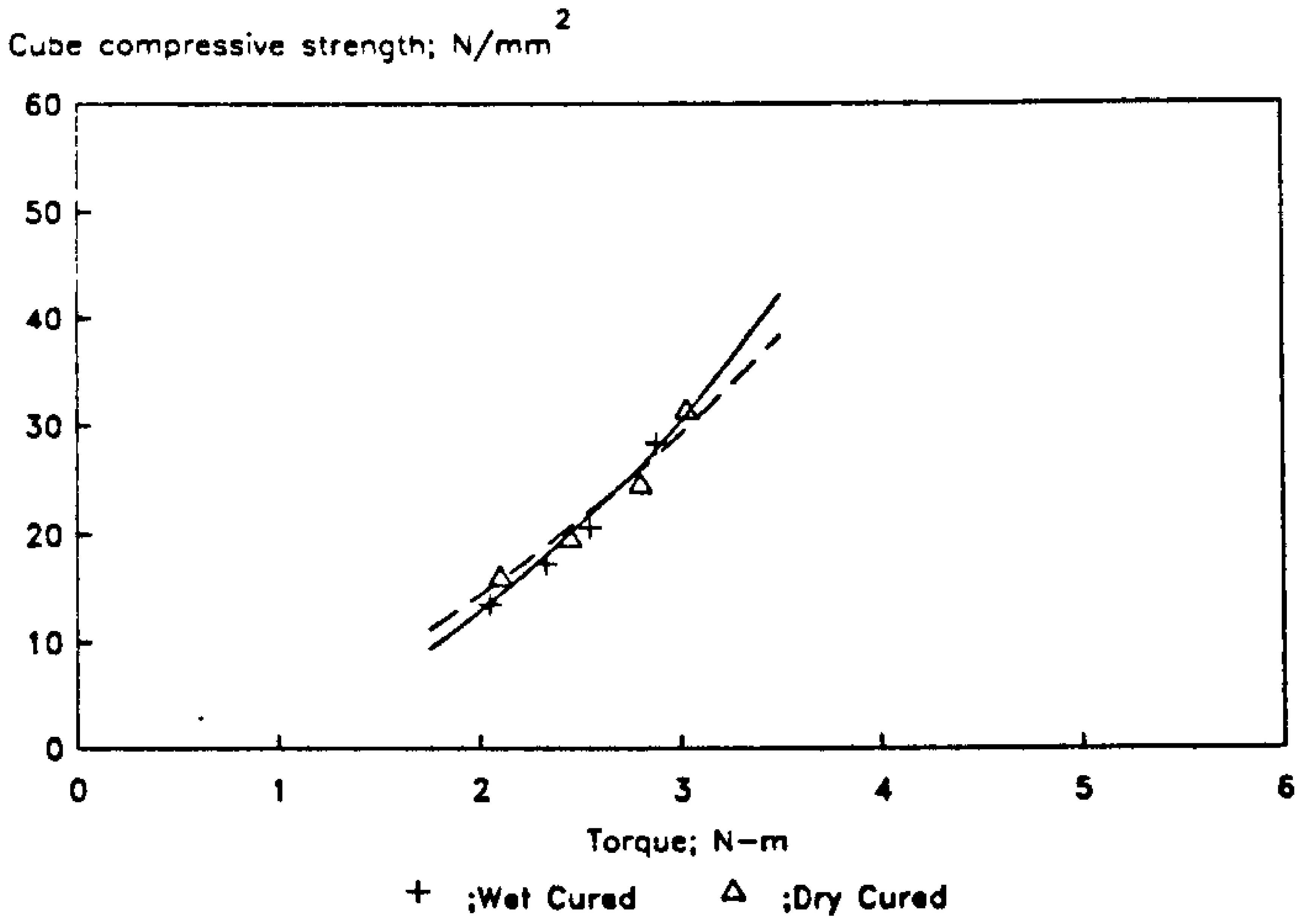


Figure 6.54.Relationship between B.R.E. internal fracture torque and compressive streng th for 7 day tests of fully Lytag concrete

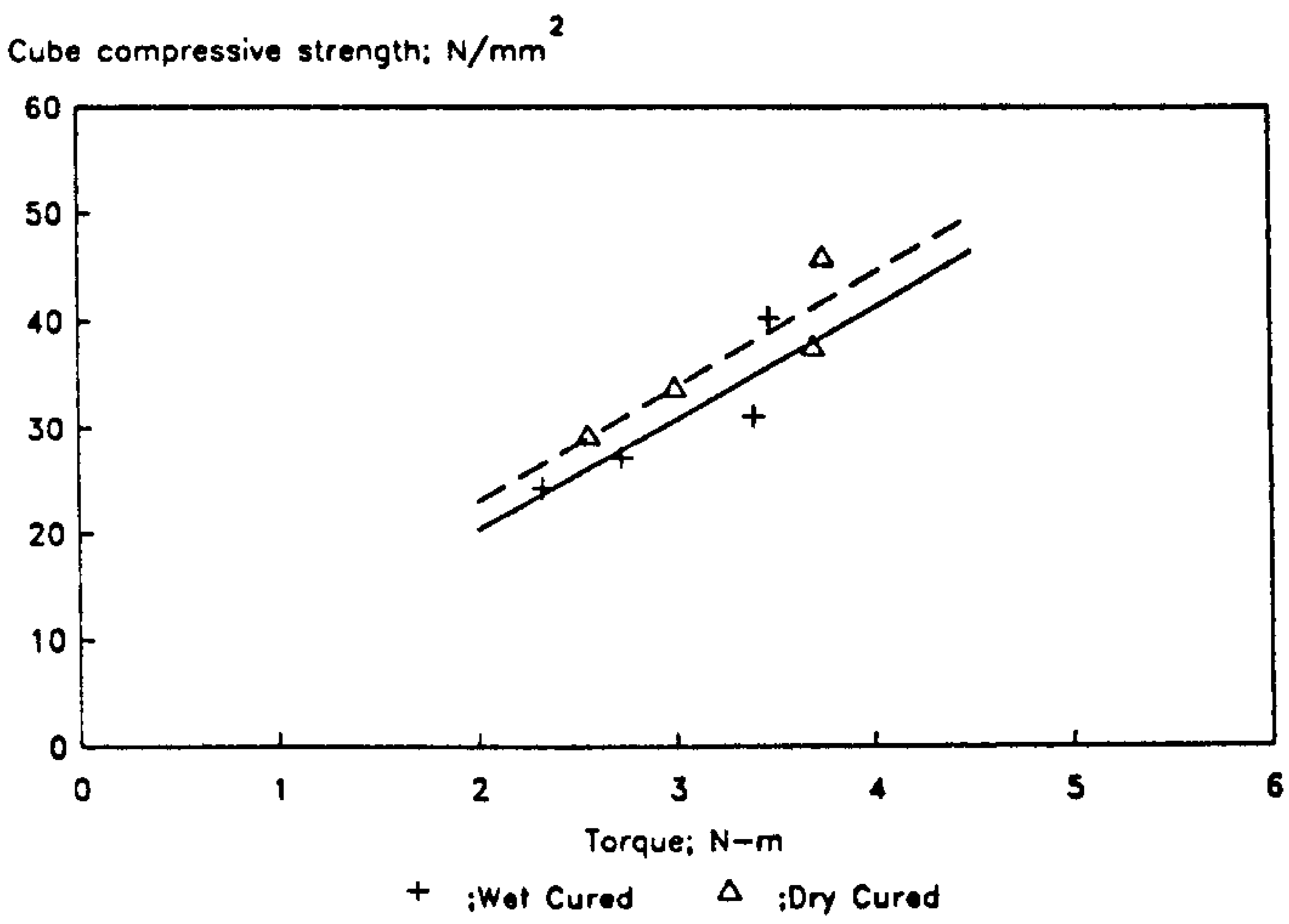


Figure 6.55:Relationship between B.R.E. internal fracture torque and compressive streng th for 28 day tests of fully Lytag concrete

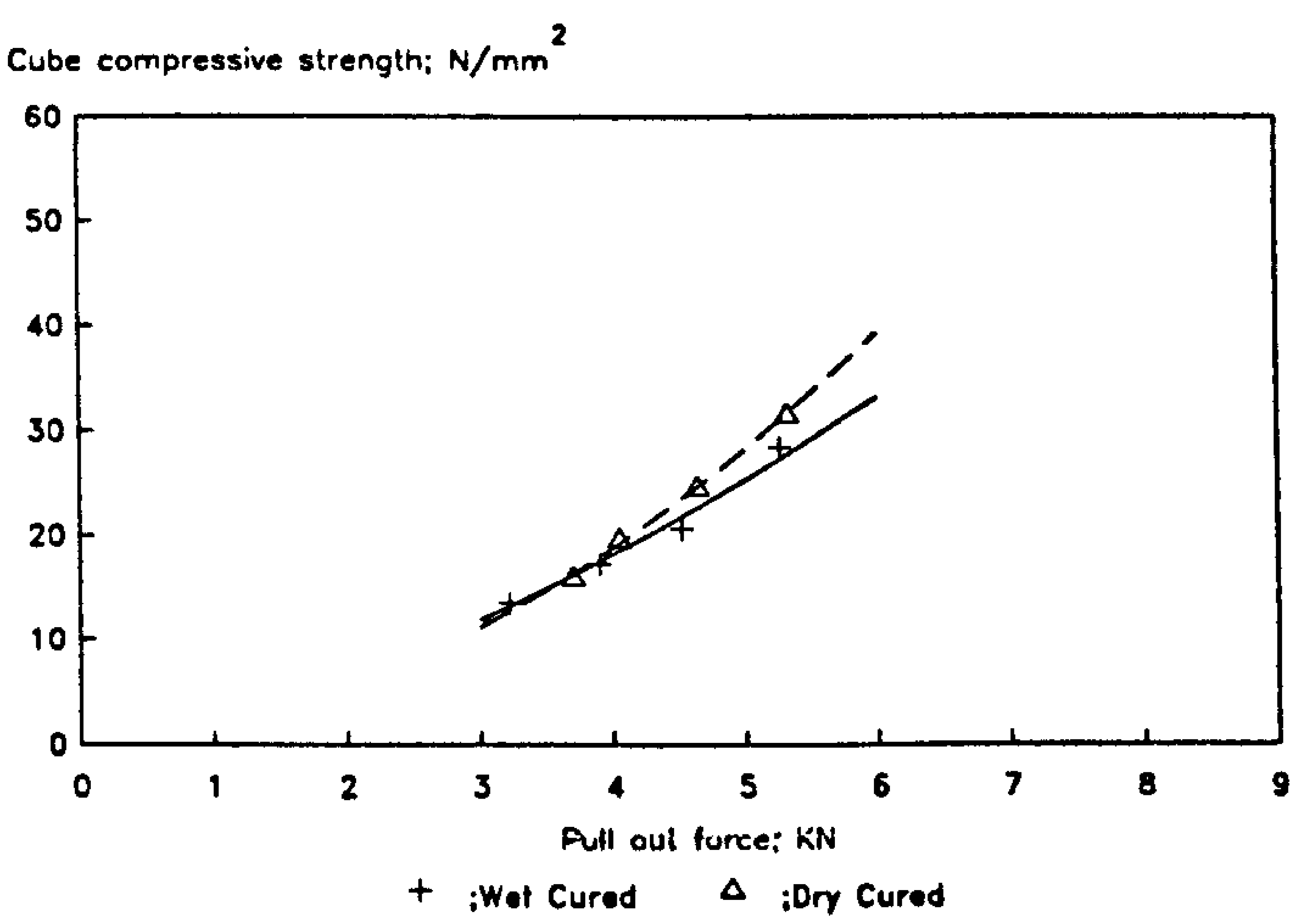


Figure 6.56.Relationship between direct pull internal fracture force and compressive strength for 7 day tests of fully Lytag concrete

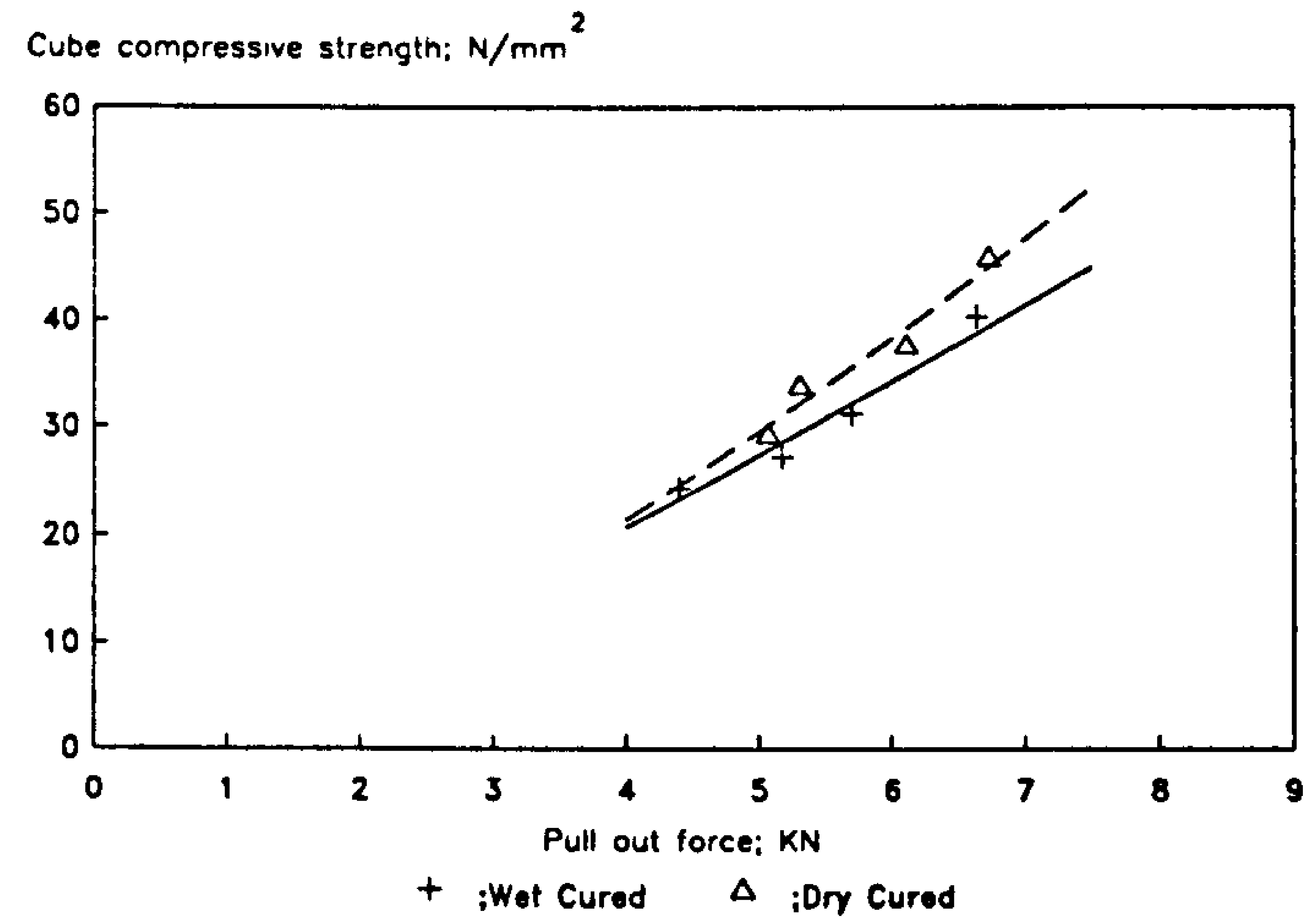


Figure 6.57:Relationship between direct pull internal fracture force and compressive strength for 28 day tests of fully Lytag concrete

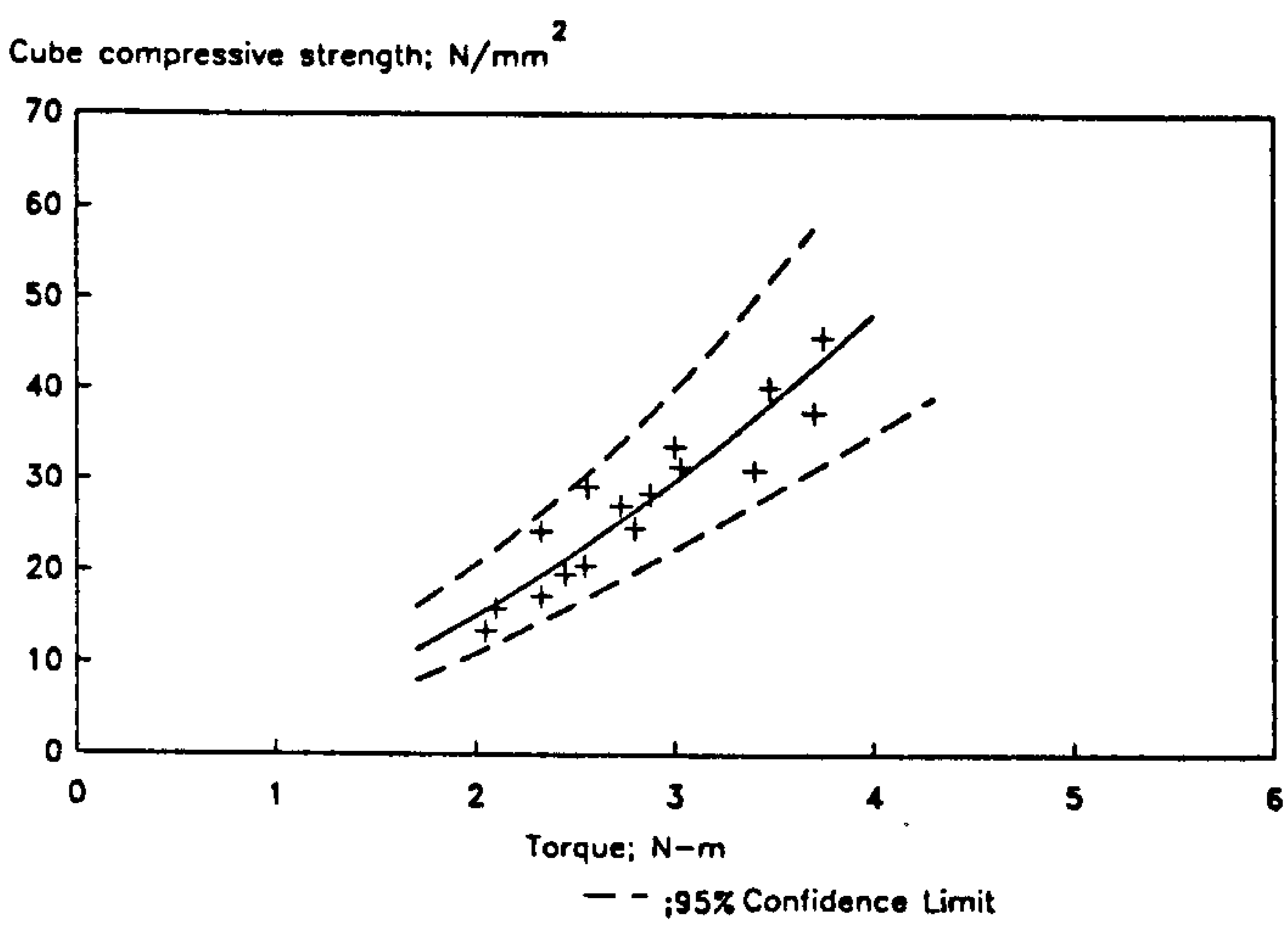


Figure 6.58:Relationship between B.R.E. internal fracture torque and compressive strength for fully Lytag concrete

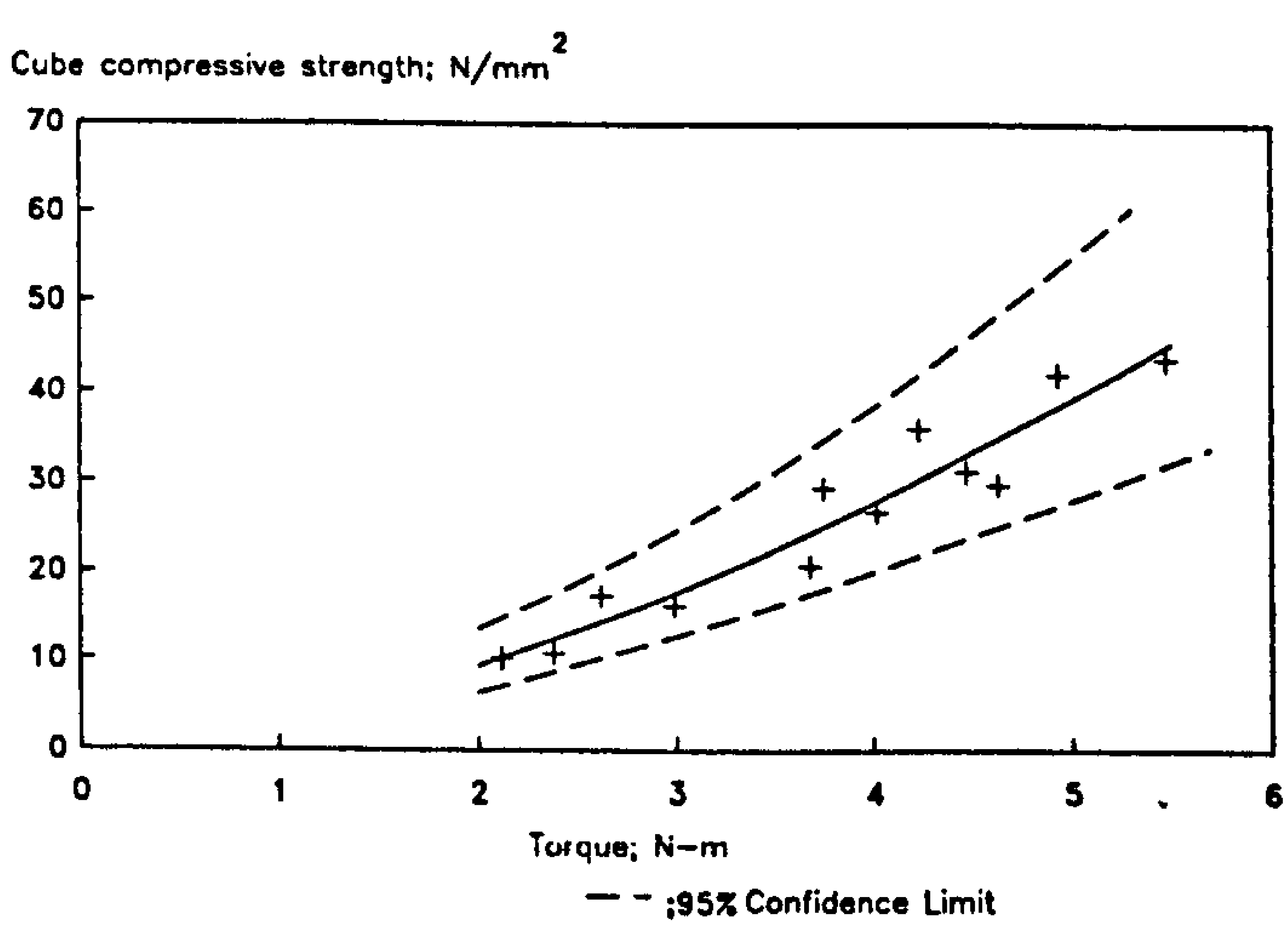


Figure 6.59:Relationship between B.R.E. internal fracture torque and compressive strength for semi Lytag concrete

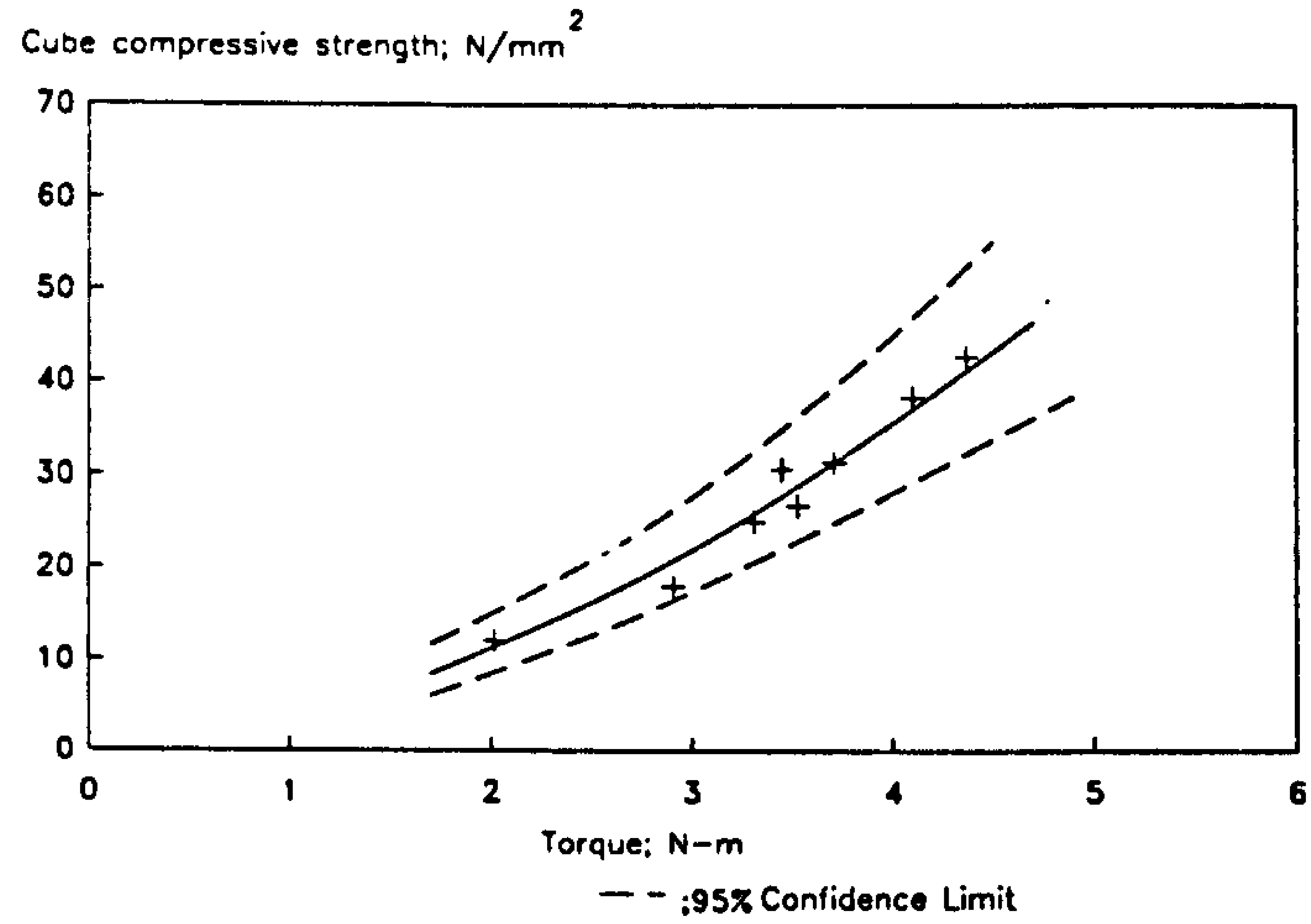


Figure 6.60.Relationship between B.R.E. internal fracture torque and compressive strength for Pellite concrete

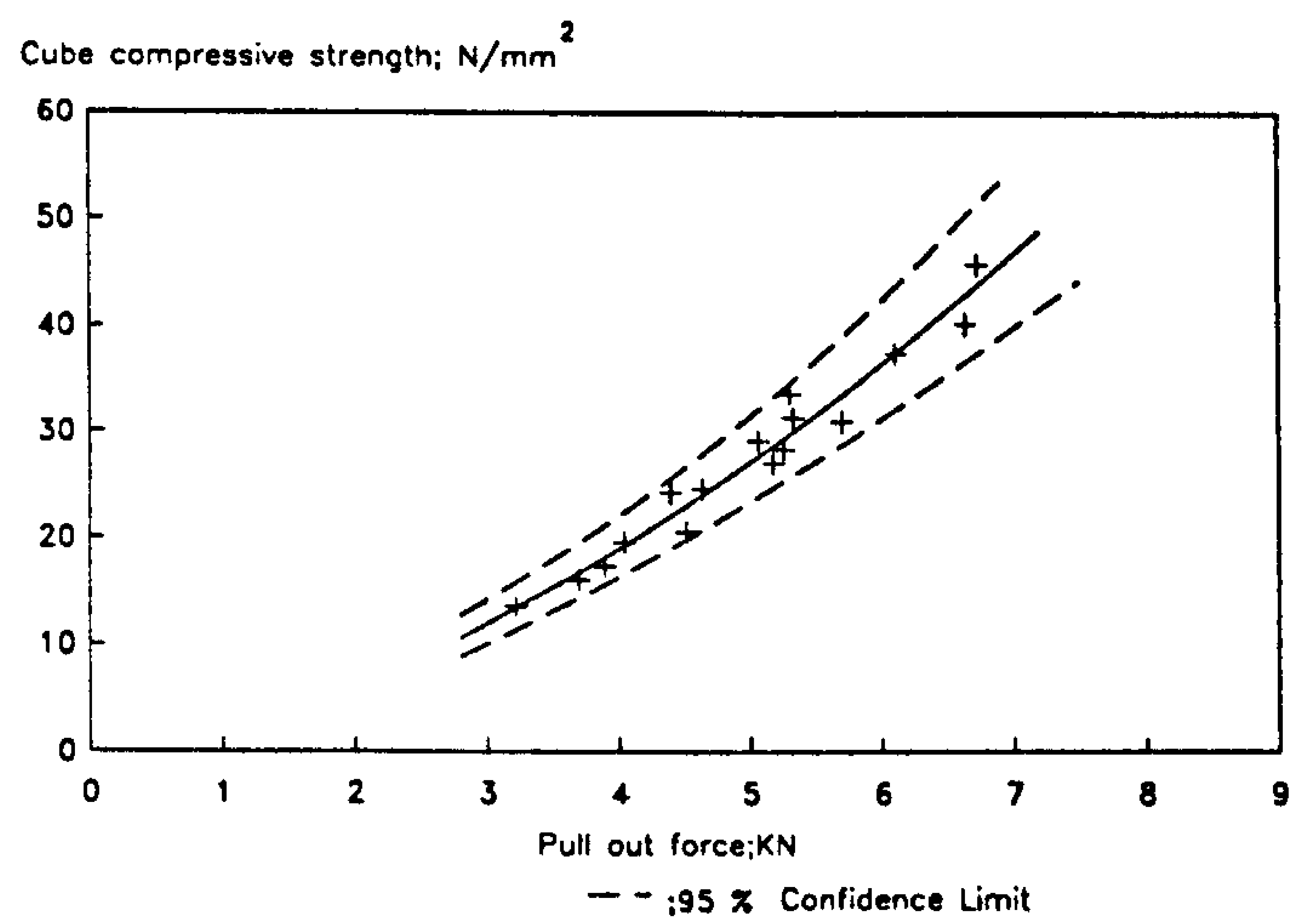


Figure 6.61:Relationship between direct pull internal fracture force and compressive strength for fully Lytag concrete

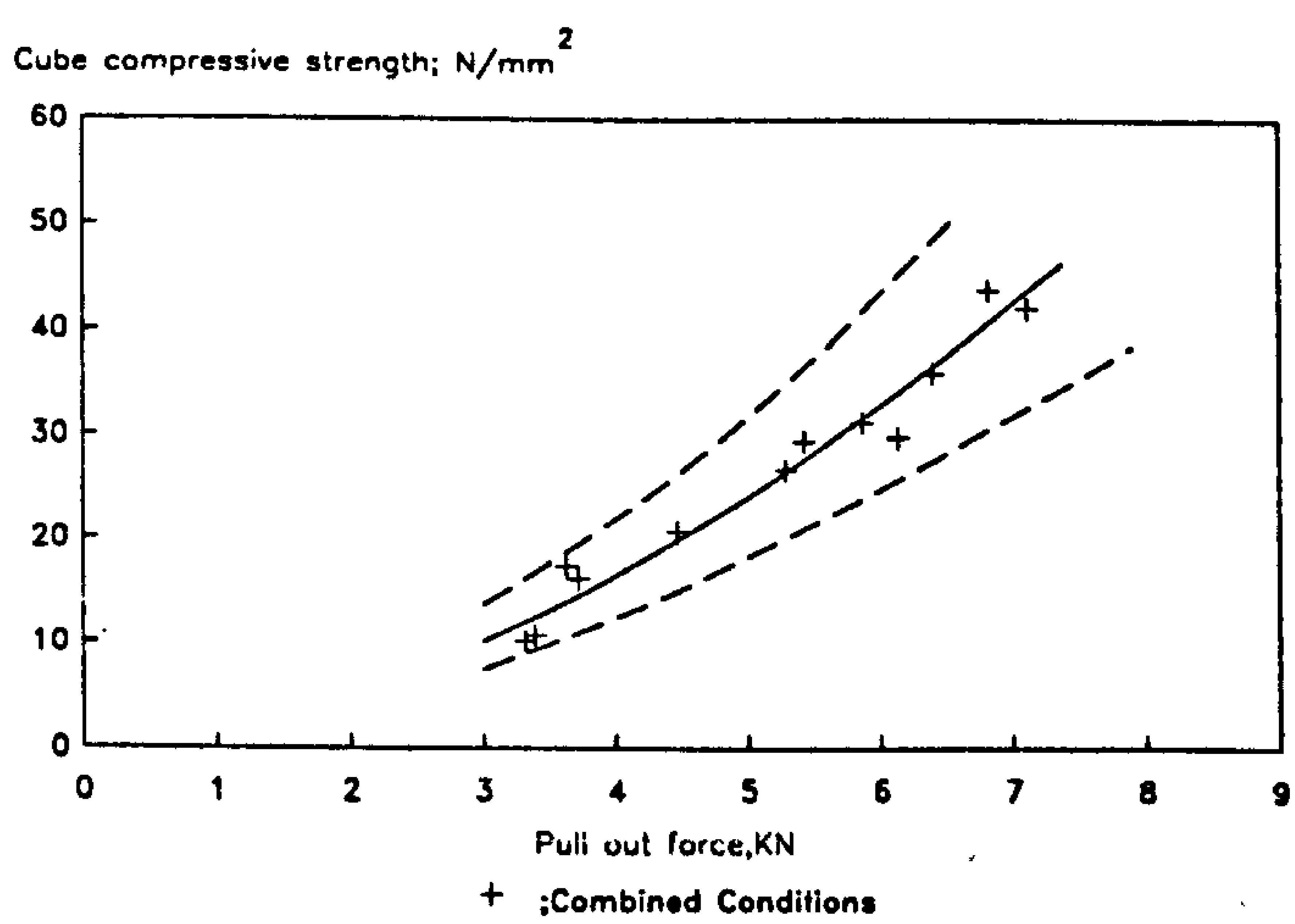


Figure 6.62:Relationship between direct pull internal fracture force and compressive strength for semi Lytag concrete

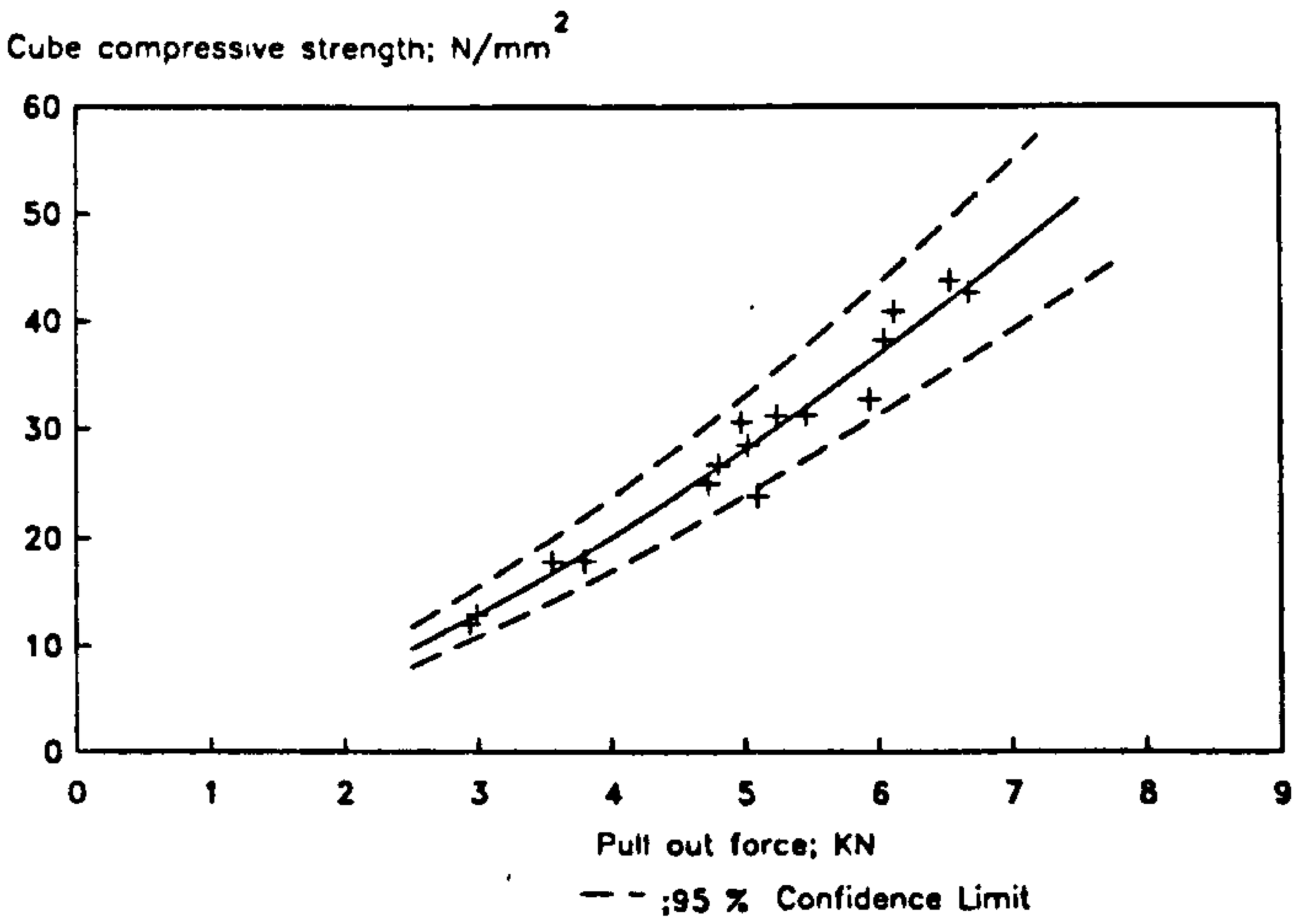


Figure 6.63:Relationship between direct pull internal fracture force and compressive strength for Pellite concrete

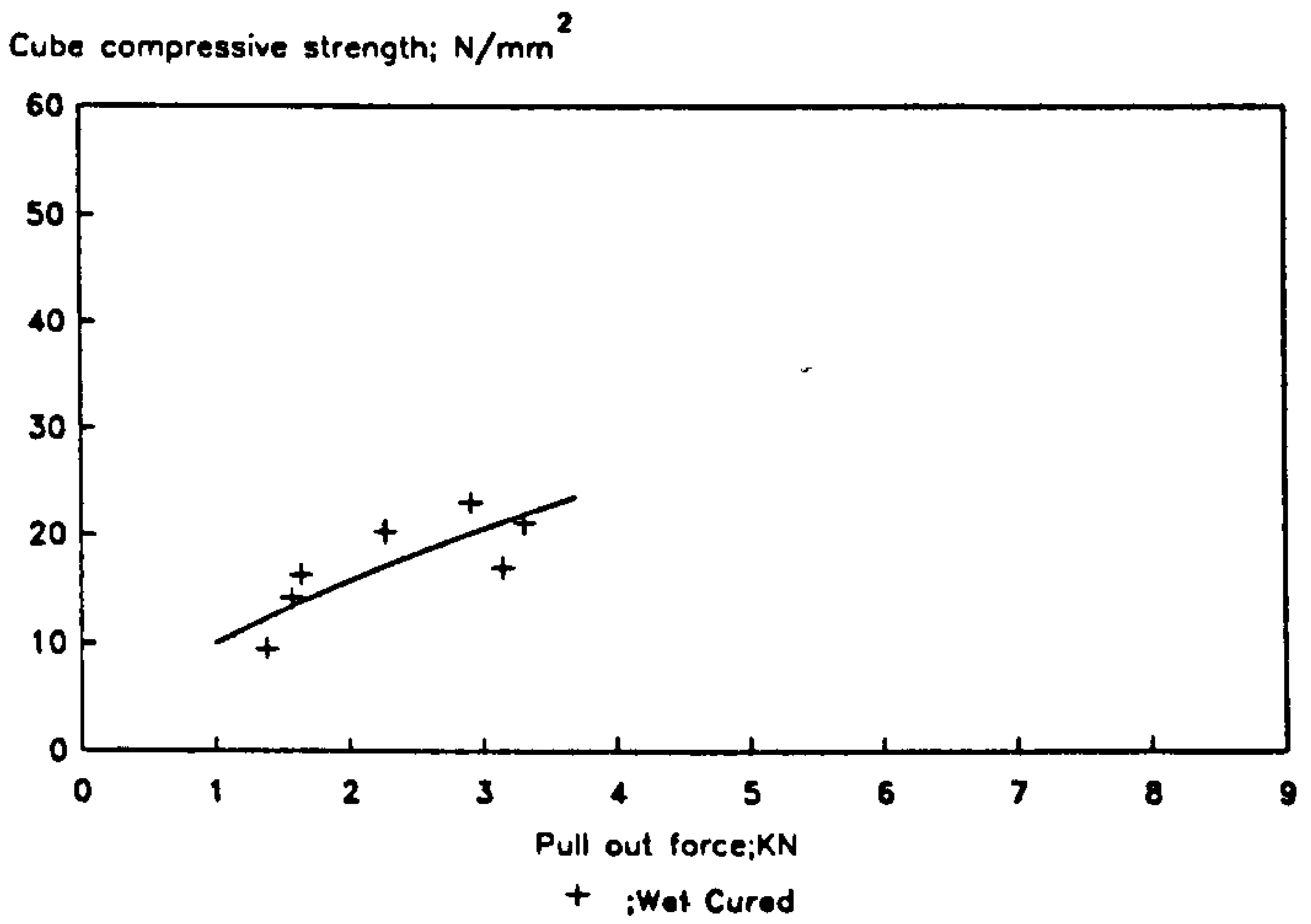


Figure 6.64:Relationship between direct pull internal fracture force and compressive strength for Leca concrete

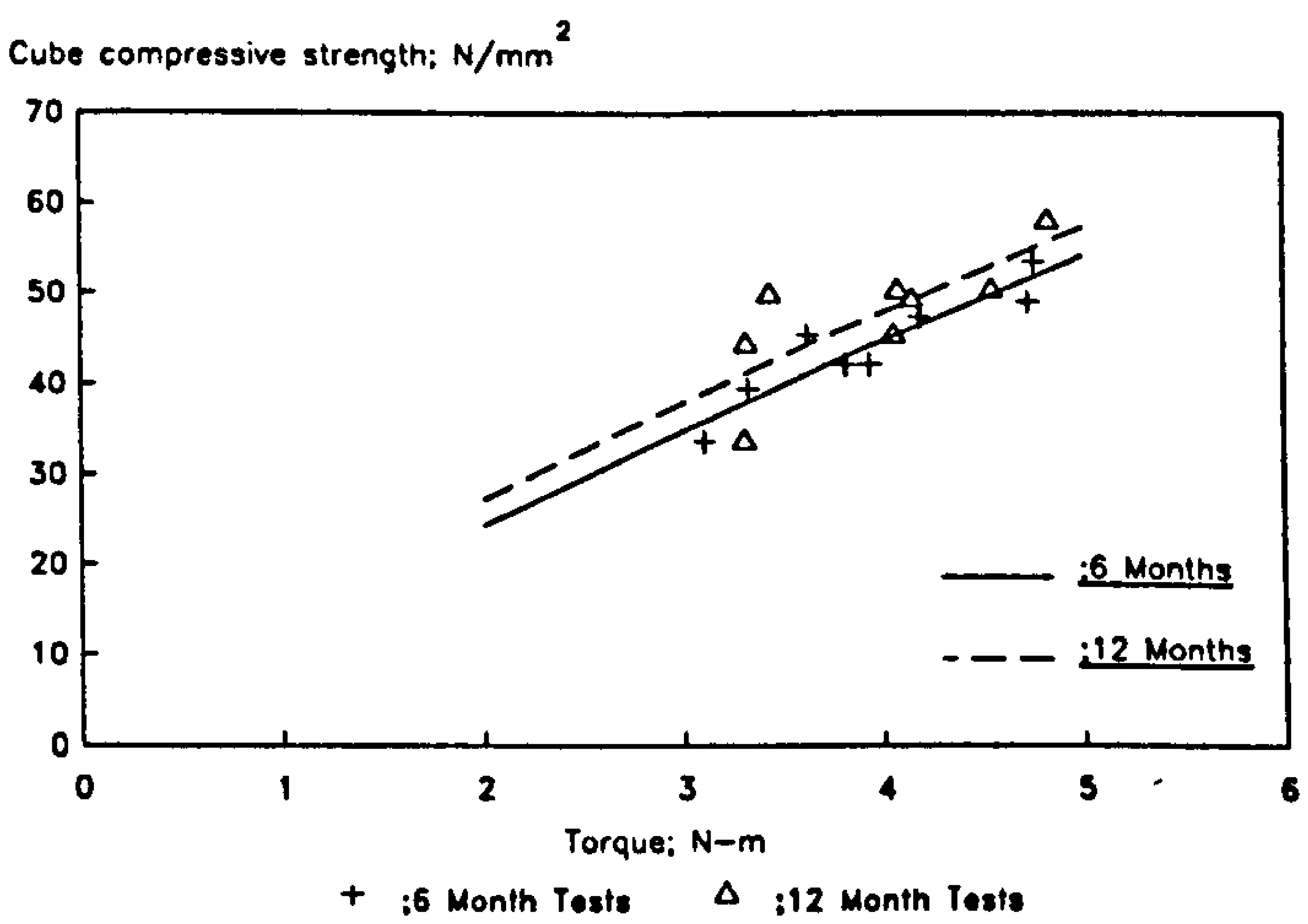


Figure 6.65:B.R.E. internal fracture torque versus compressive strength for long term tests of fully Lytag concrete

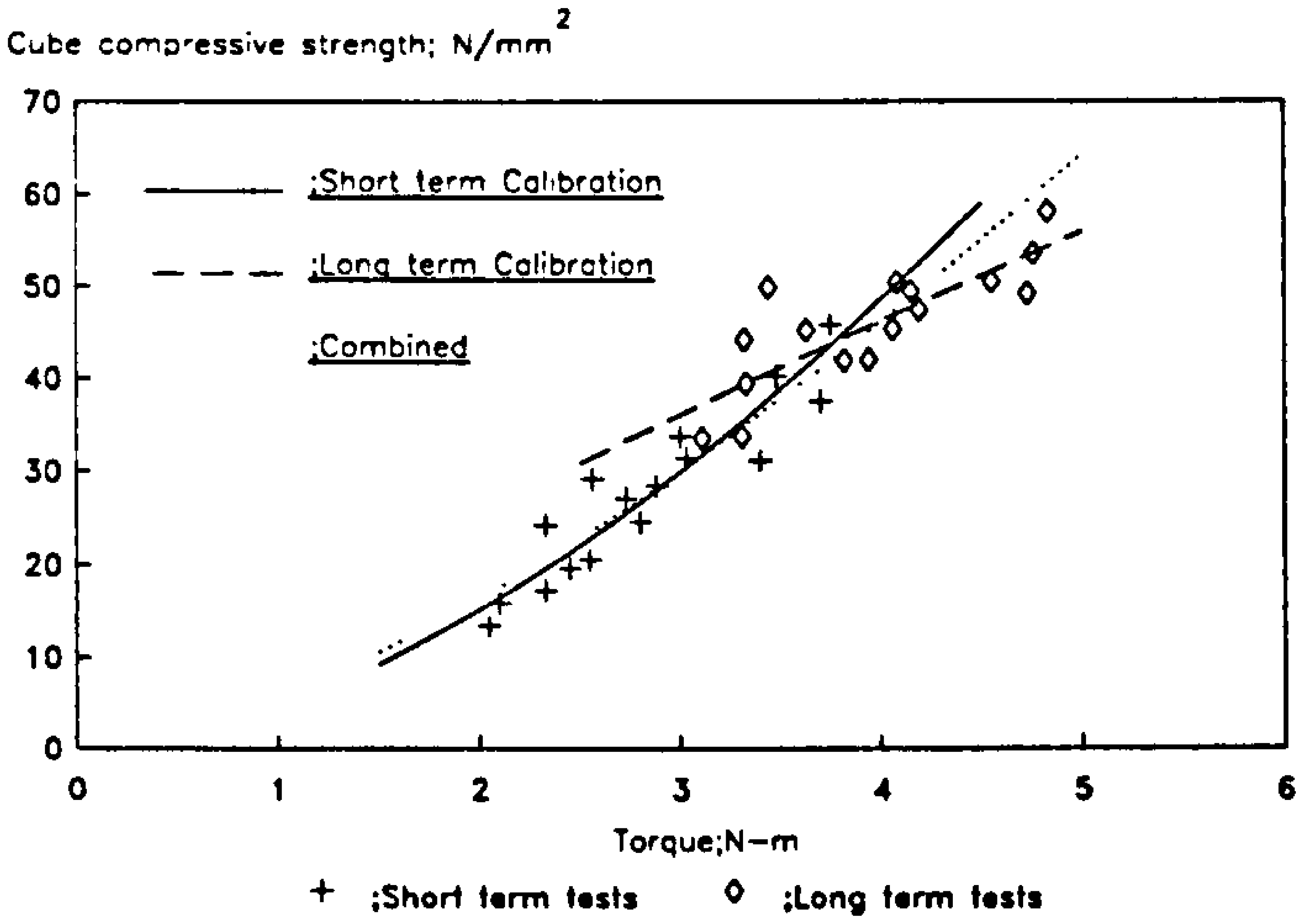


Figure 6.66.Comparison of B.R.E internal fracture torque calibrations for fully Lytag concrete at various ages

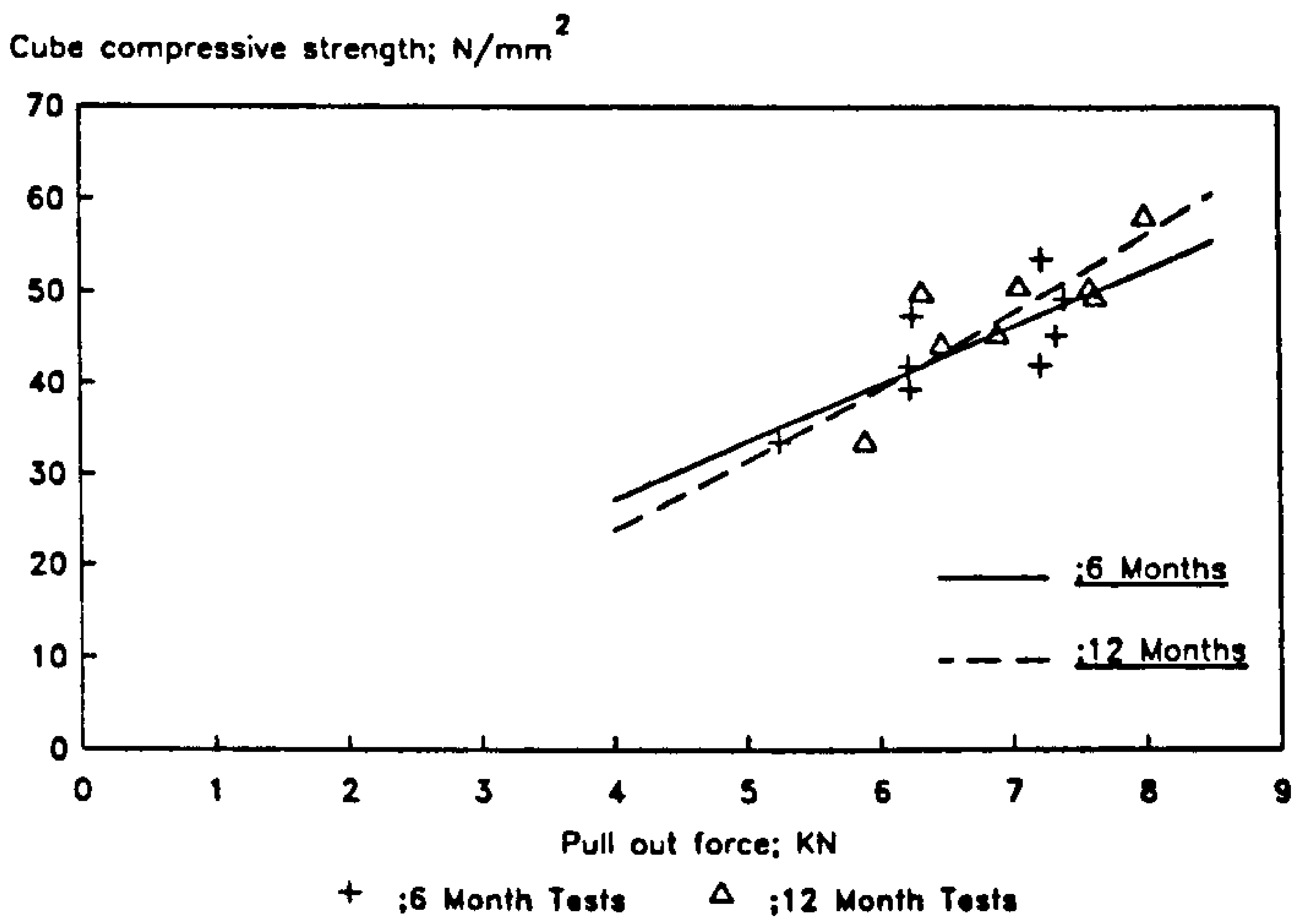


Figure 6.67:Direct pull internal frcture force versus compressive strength for long term tests of fully Lytag concrete

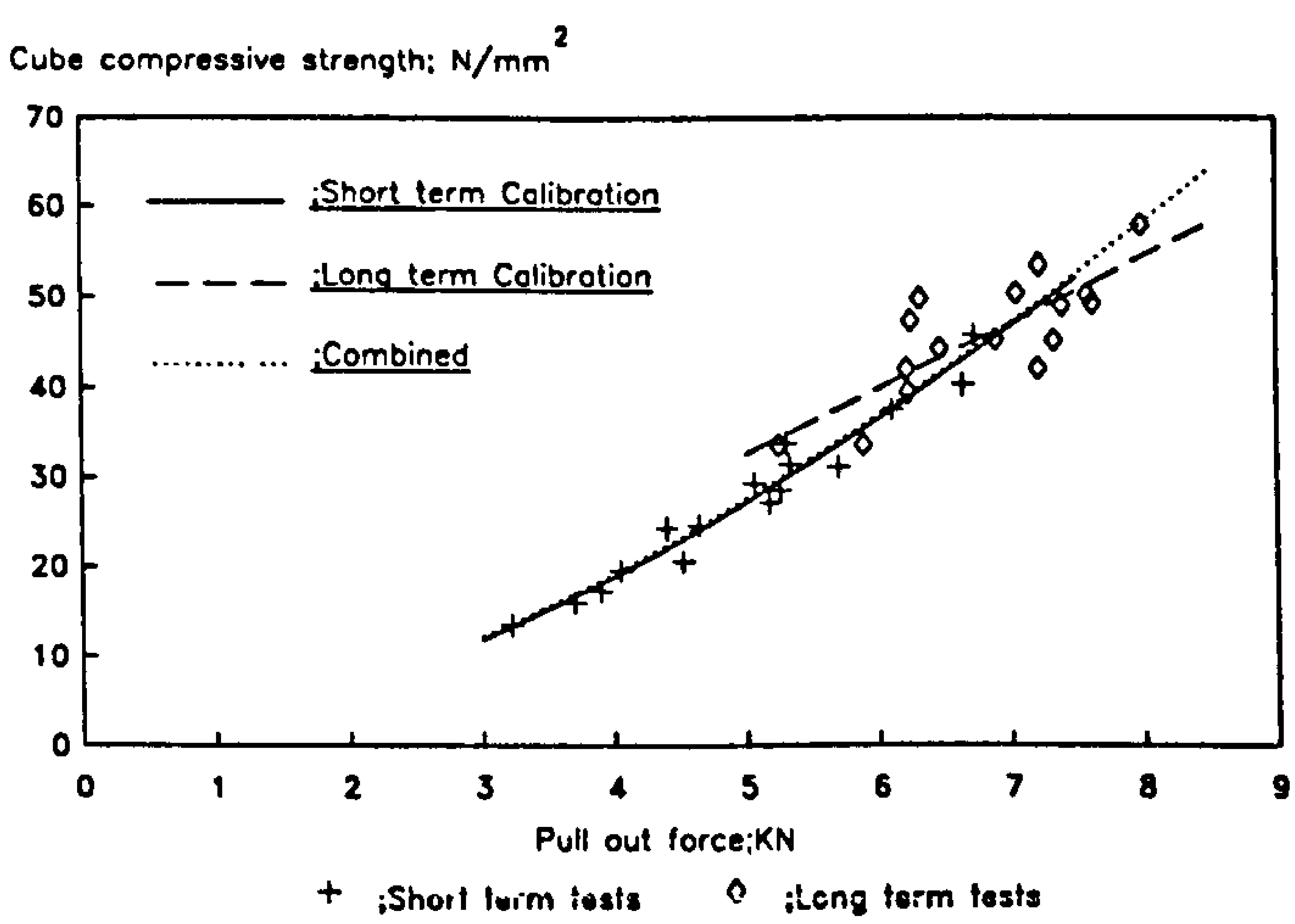


Figure 6.68:Comparison of direct pull internal fracture calibrations for fully Lytag concrete at various ages

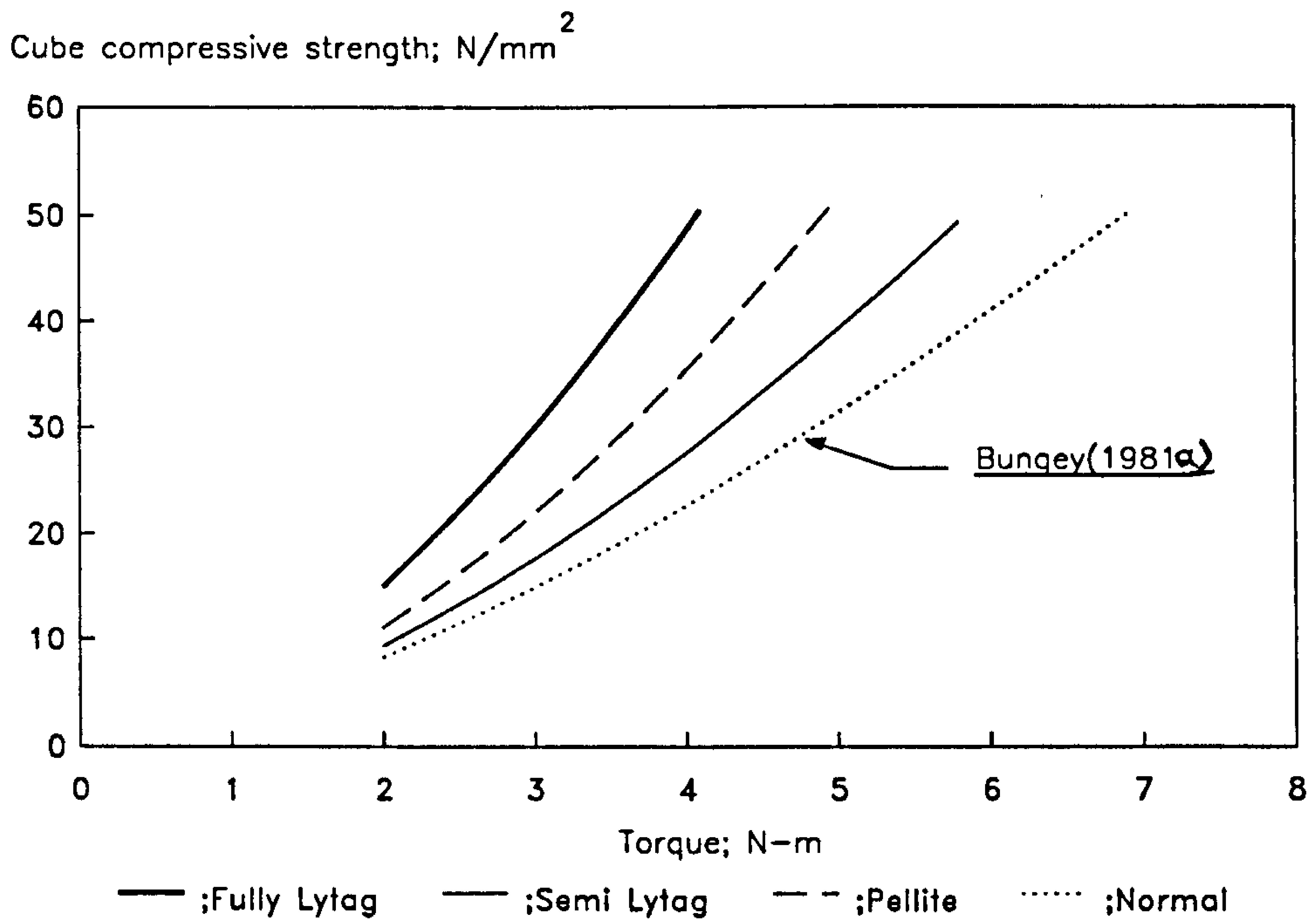


Figure 6.69:Comparison of B.R.E. internal fracture calibration for different types of concrete

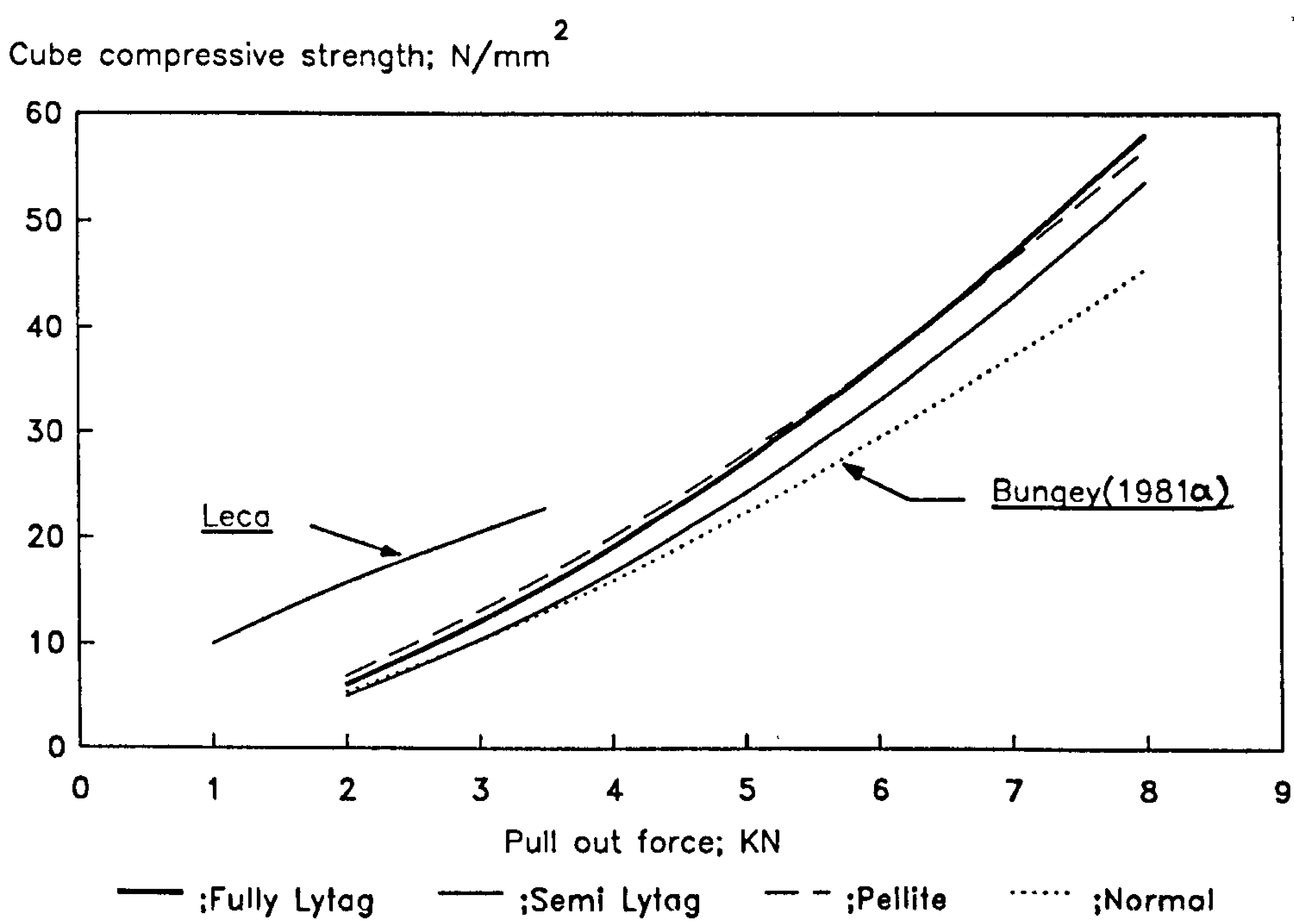


Figure 6.70:Comparison of direct pull internal fracture calibrations for different types of concrete

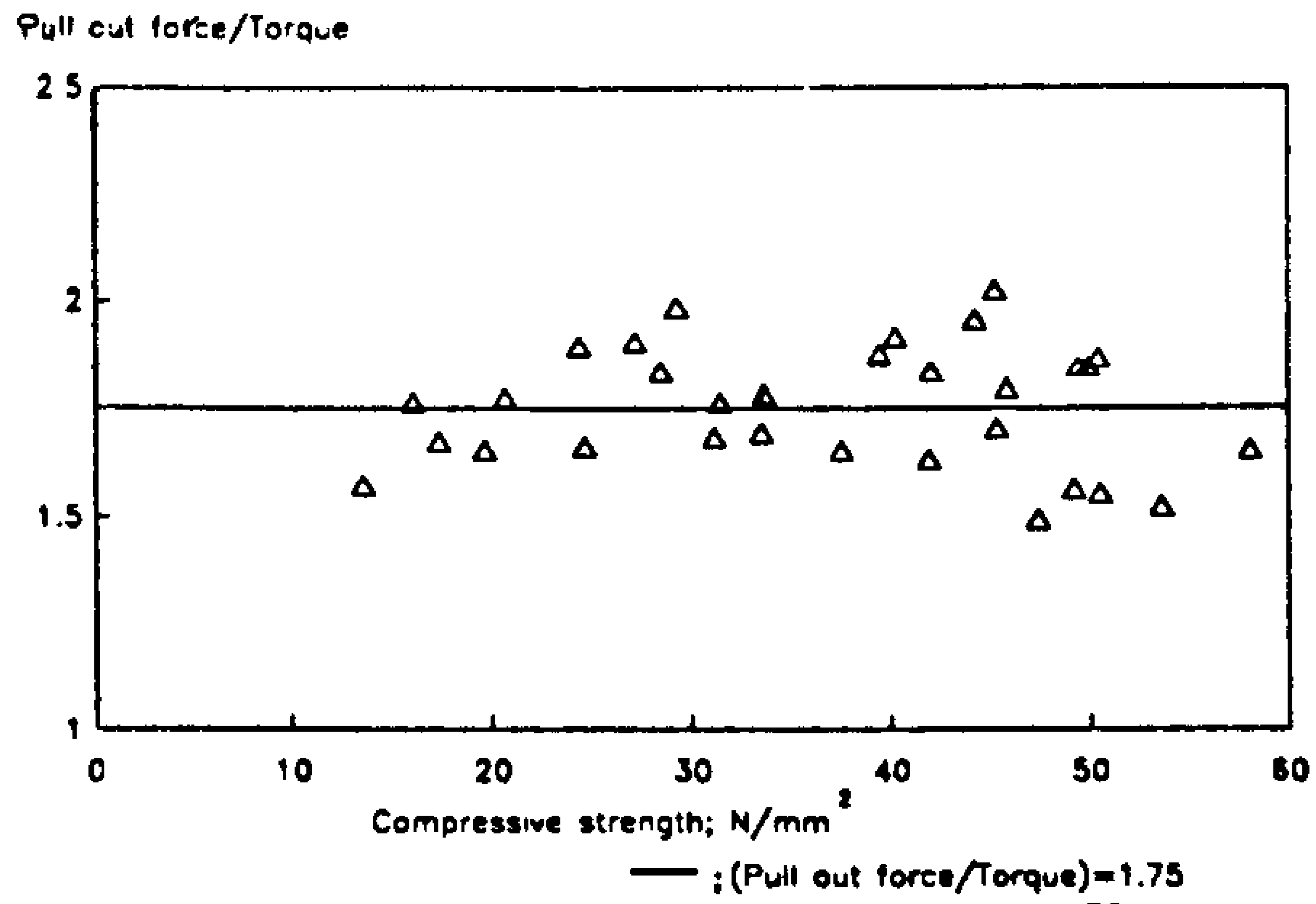


Figure 6.71:Pull out force/torque ratio versus compressive strength for fully Lytag concrete

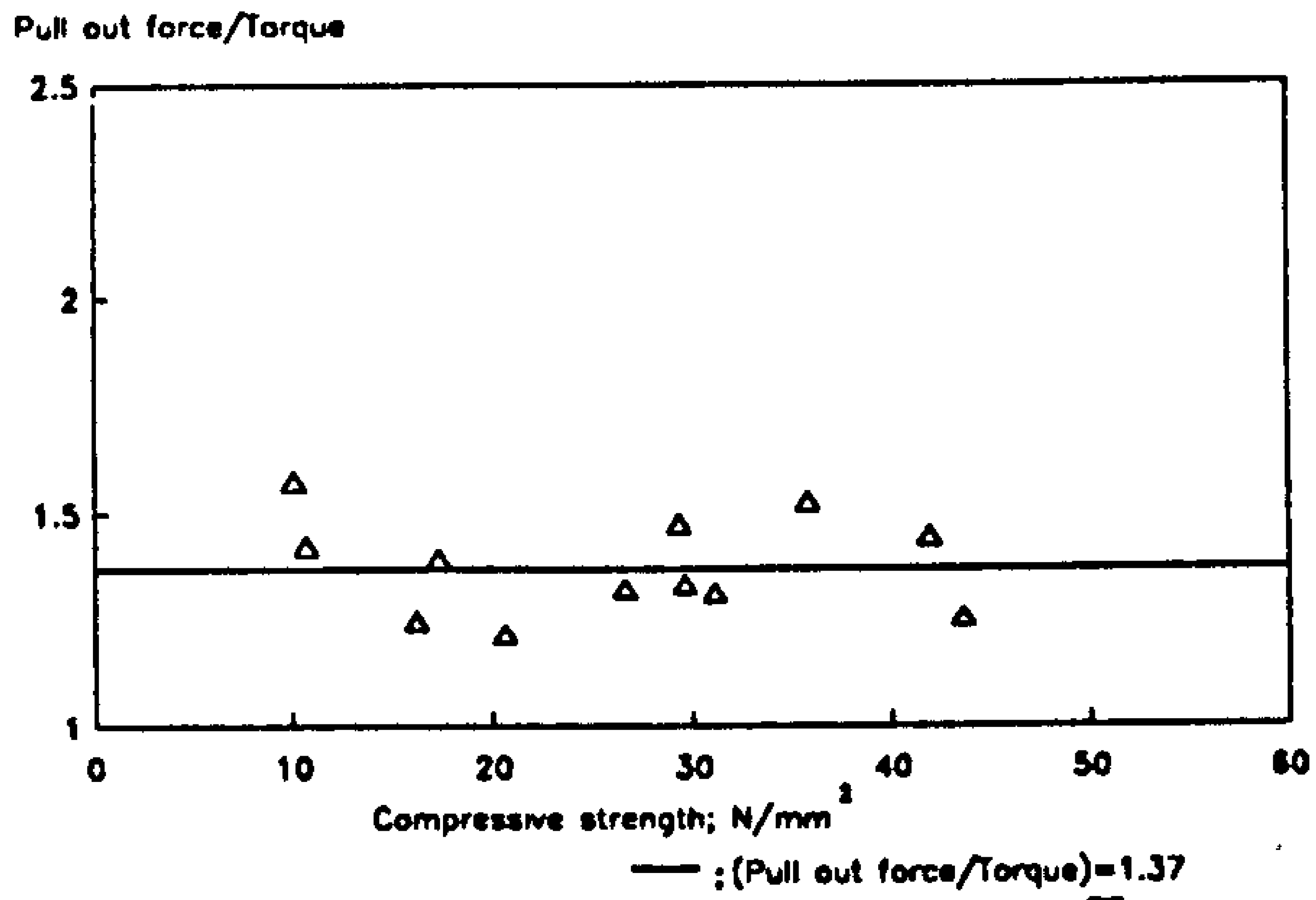


Figure 6.72:Pull out force/torque ratio versus compressive strength for semi Lytag concrete

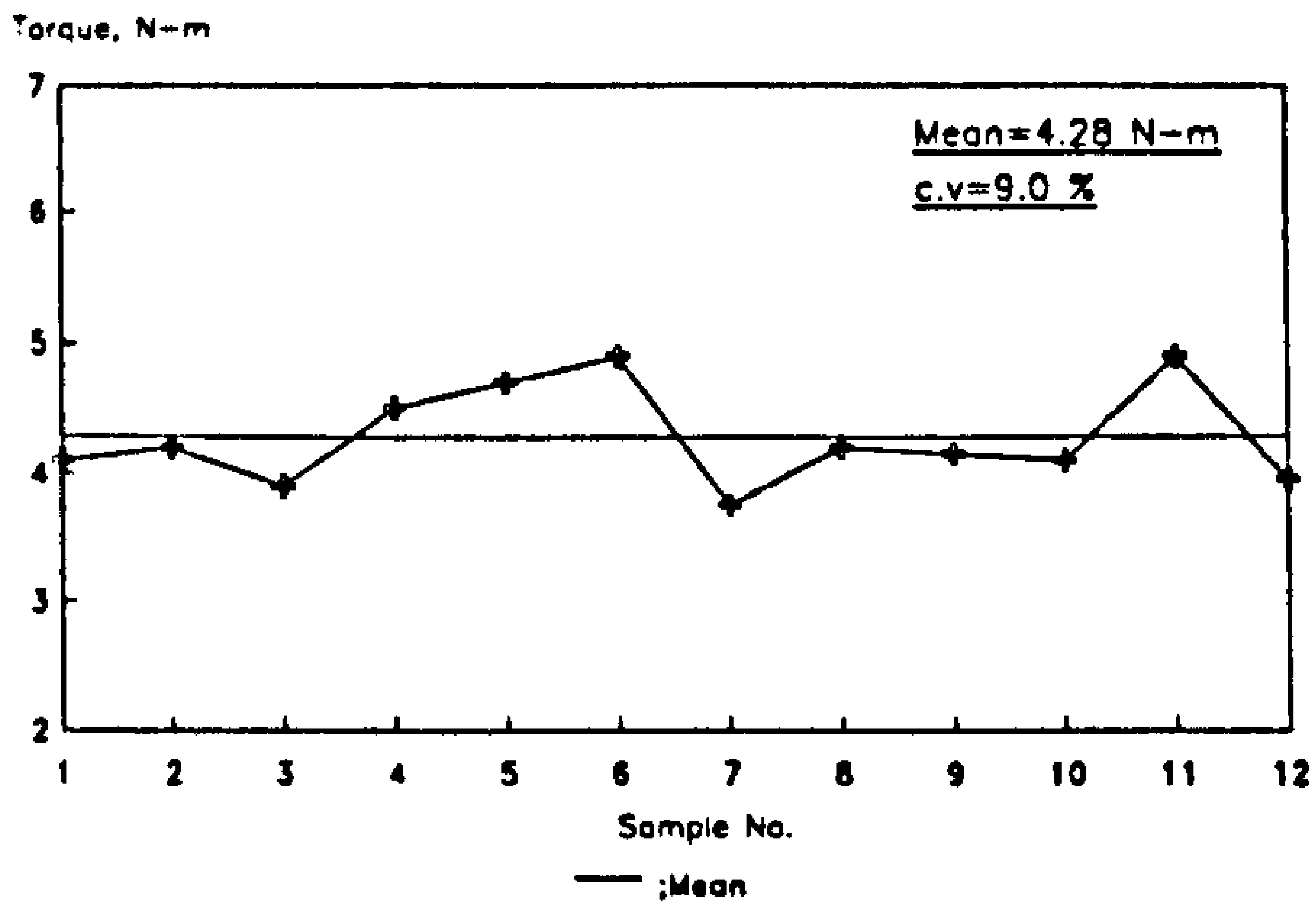


Figure 6.73:Variation of B.R.E. internal fracture torque on fully Lytag concrete

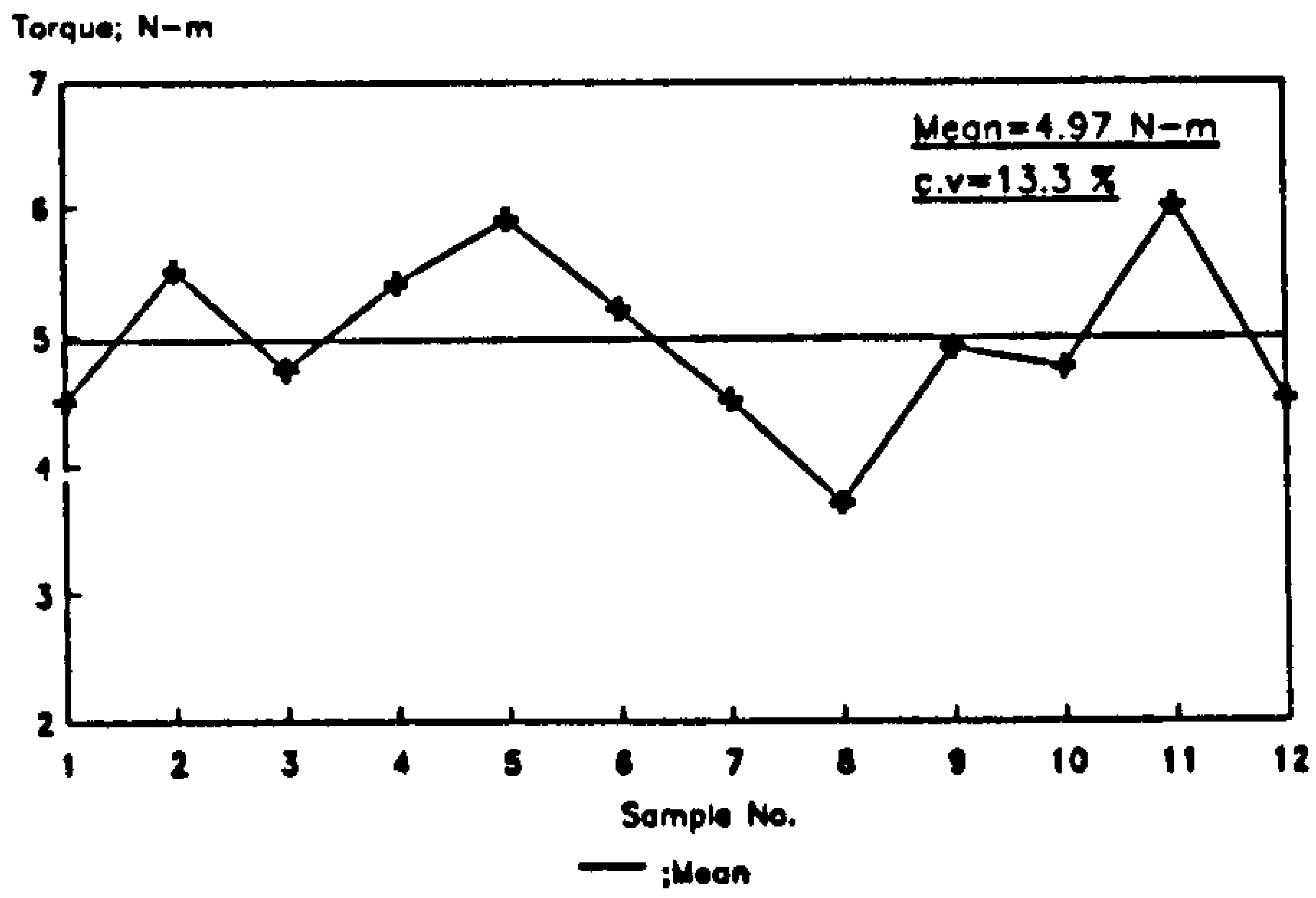


Figure 6.74:Variation of B.R.E. internal fracture torque on semi Lytag concrete

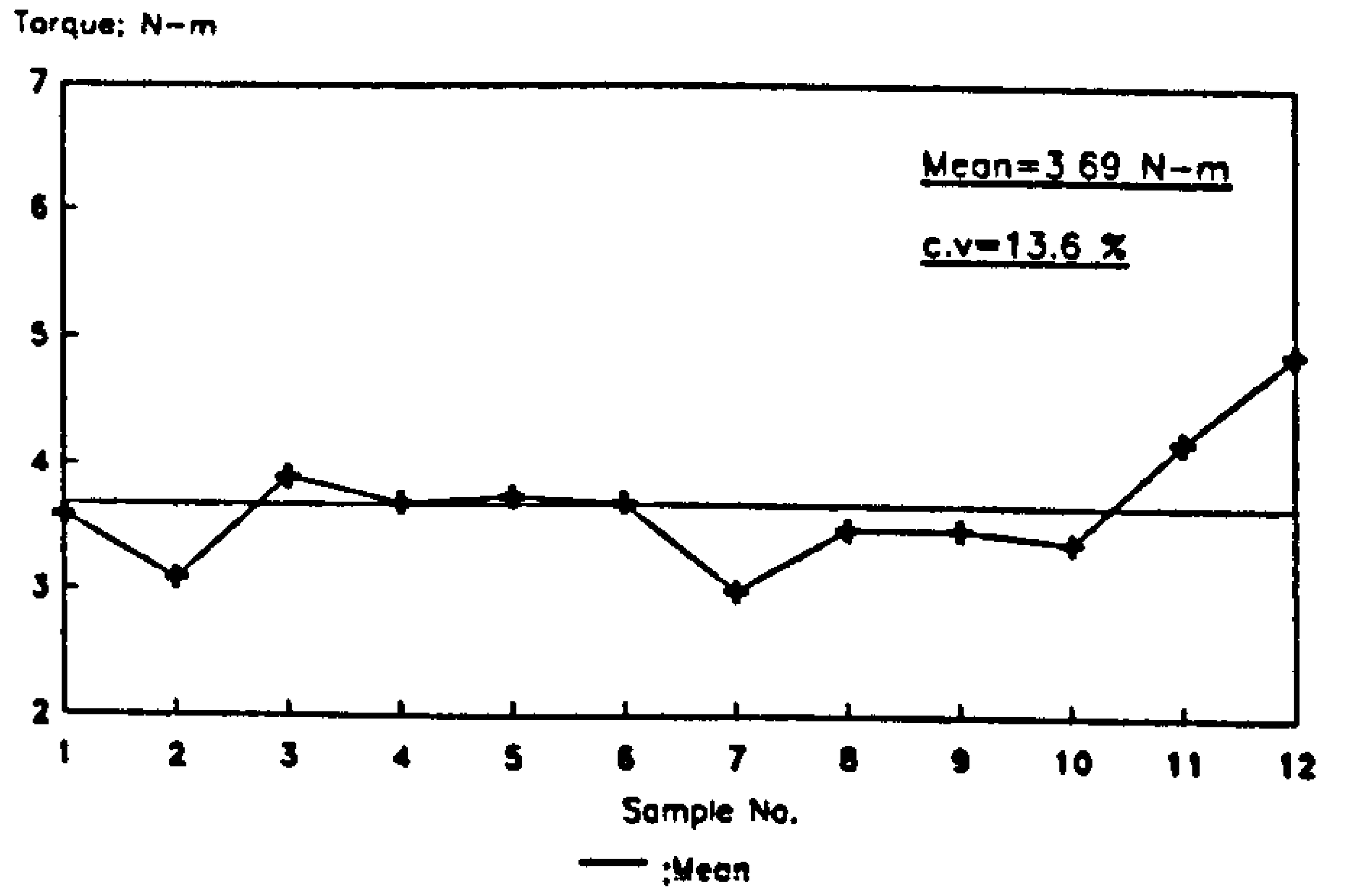


Figure 6.75.Variation of B.R.E. internal fracture torque on Pellite concrete

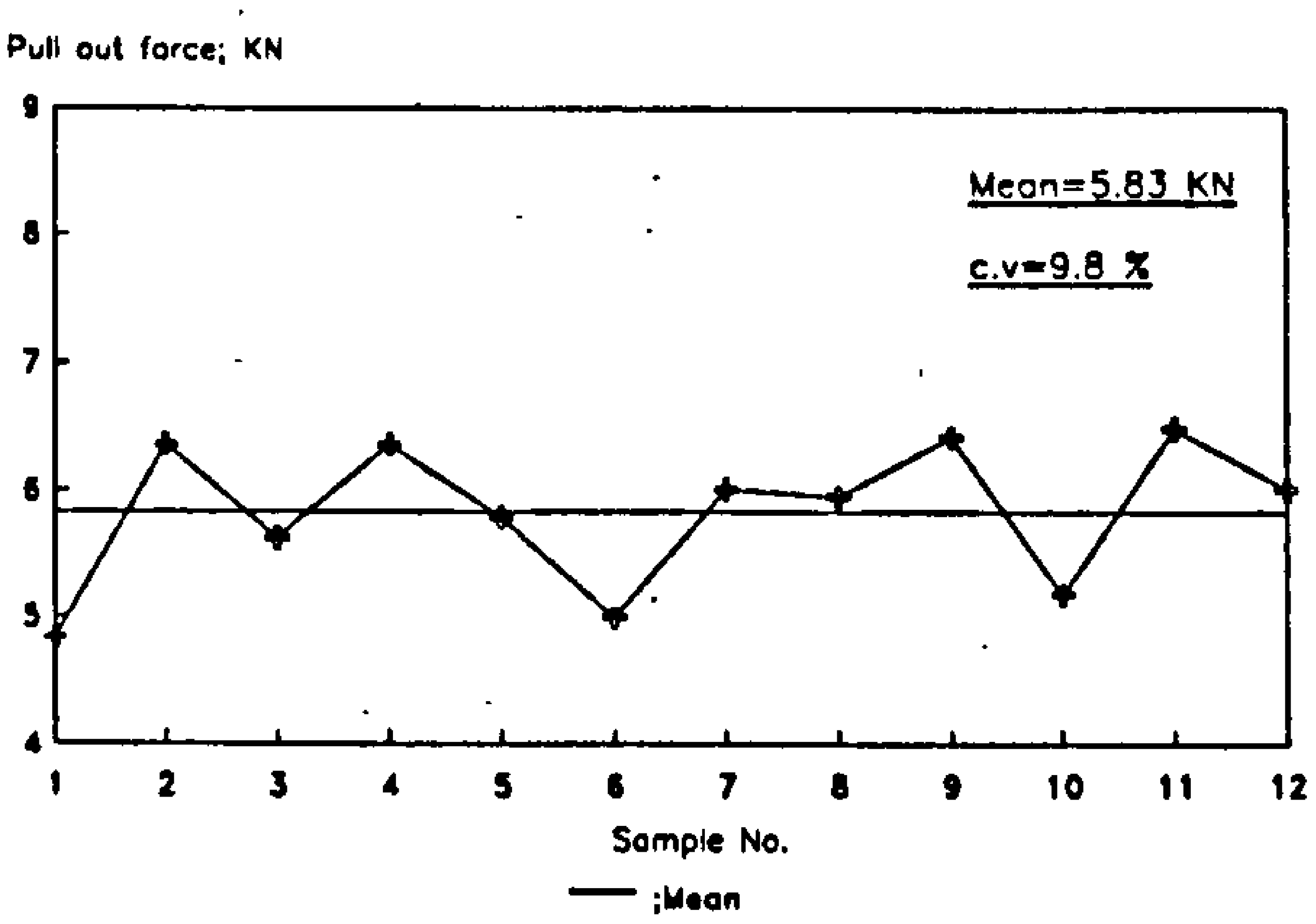


Figure 6.76:Variation of direct pull internal fracture force on fully Lytag concrete

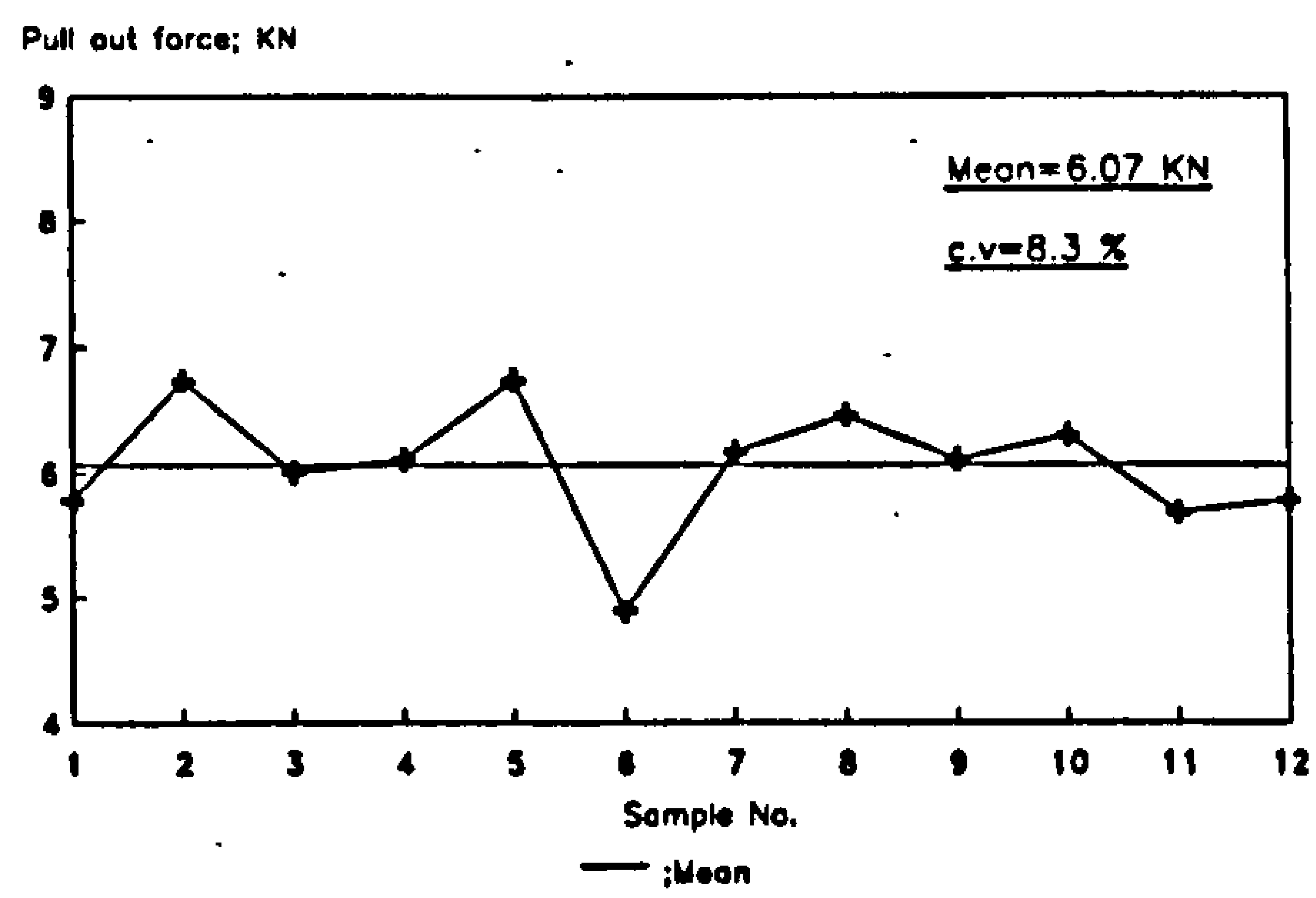


Figure 6.77:Variation of direct pull internal fracture force on semi Lytag concrete

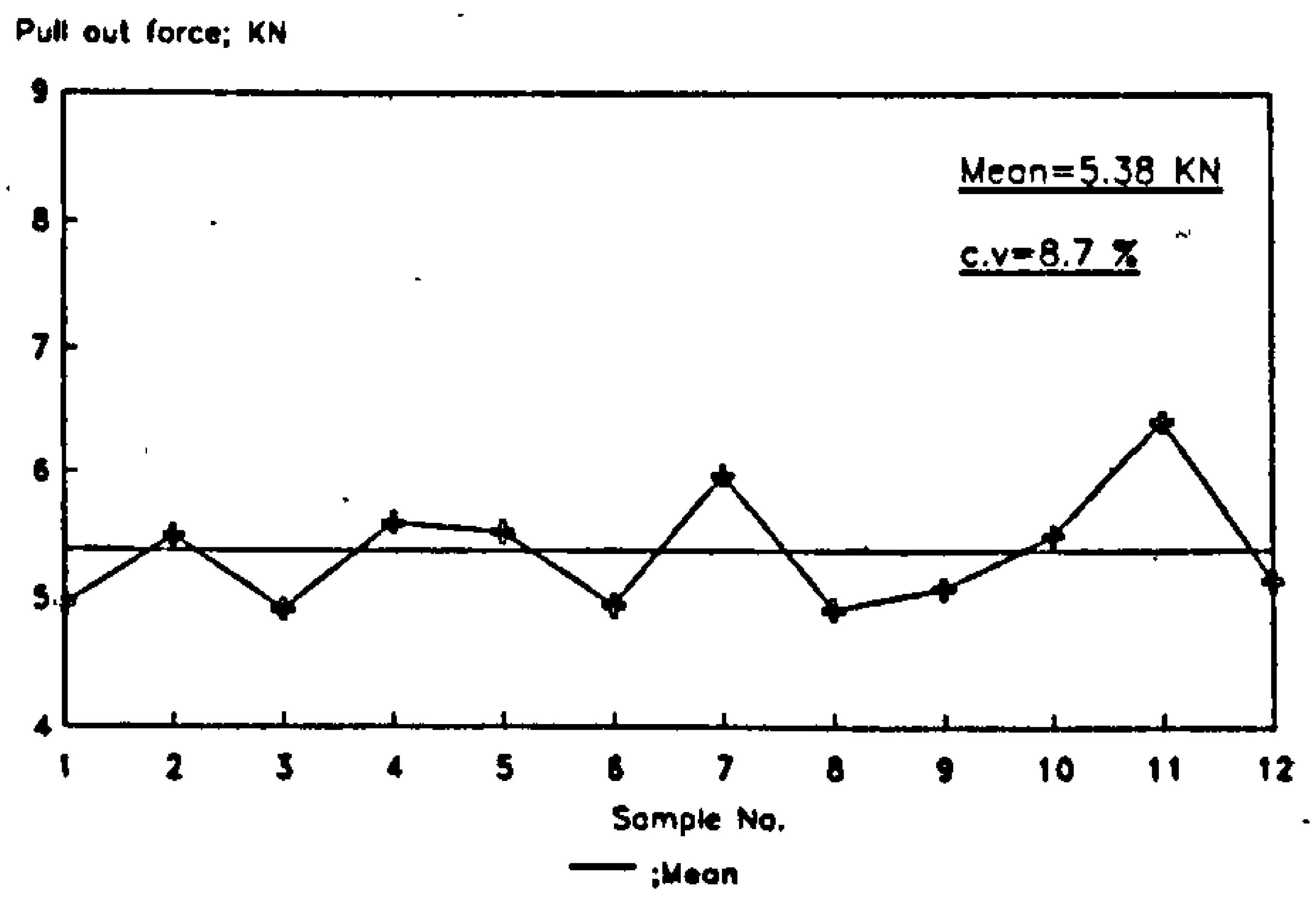


Figure 6.78:Variation of direct pull internal fracture force on Pellite concrete

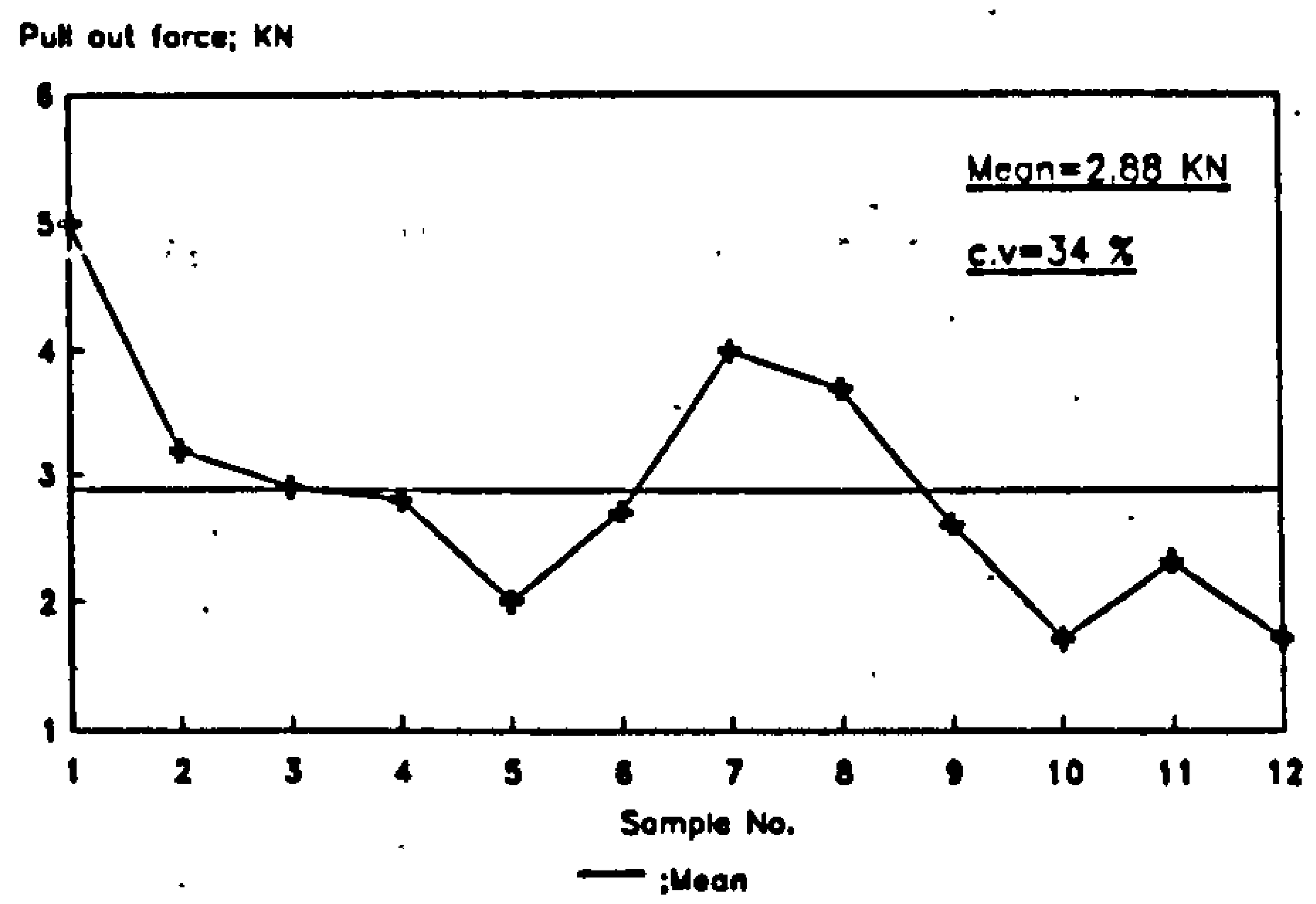


Figure 6.79:Variation of direct pull internal fracture force on Leca concrete

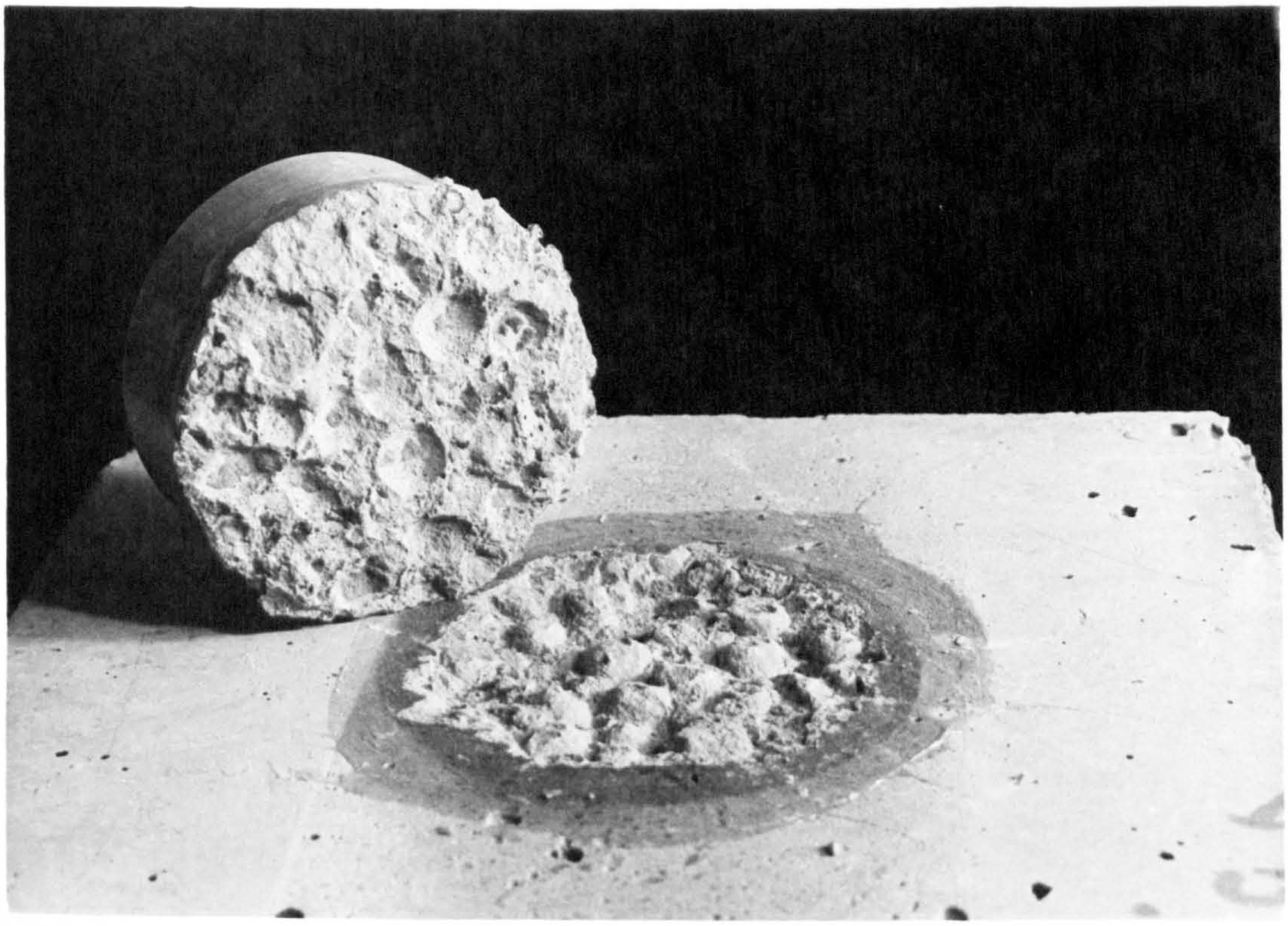


Figure 6.30: Typical failure from surface pull off test on semi Lytag concrete with low strength

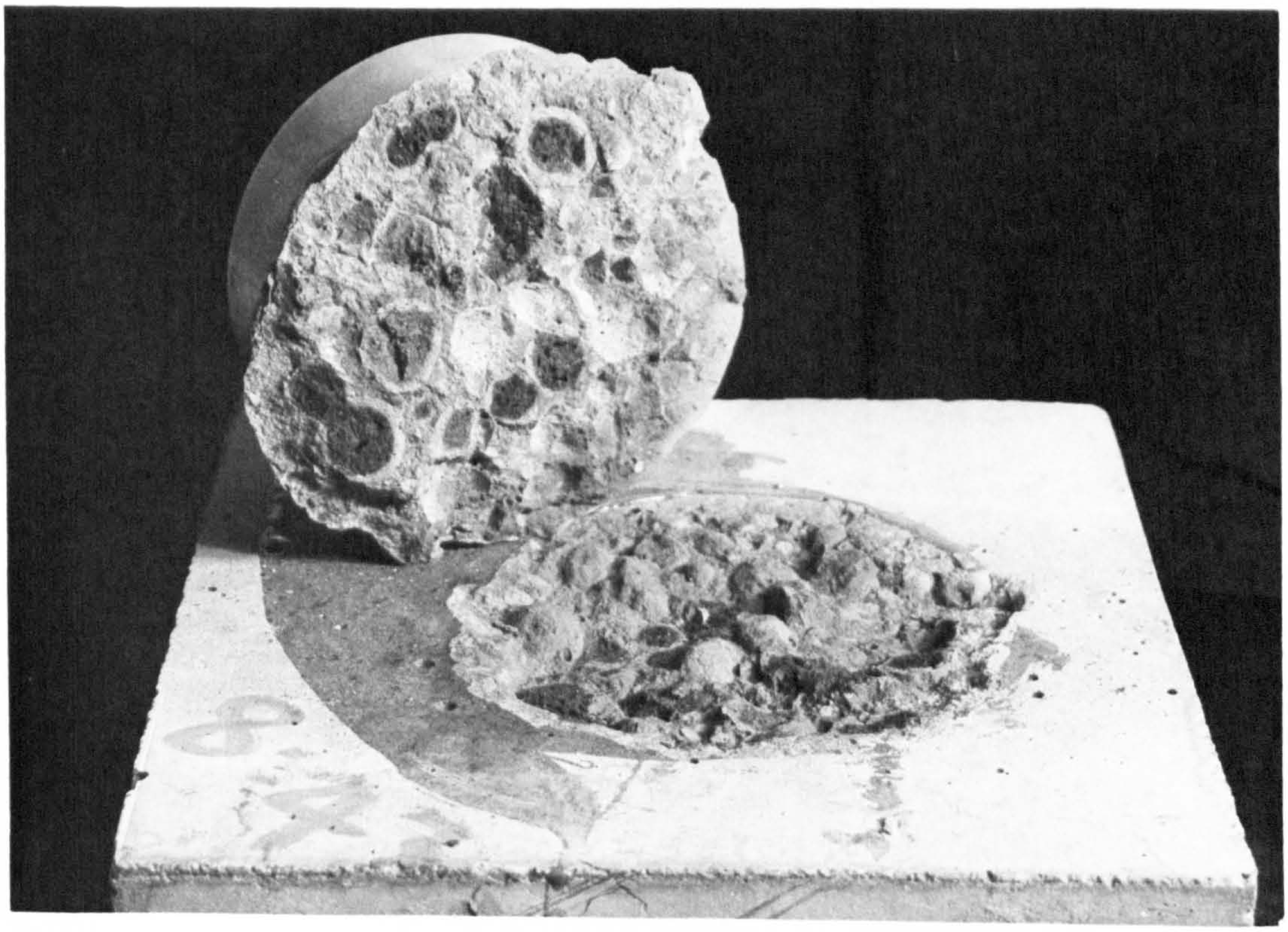


Figure 6.81: Typical failure from surface pull off test on semi Lytag concrete with high strength

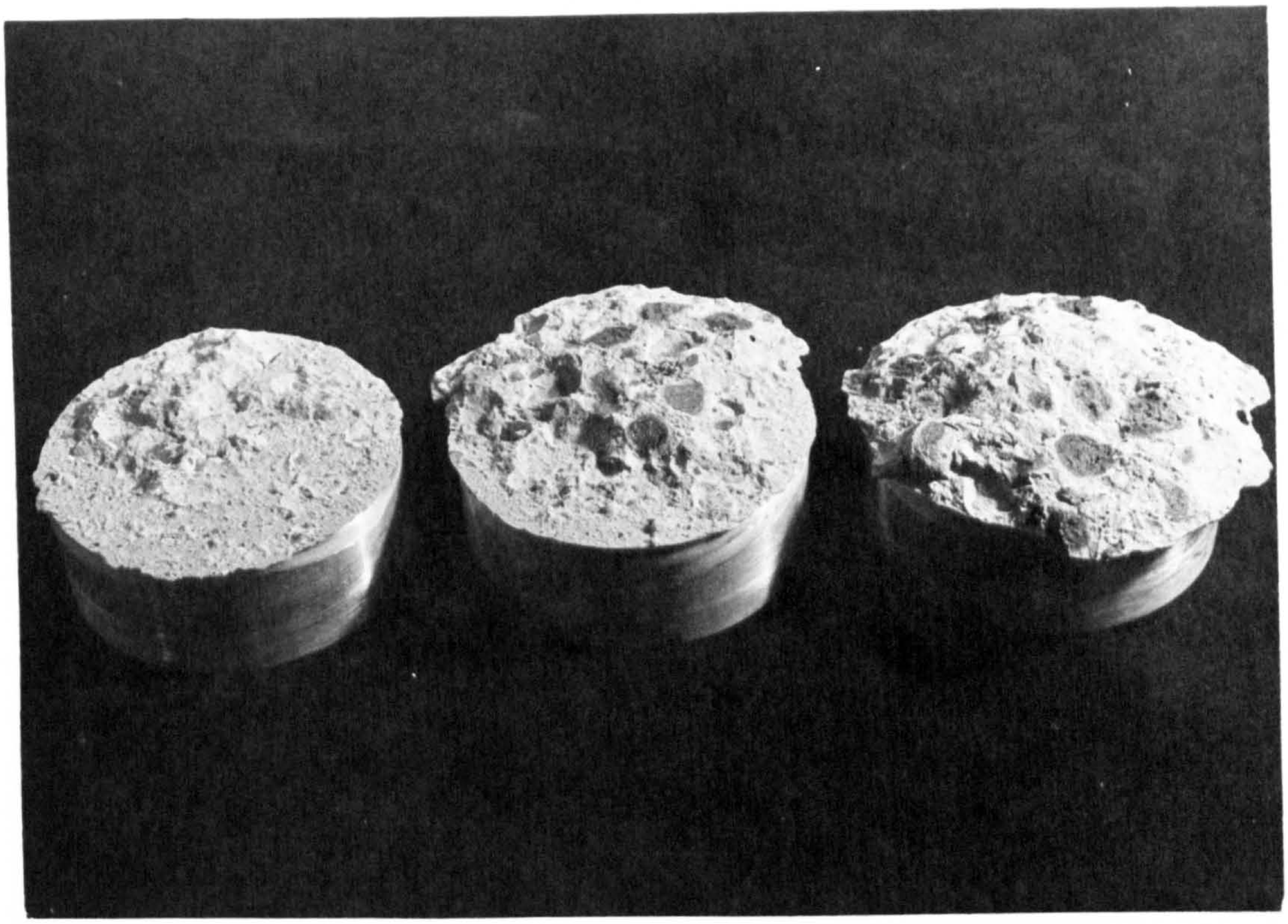


Figure 6.82a: Typical failures from surface pull off tests on fully Lytag concretes, showing variation in the failure depth

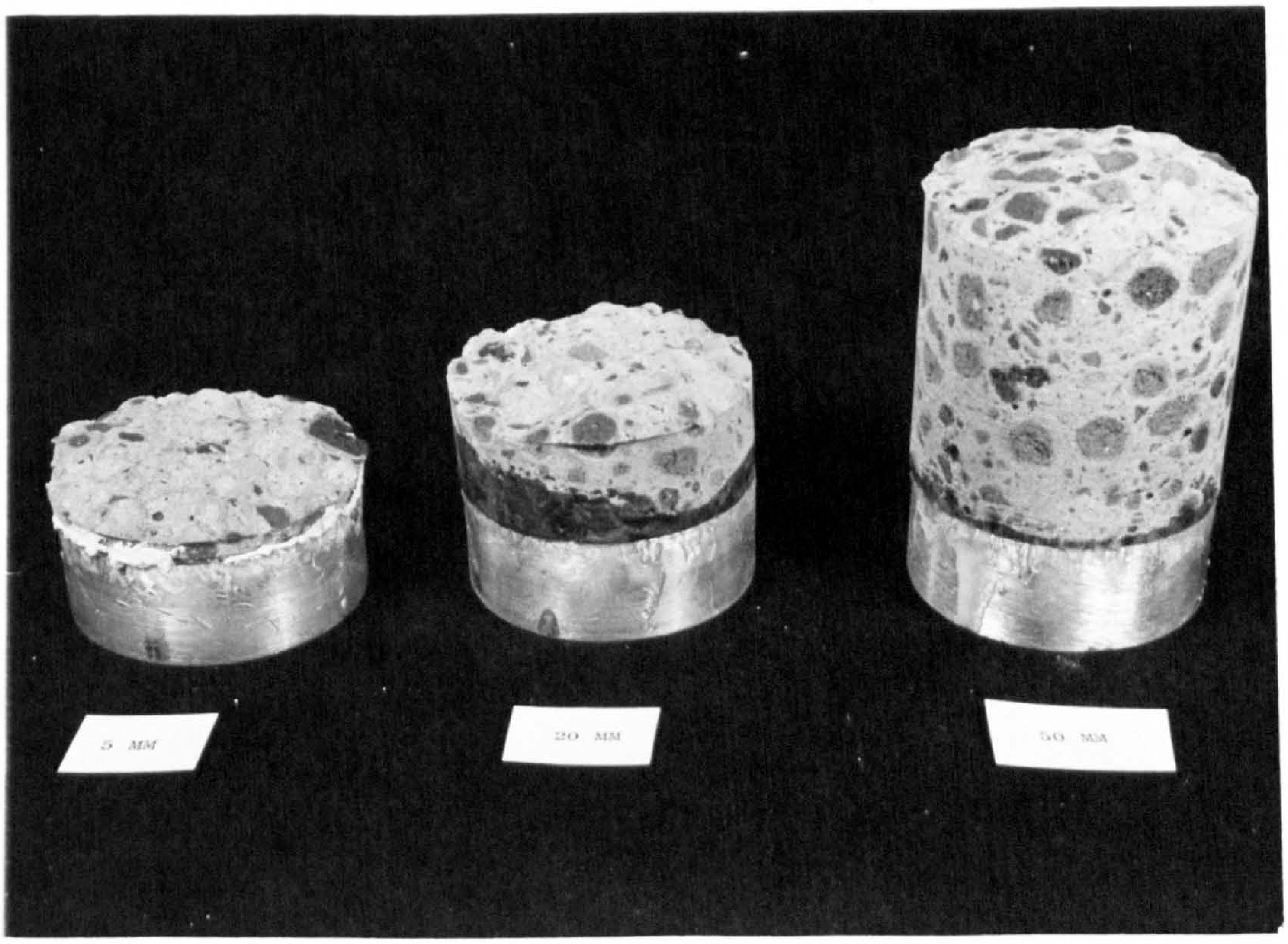


Figure 6.82b: Typical failures from partial cored pull off tests on fully Lytag concretes with different core lengths

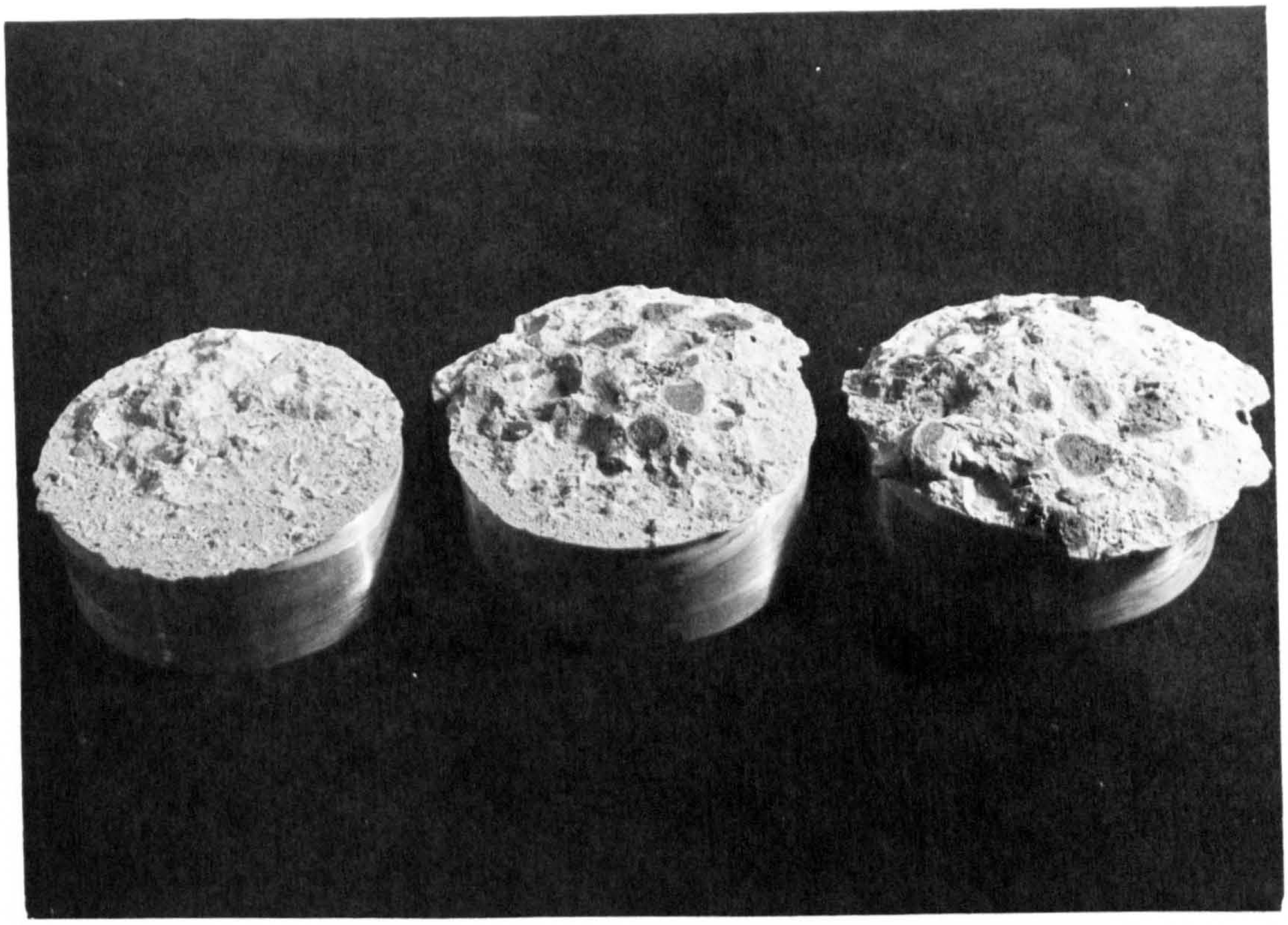


Figure 6.82a: Typical failures from surface pull off tests on fully Lytag concretes, showing variation in the failure depth

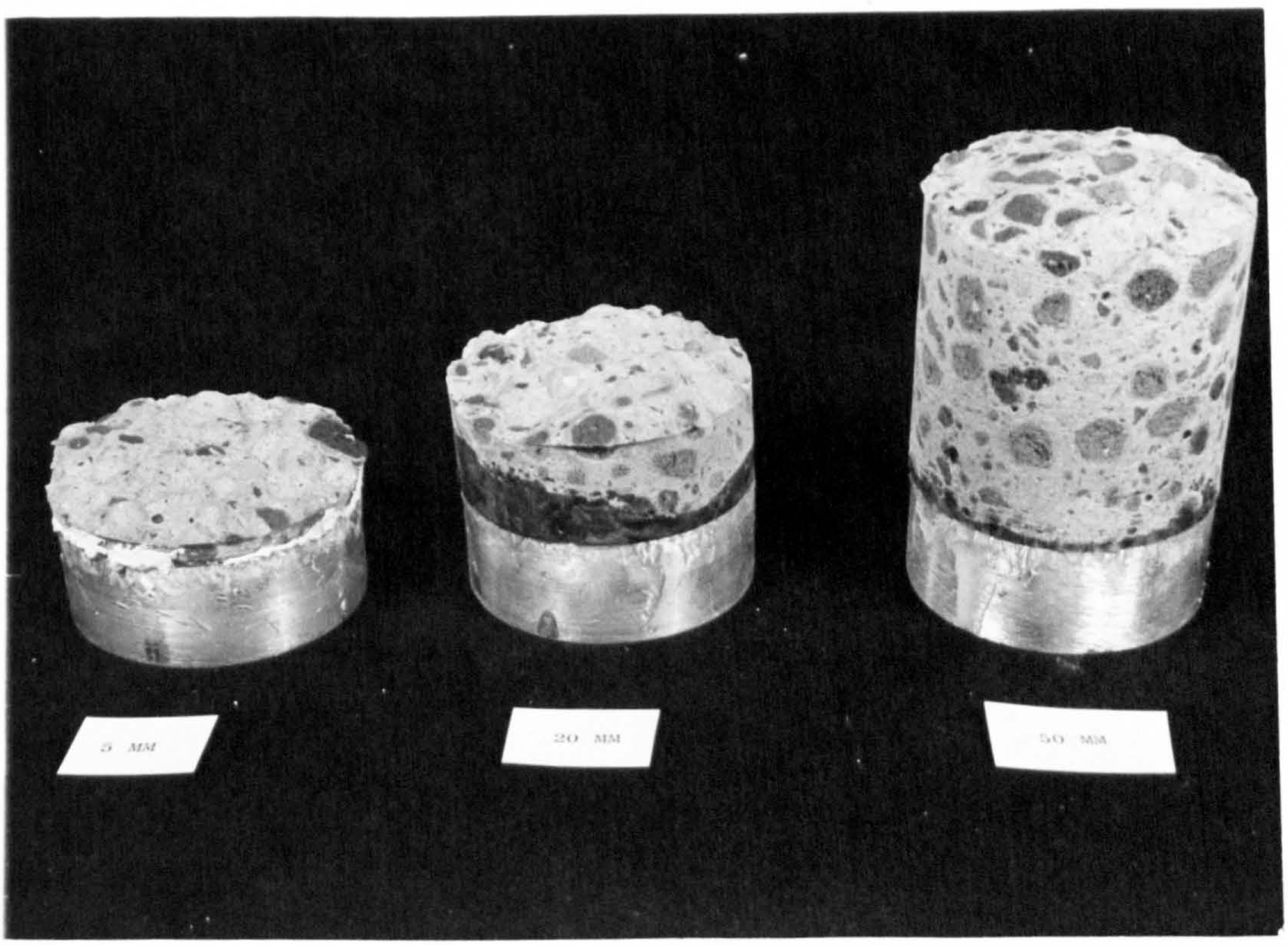


Figure 6.82b: Typical failures from partial cored pull off tests on fully Lytag concretes with different core lengths

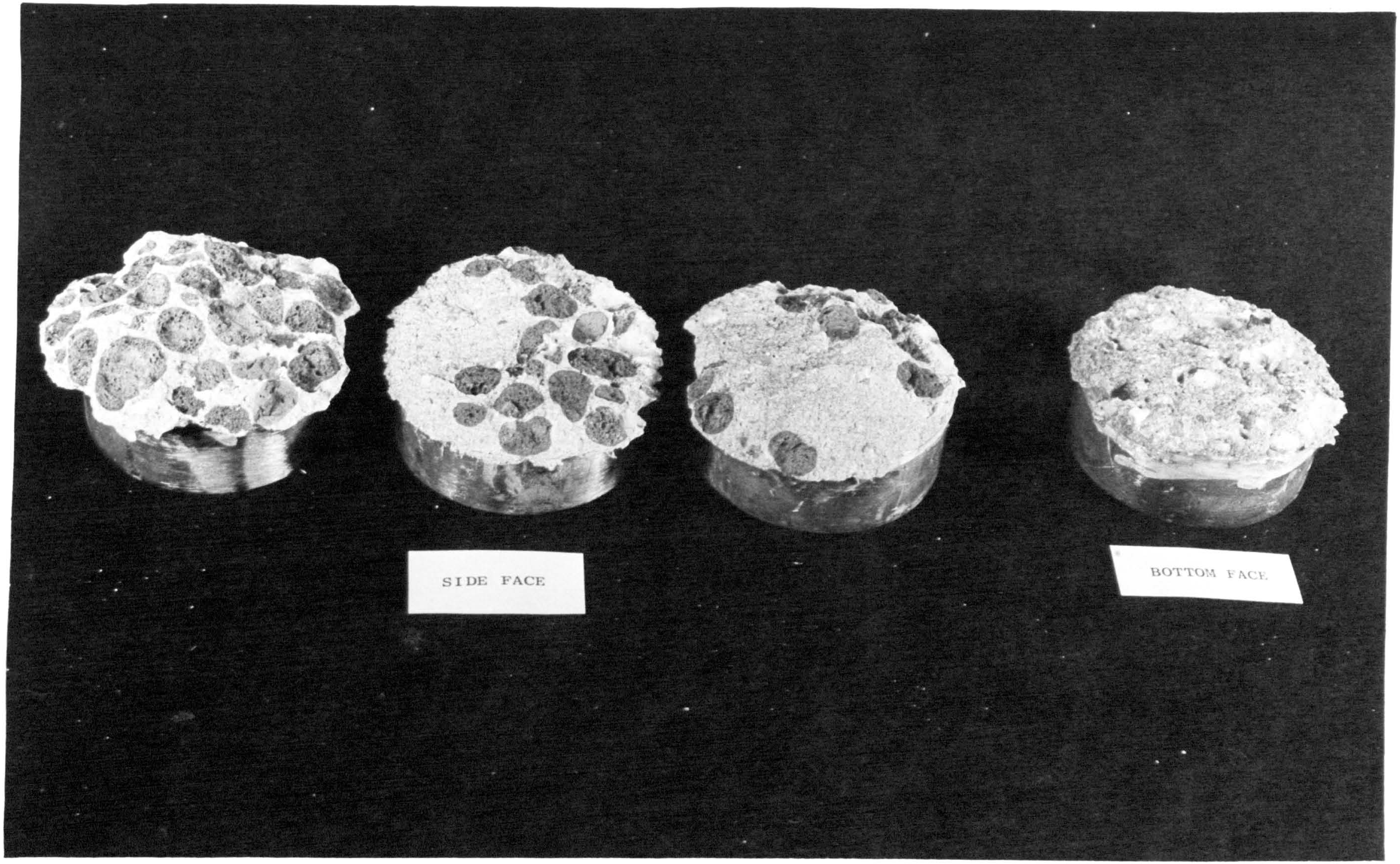


Figure 6.83: Typical failures from surface pull off tests on Leca concrete, showing non-uniform distribution of aggregates

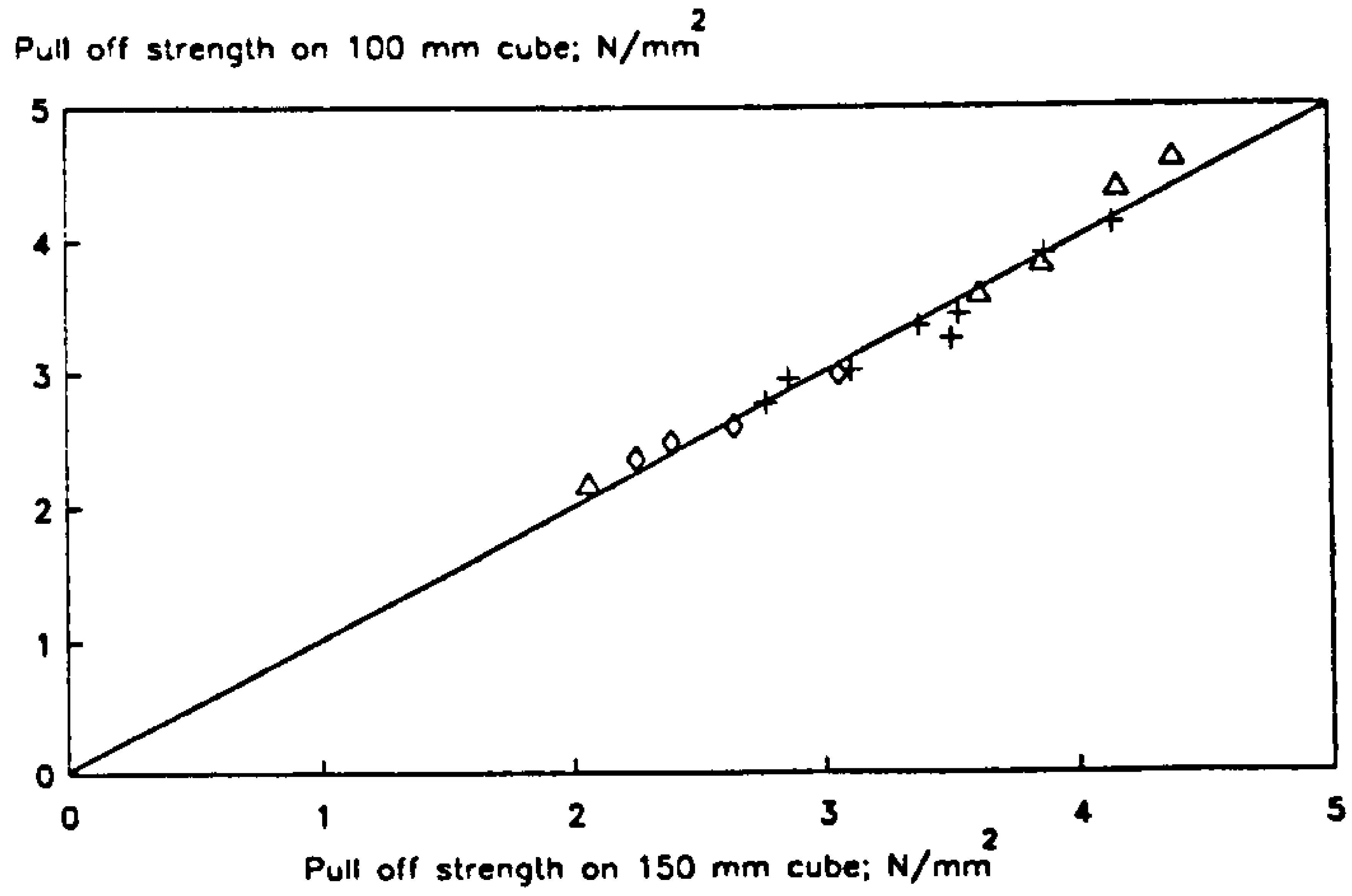


Figure 6.84:Relationship between pull off strength on 150 and 100 mm cube specimens

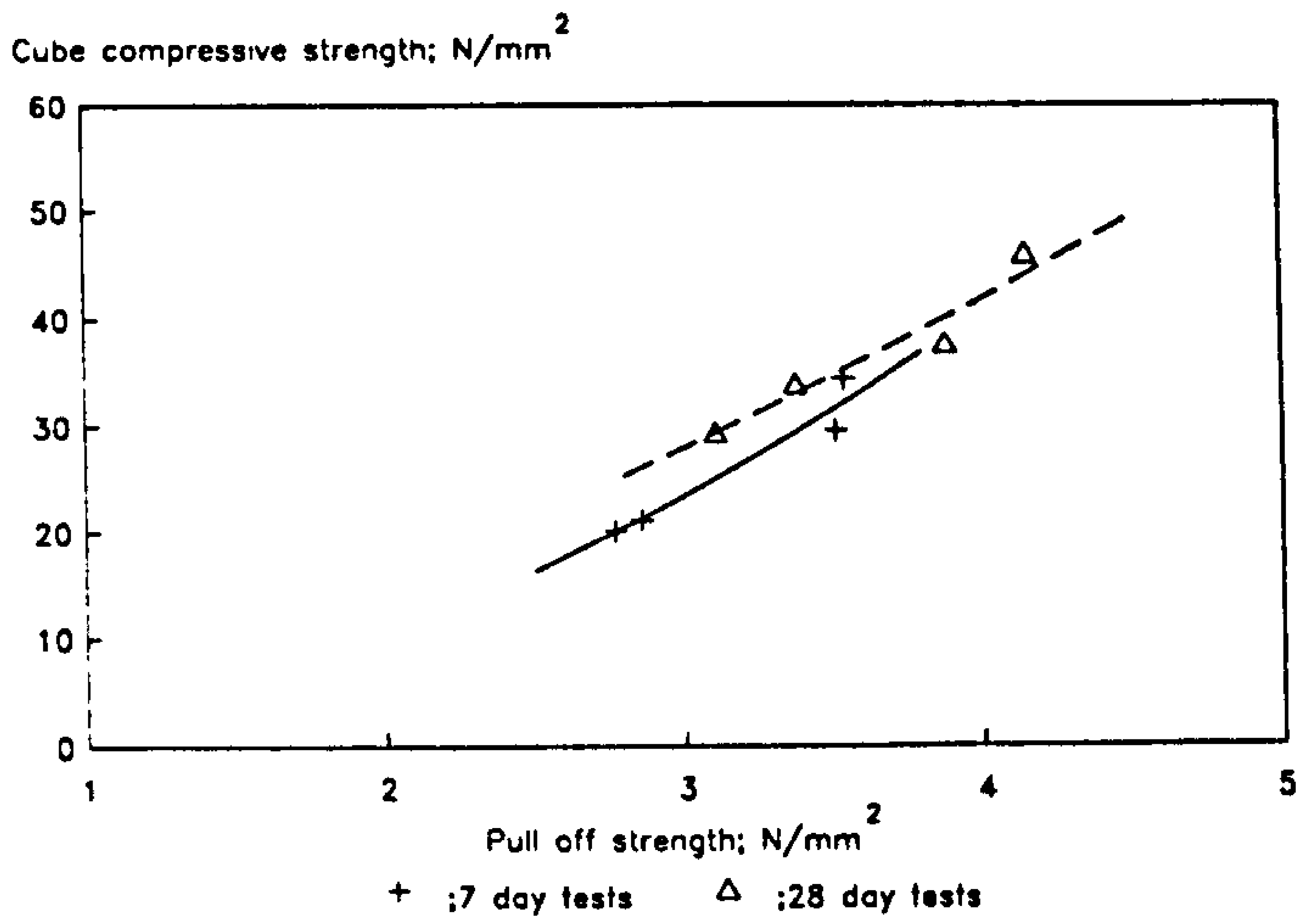


Figure 6.85:Relationships between surface pull off and compressive strengths for 7 and 28 day tests on fully Lytag concrete

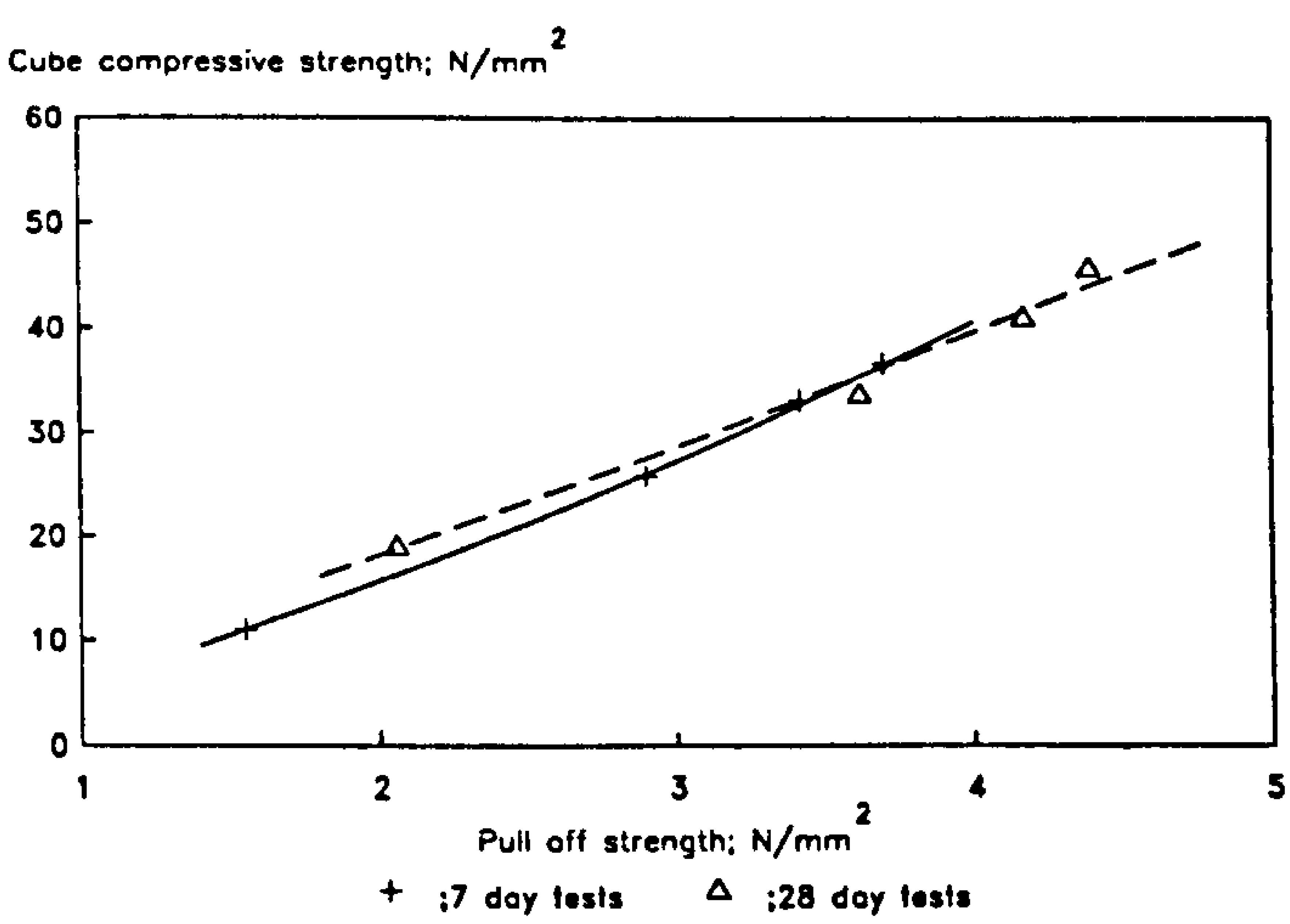


Figure 6.86:Relationships between surface pull off and compressive strengths for 7 and 28 day tests on semi Lytag concrete

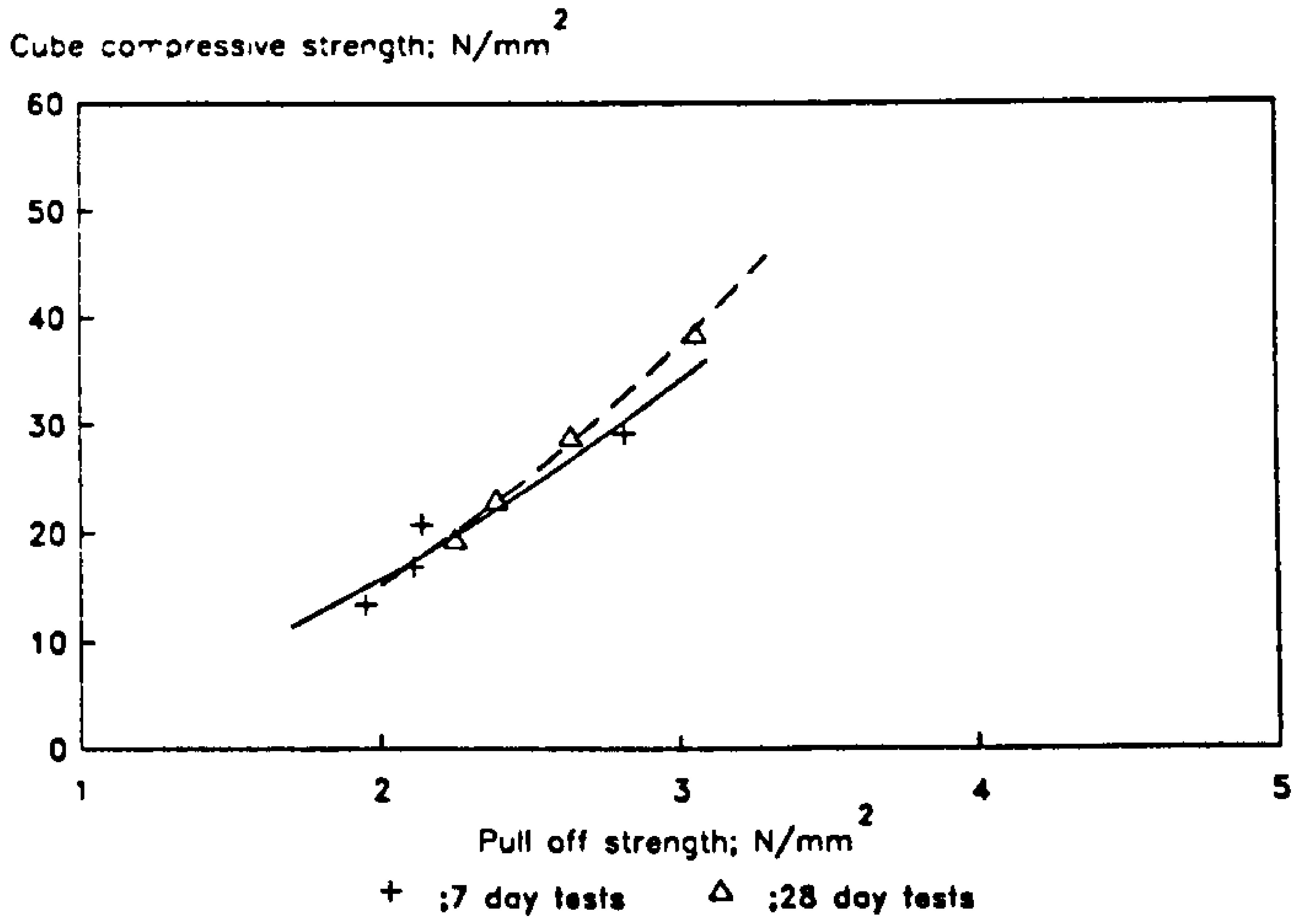


Figure 6.87:Relationships between surface pull off and compressive strengths for 7 and 28 day tests on normal weight concrete

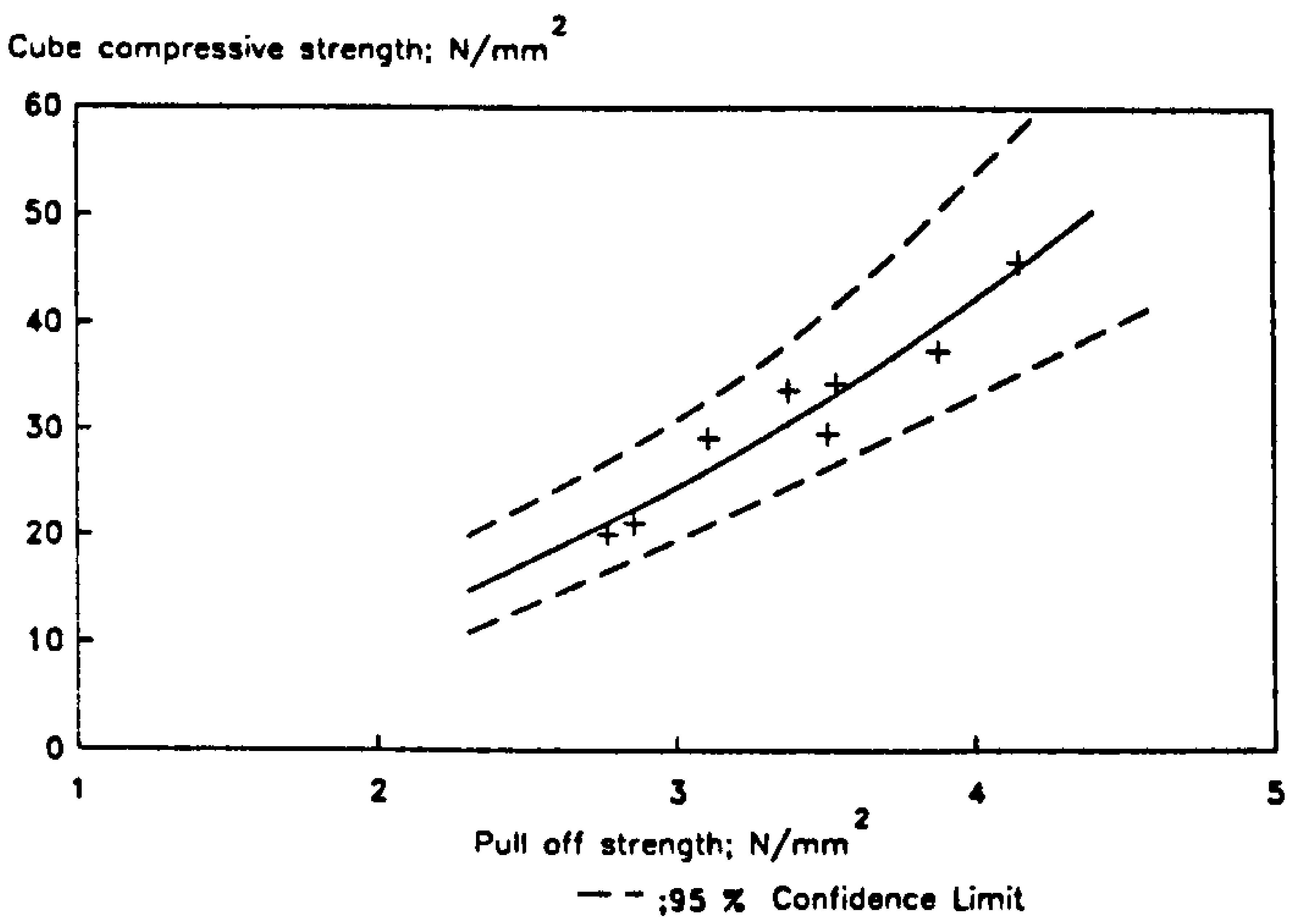


Figure 6.88:Relationship between surface pull off and compressive strengths of fully Lytag concrete in short term

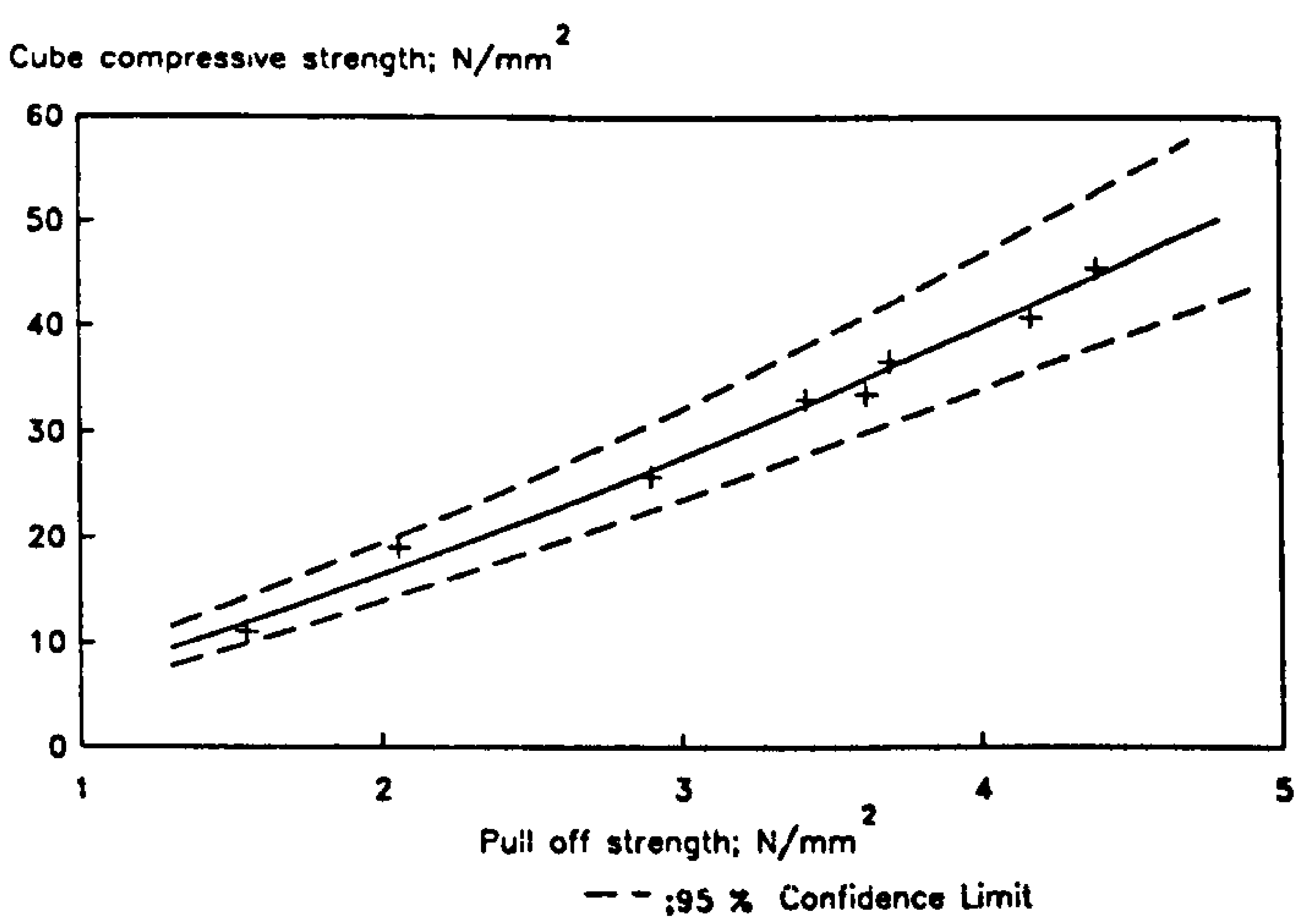


Figure 6.89:Relationship between surface pull off and compressive strengths of semi Lytag concrete at various ages

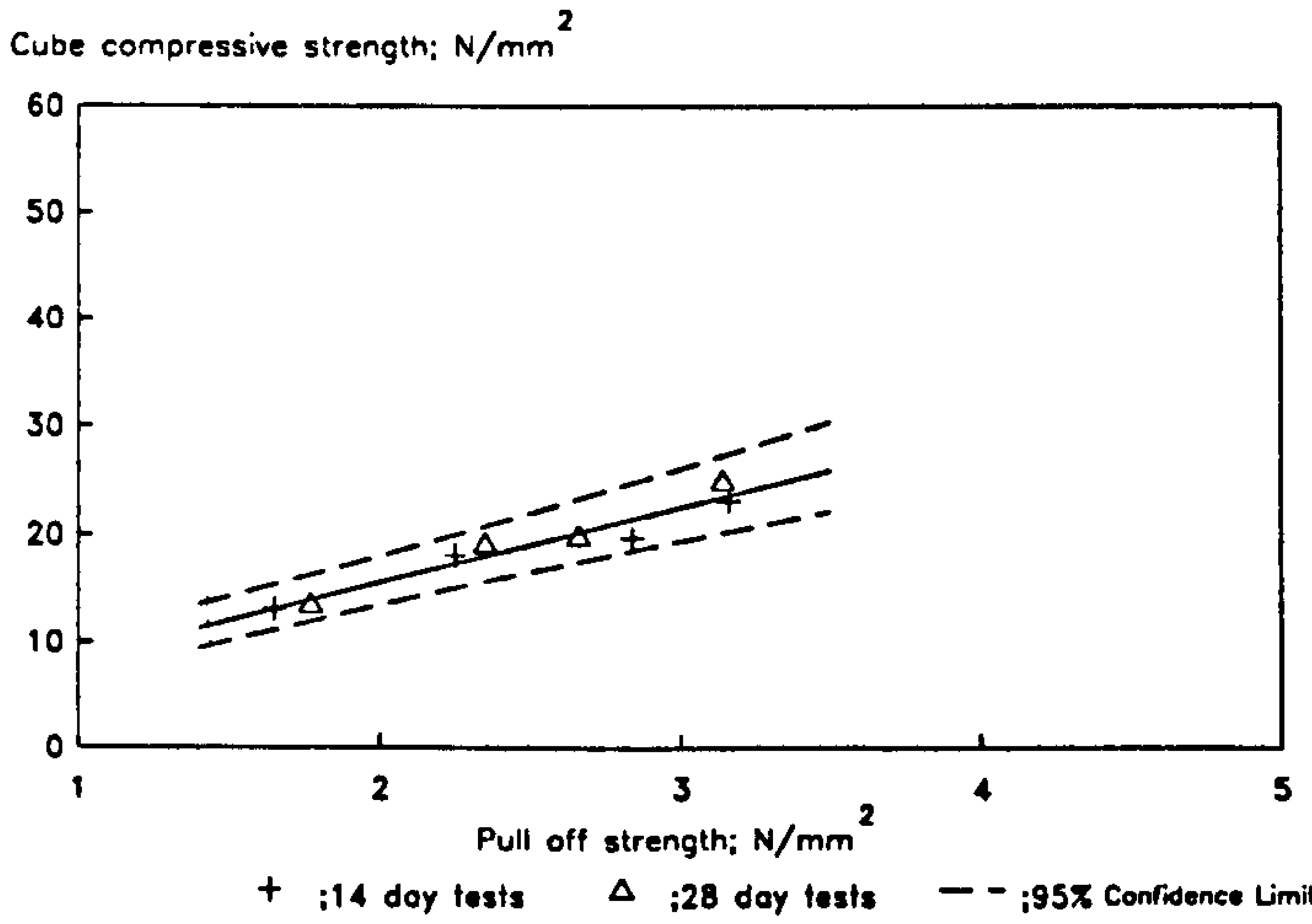


Figure 6.90:Relationship between surface pull off and compressive strengths of Leca concrete at various ages

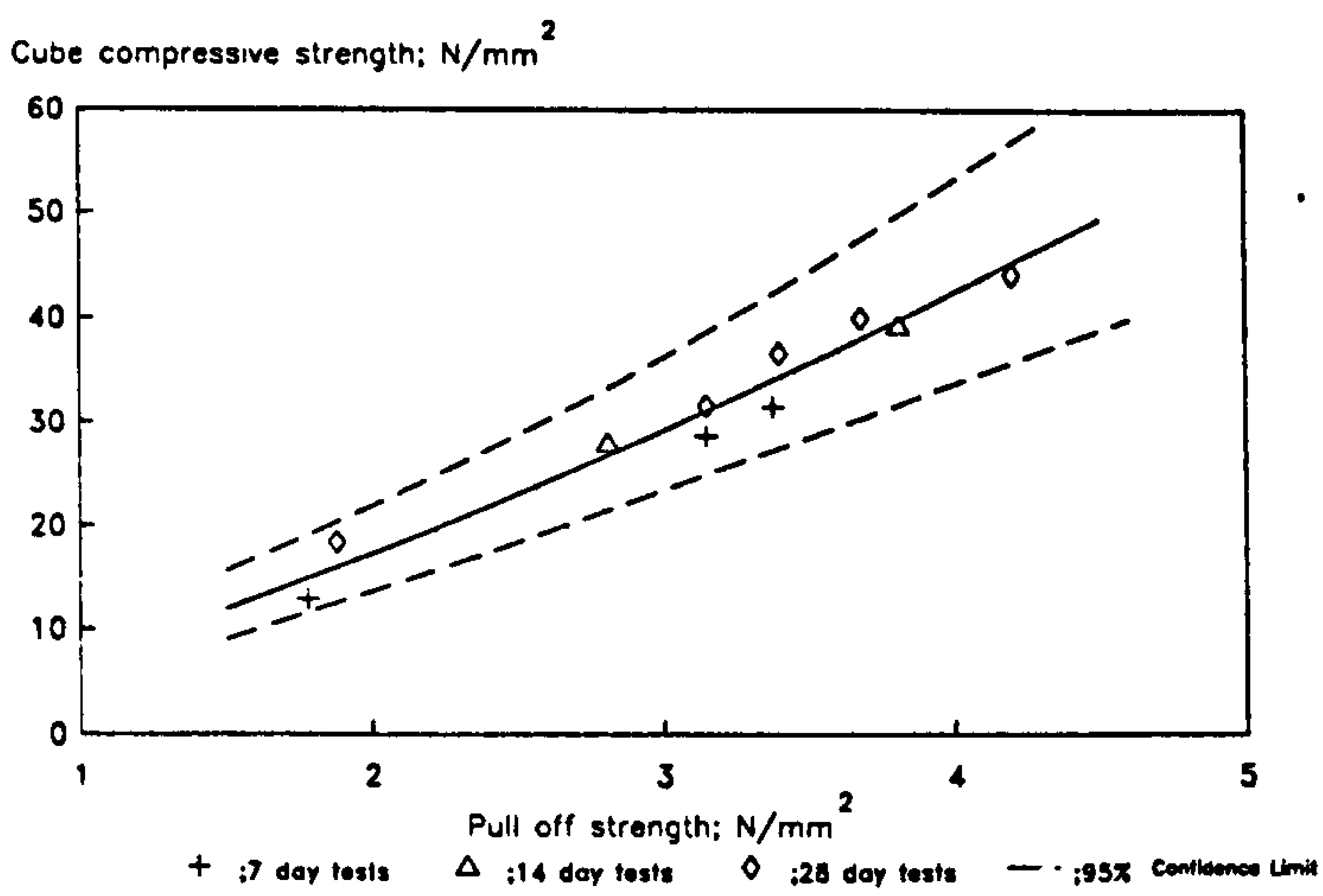


Figure 6.91:Relationship between surface pull off and compressive strengths of Pellite concrete at various ages

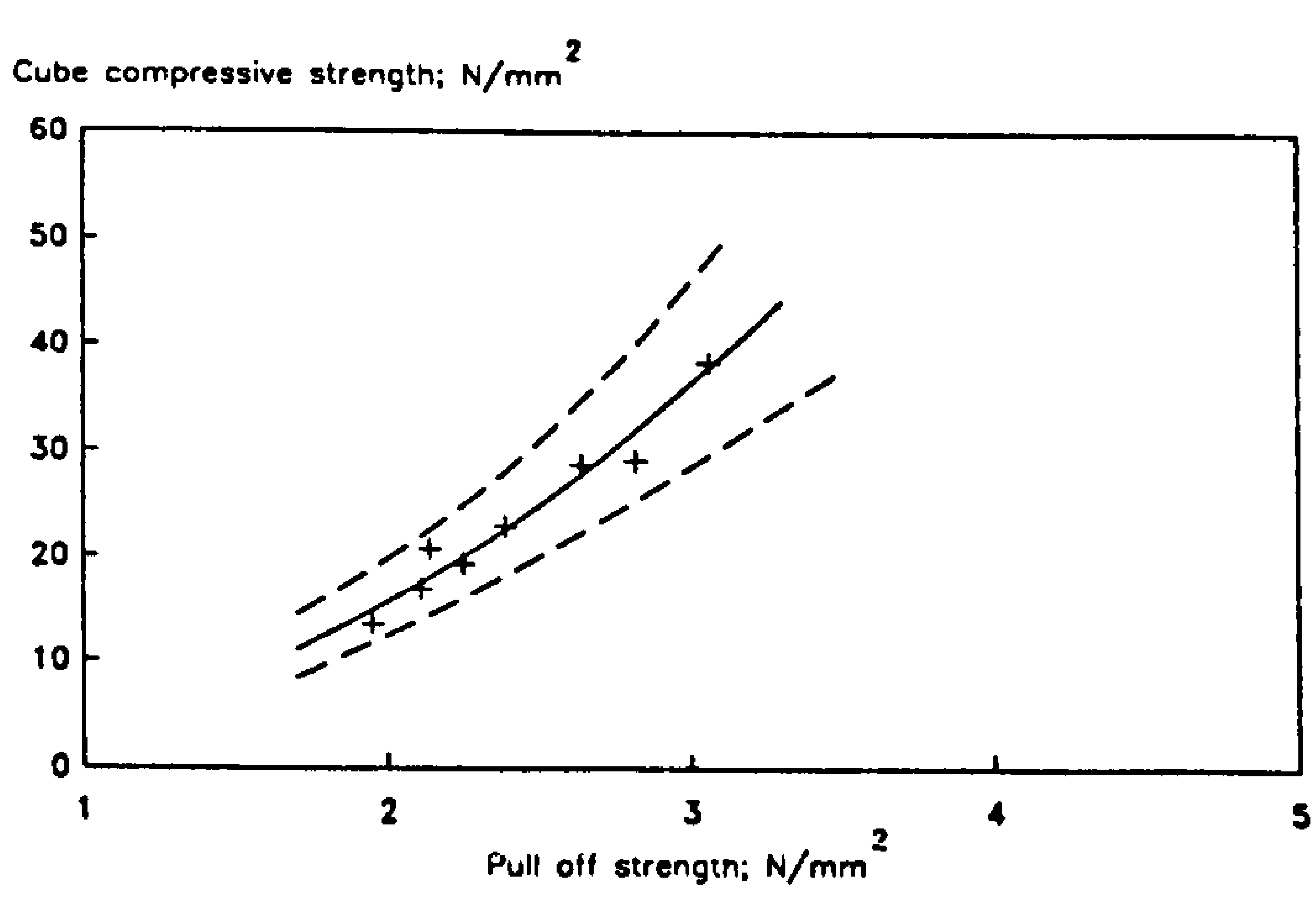


Figure 6.92:Relationships between surface pull off and compressive strengths of normal weight concrete at various ages

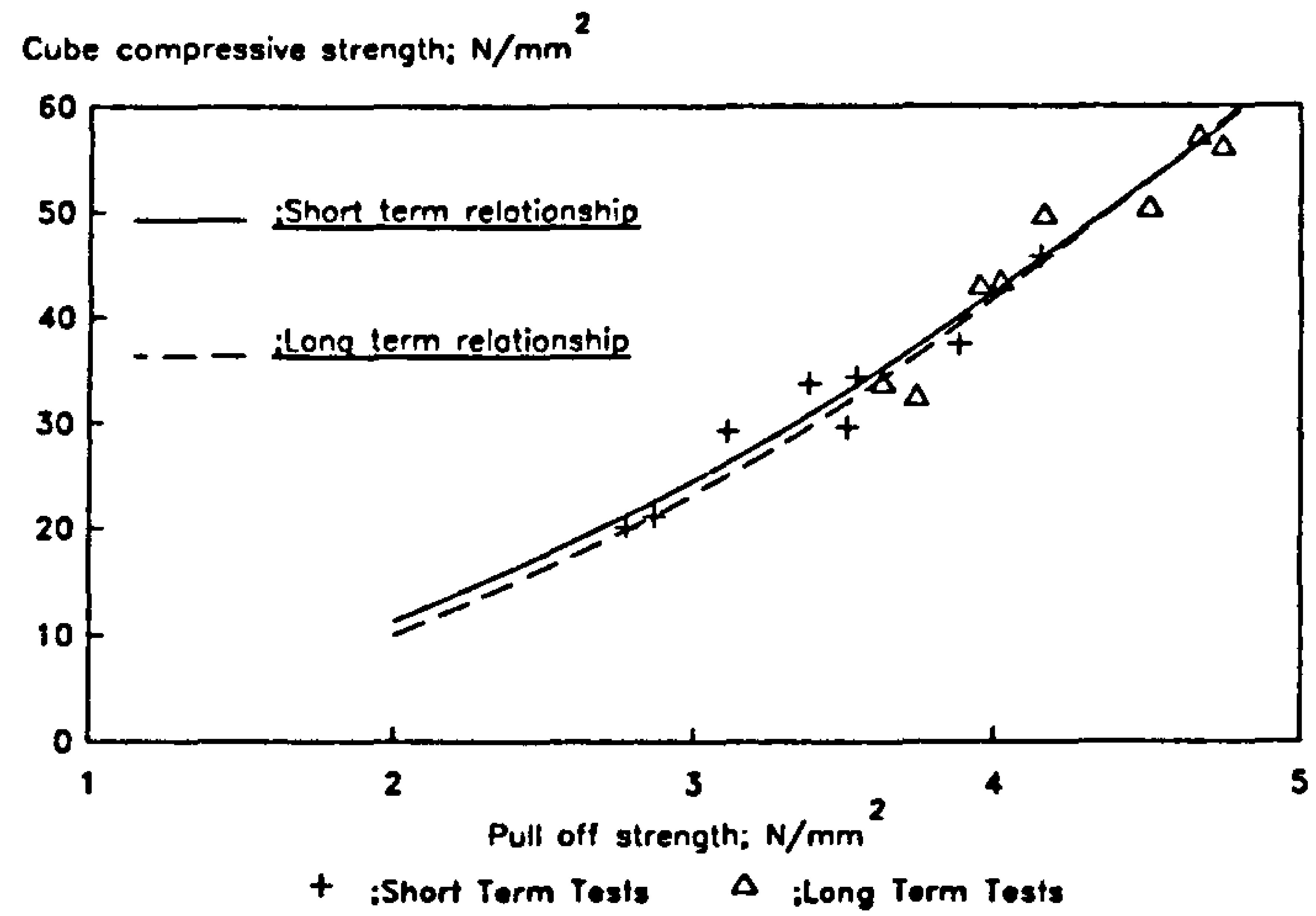


Figure 6.93:relationships between surface pull off and compressive strengths of fully Lytag concrete at short and long terms

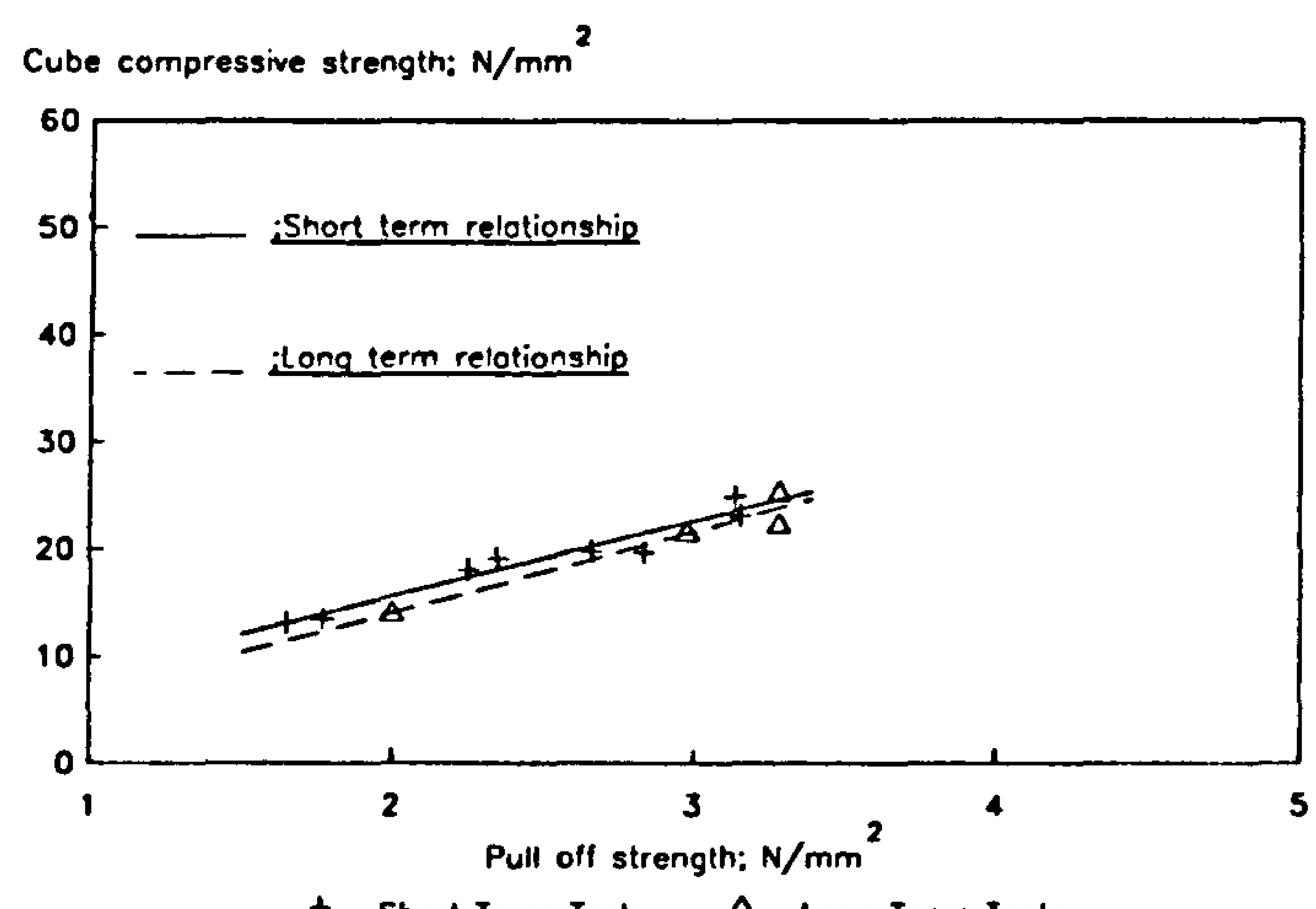


Figure 6.94:relationships between surface pull off and compressive strengths of Leca concrete at short and long terms

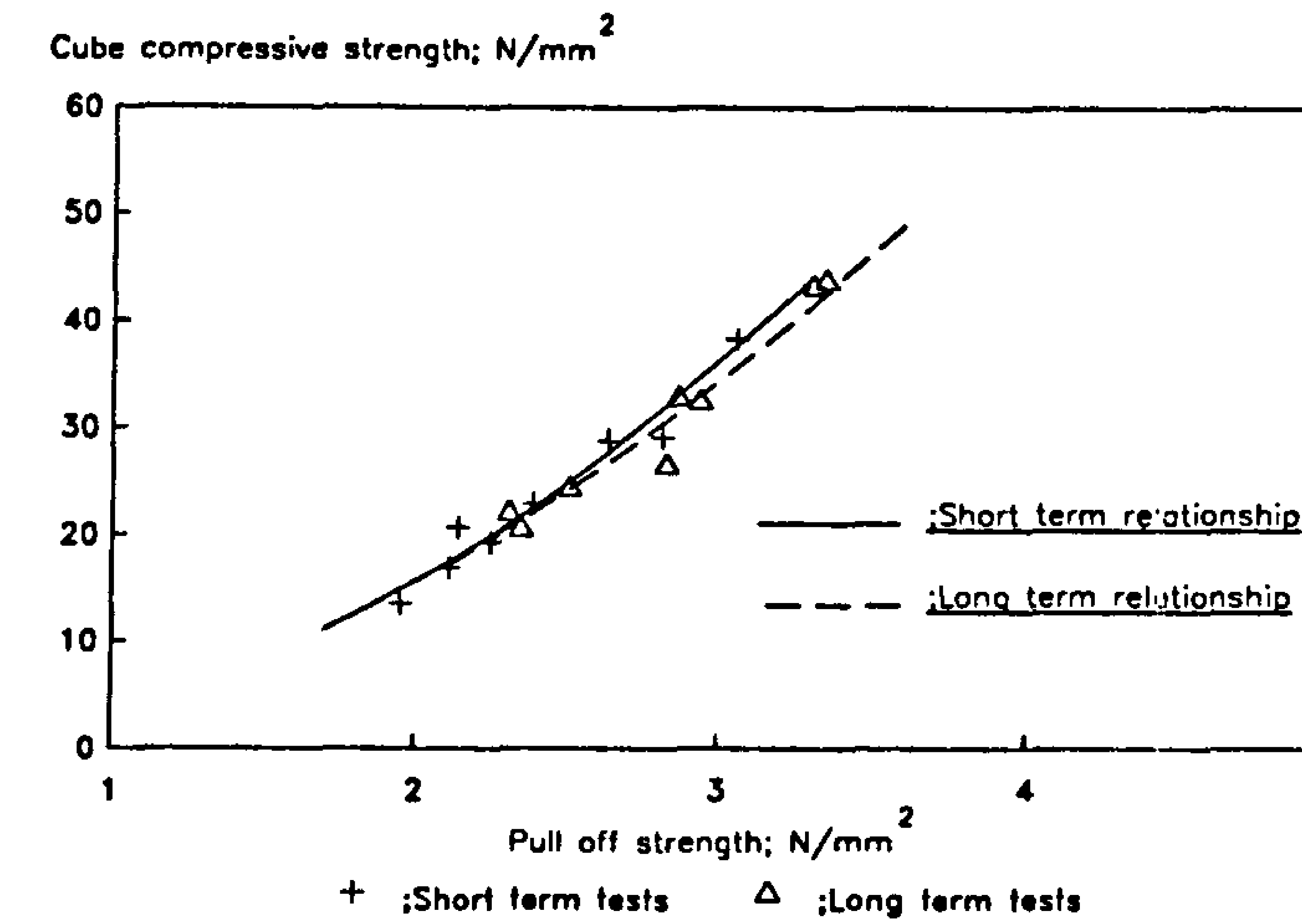


Figure 6.95.Relationships between surface pull off and compressive strengths of normal weight concrete in short and long terms

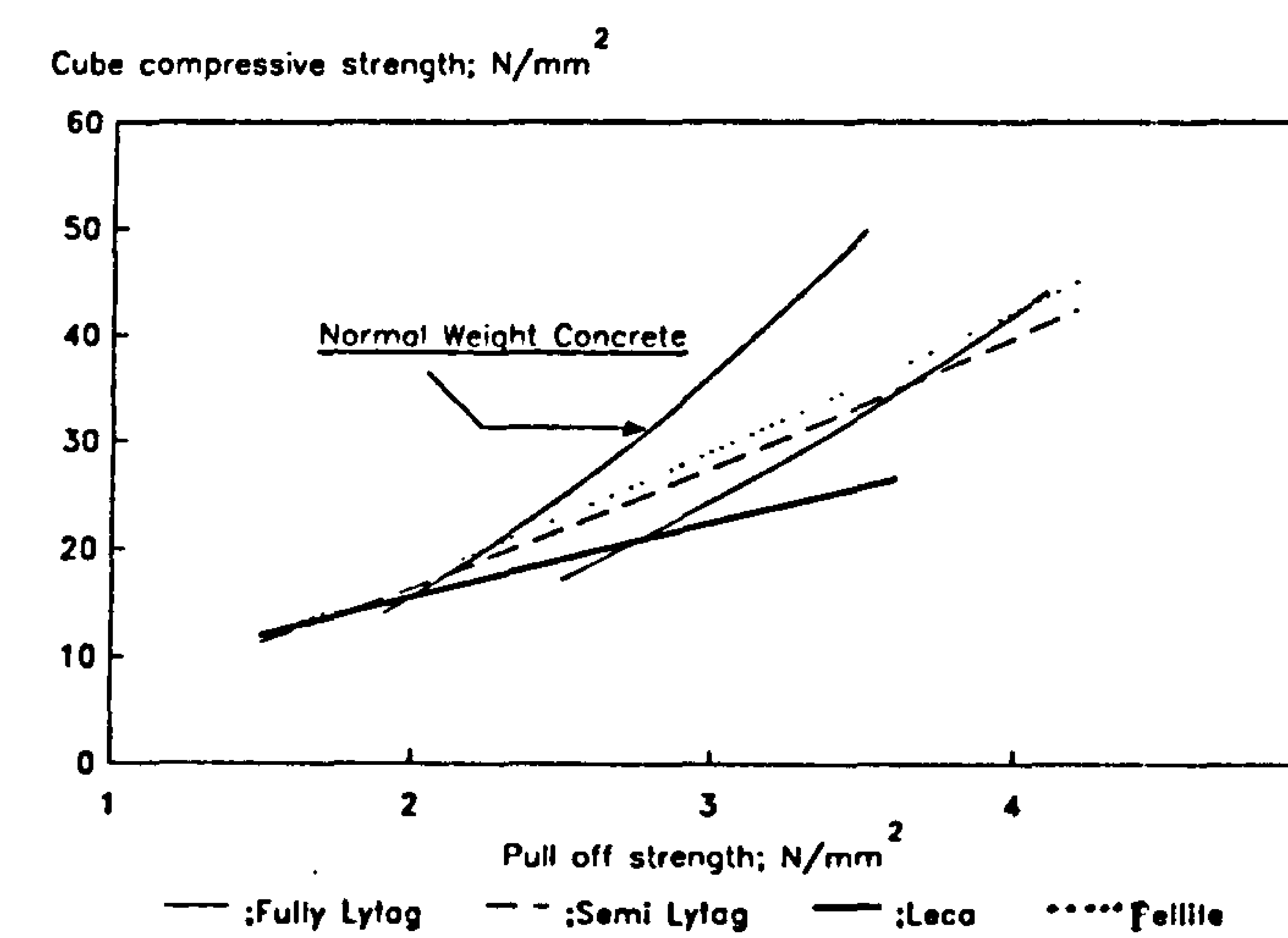


Figure 6.96:Comparison of relationships between surface pull off and compressive strengths of different types of concrete

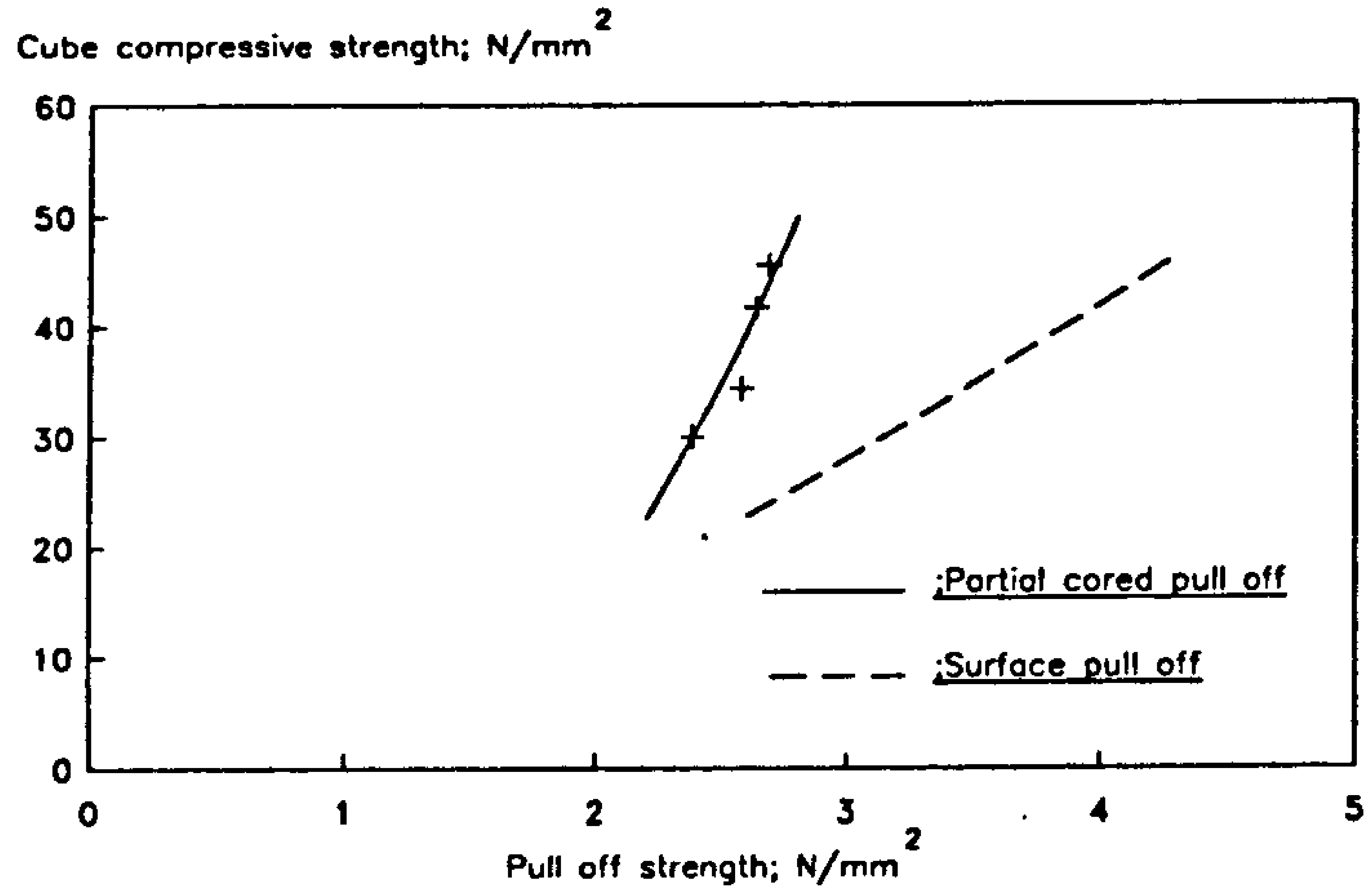


Figure 6.97:Relationships between pull off and compressive strengths of fully Lytag concrete at the age of 28 days

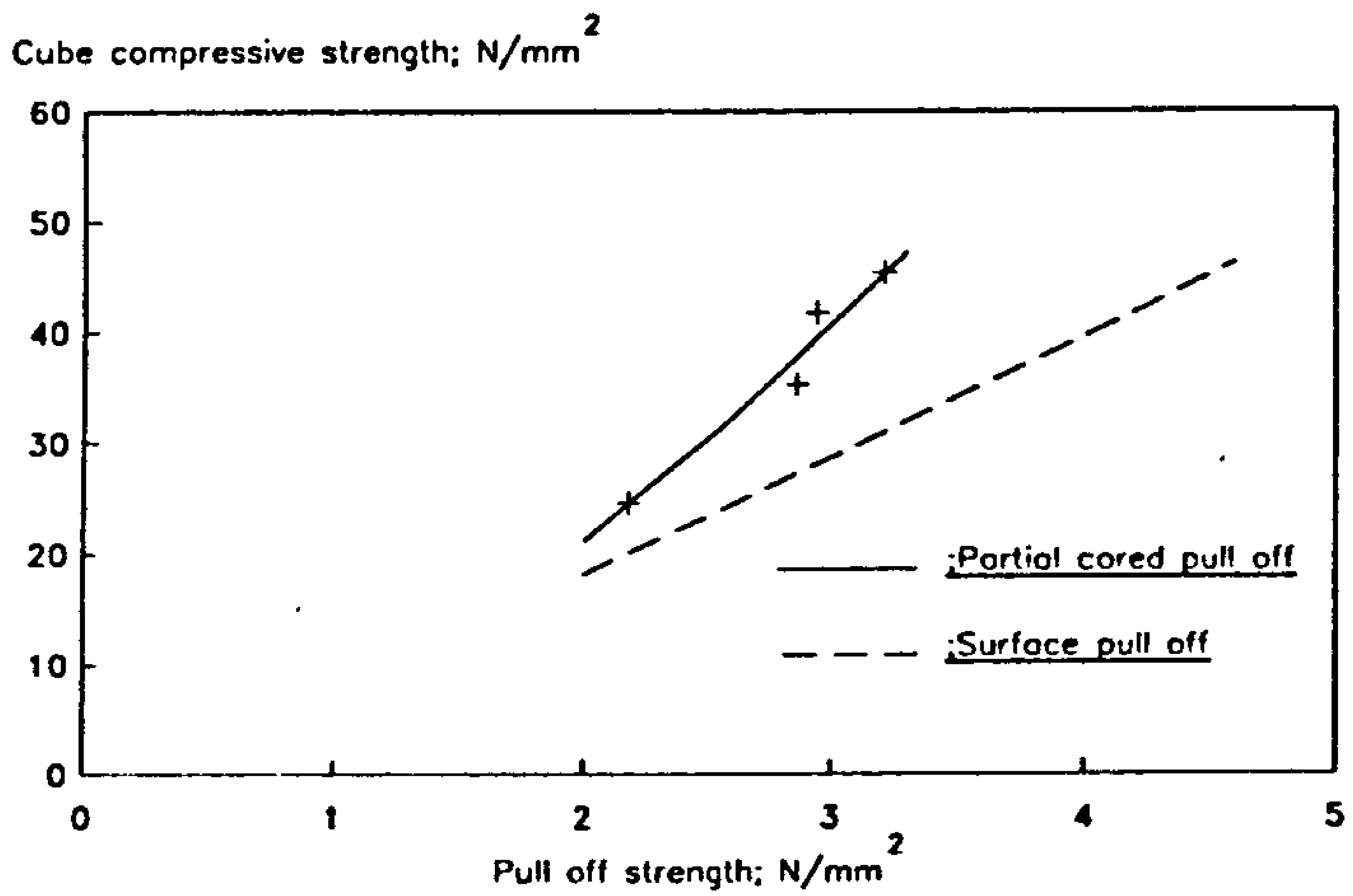


Figure 6 98.Relationships between pull off and compressive strengths of semi Lytag concrete at the age of 28 days

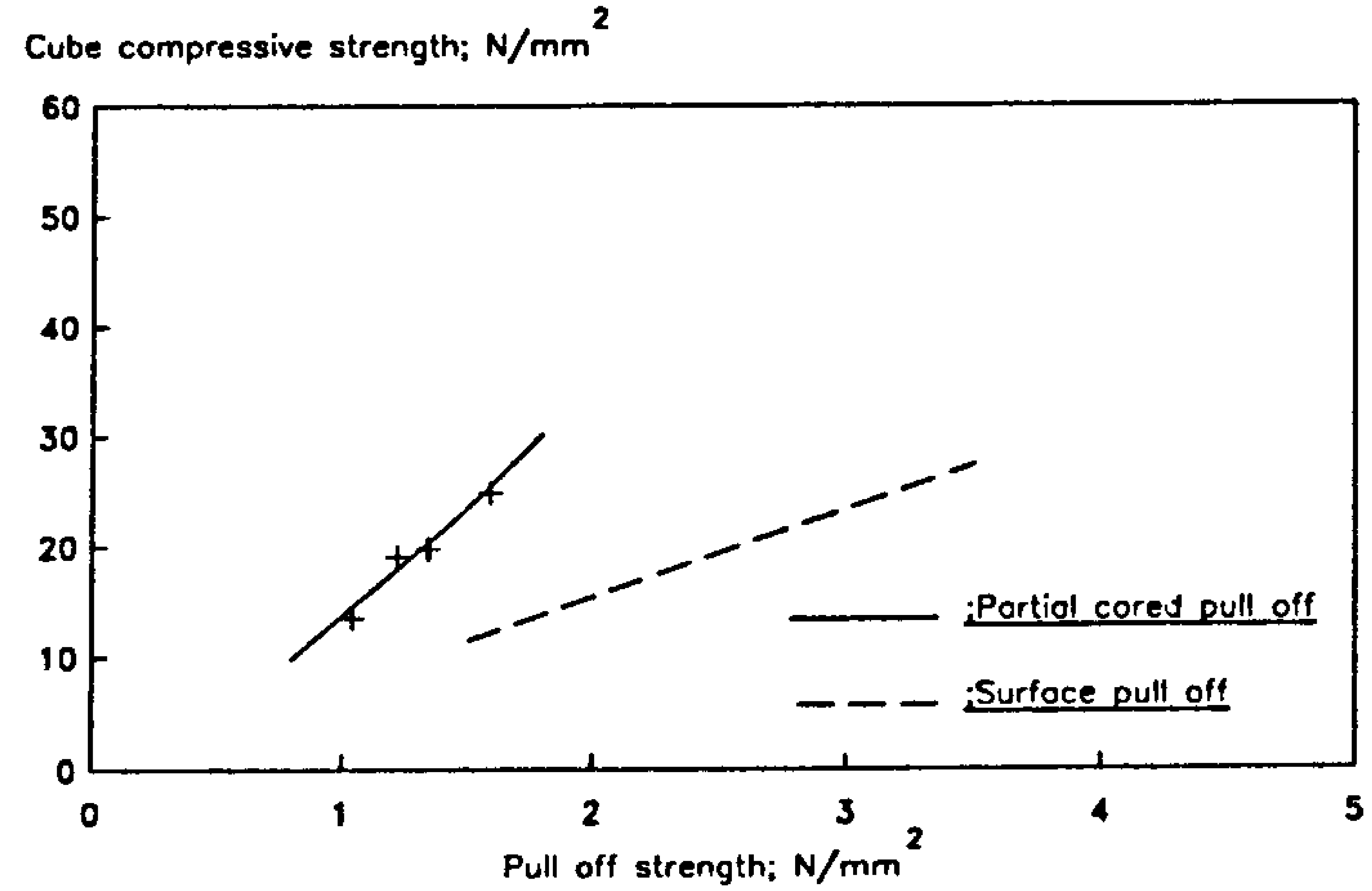


Figure 6.99:Relationships between pull off and compressive strengths of Leca concrete at the age of 28 days

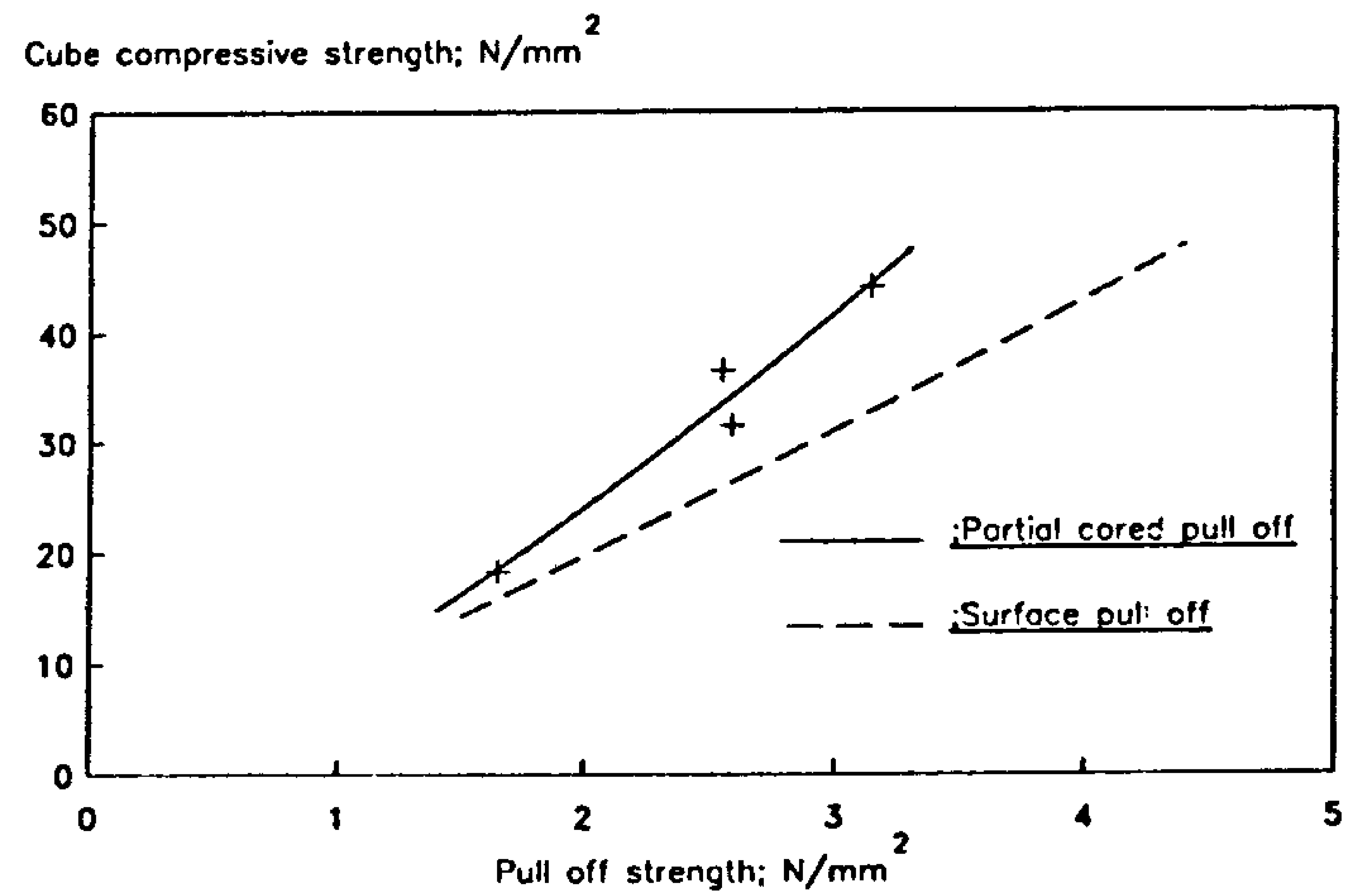


Figure 6 100:Relationships between pull off and compressive strengths of Pellite concrete at the age of 28 days

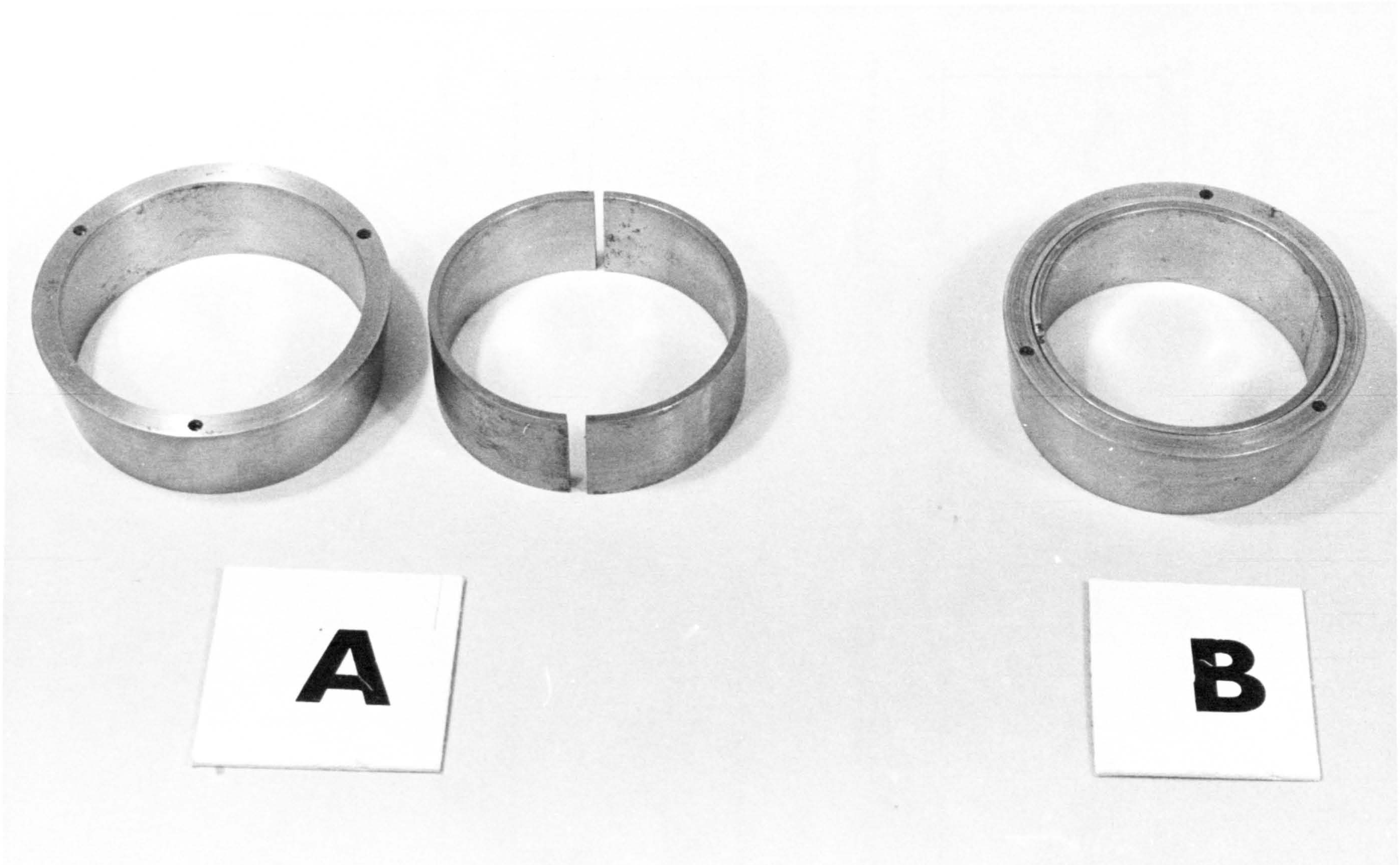


Figure 6.101: Split core former, showing (A) the outer and inner rings and (B) the assembled form

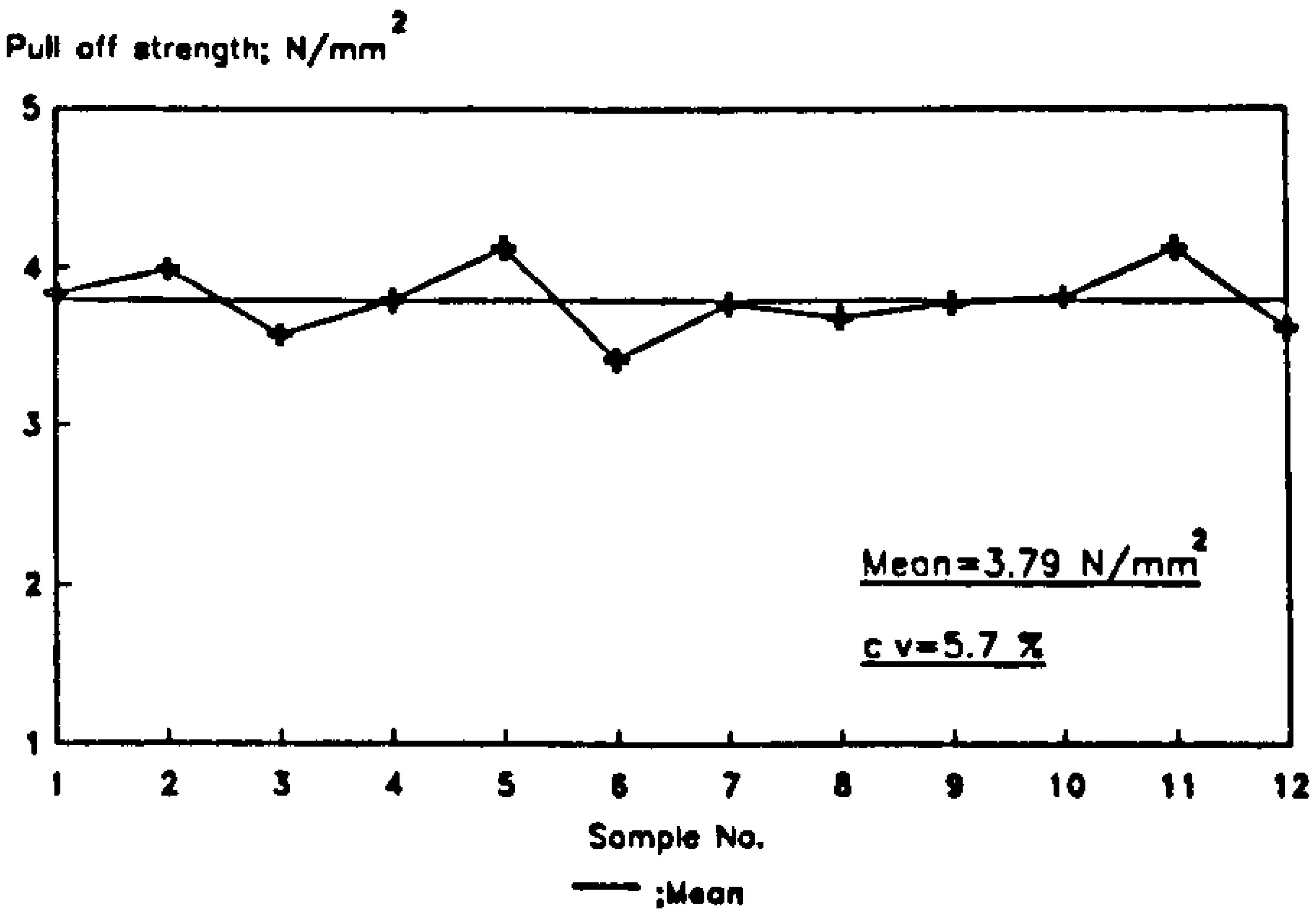


Figure 6.102:Typical variation of surface pull off strengths on fully Lytag concrete

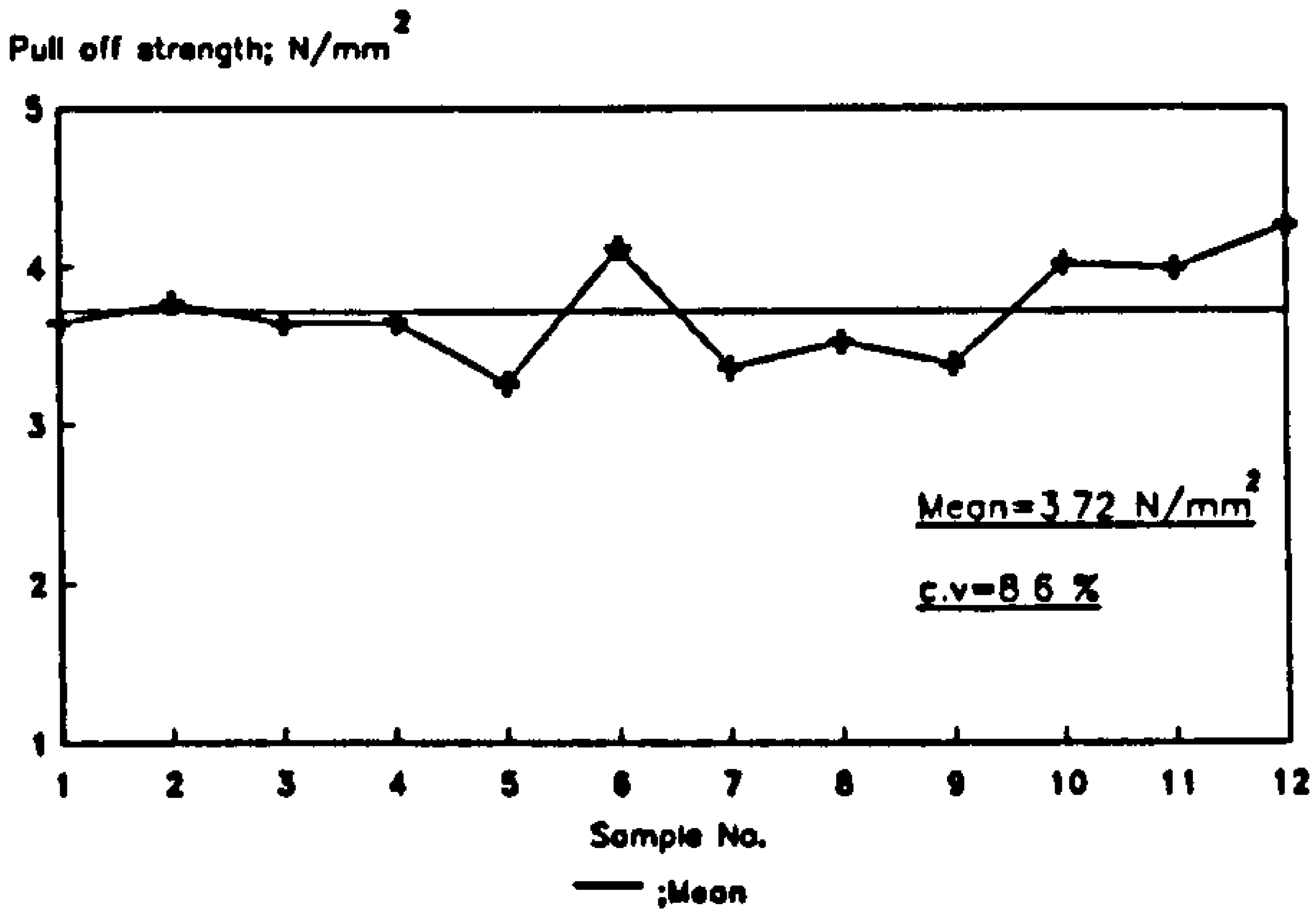


Figure 6.103:Typical variation of surface pull off strengths on semi Lytag concrete

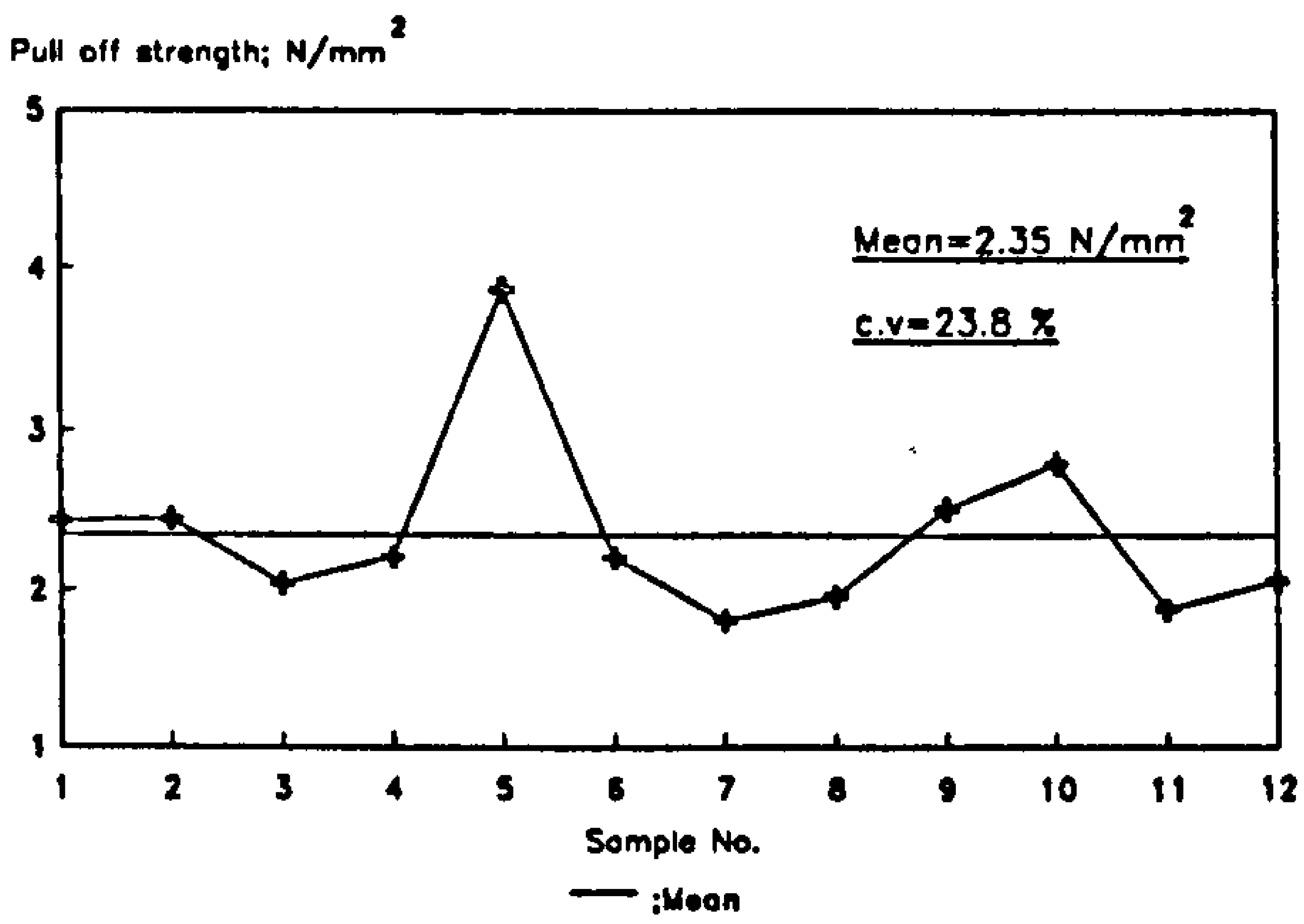


Figure 6.104:Typical variation of surface pull off strengths on Leca concrete

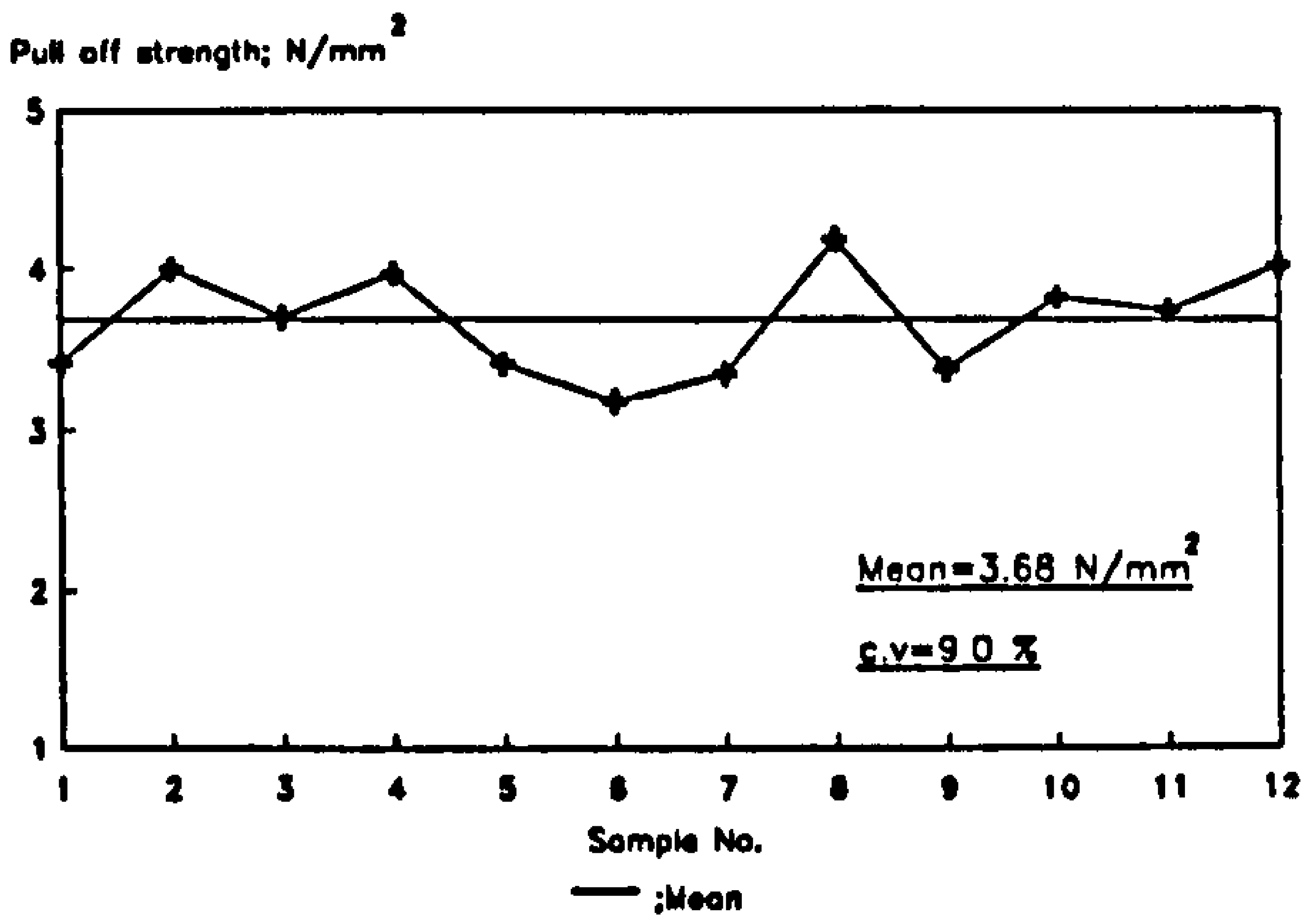


Figure 6.105:Typical variation of surface pull off strengths on Pellite concrete

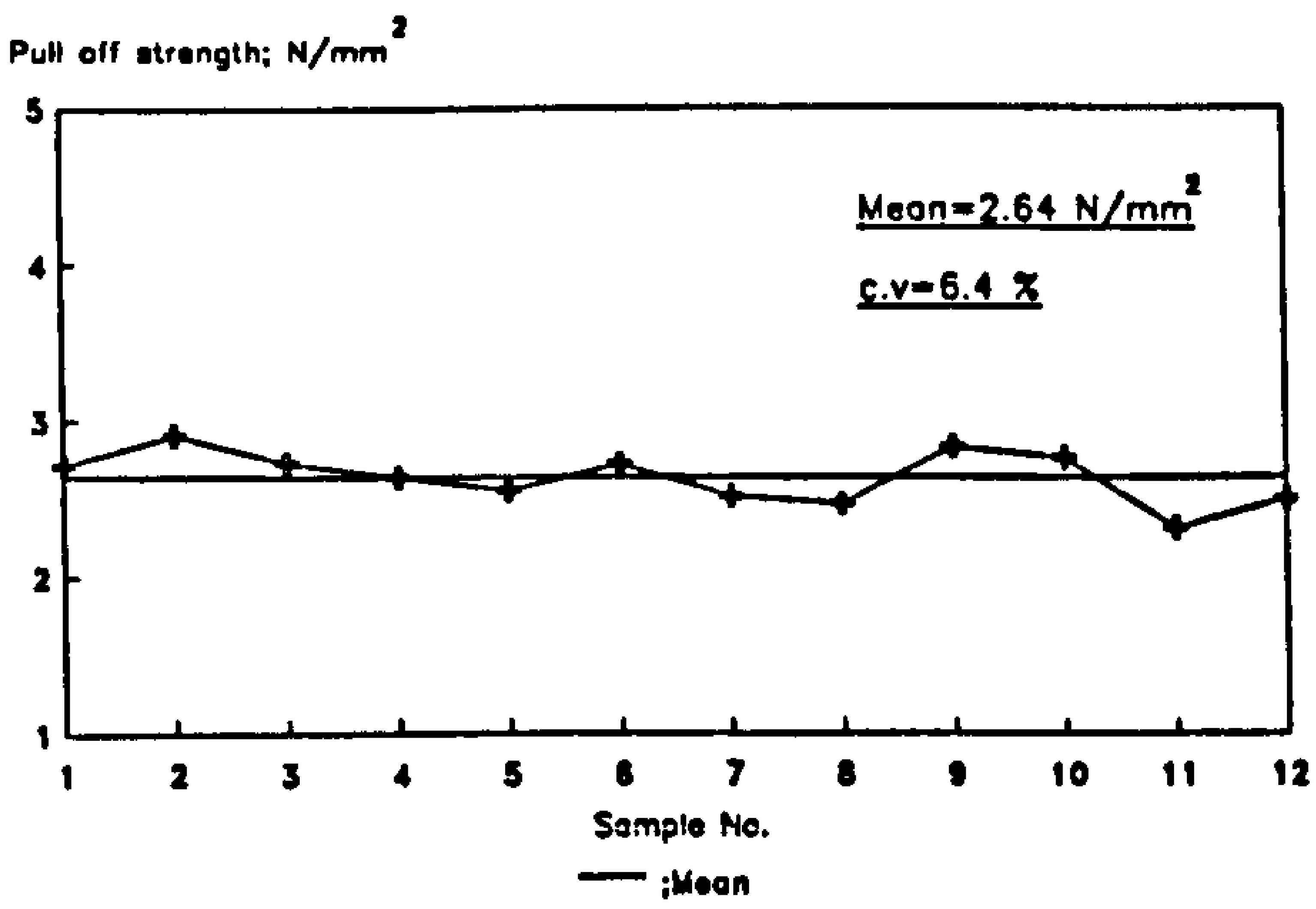


Figure 6.106:Typical variation of partial cored pull off strengths on fully Lytag concrete

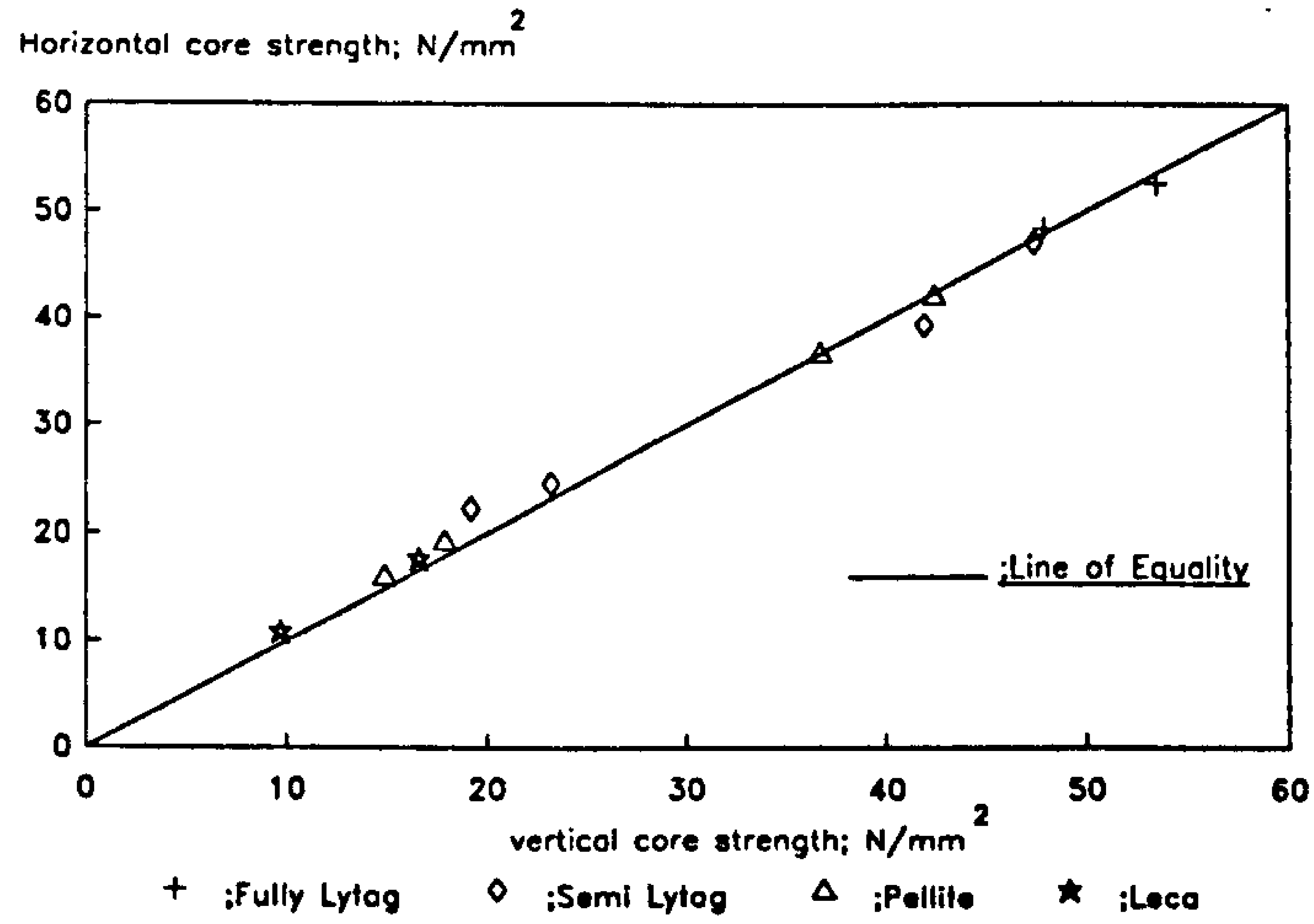


Figure 6.107:relationship between vertical core strength and horizontal core strength

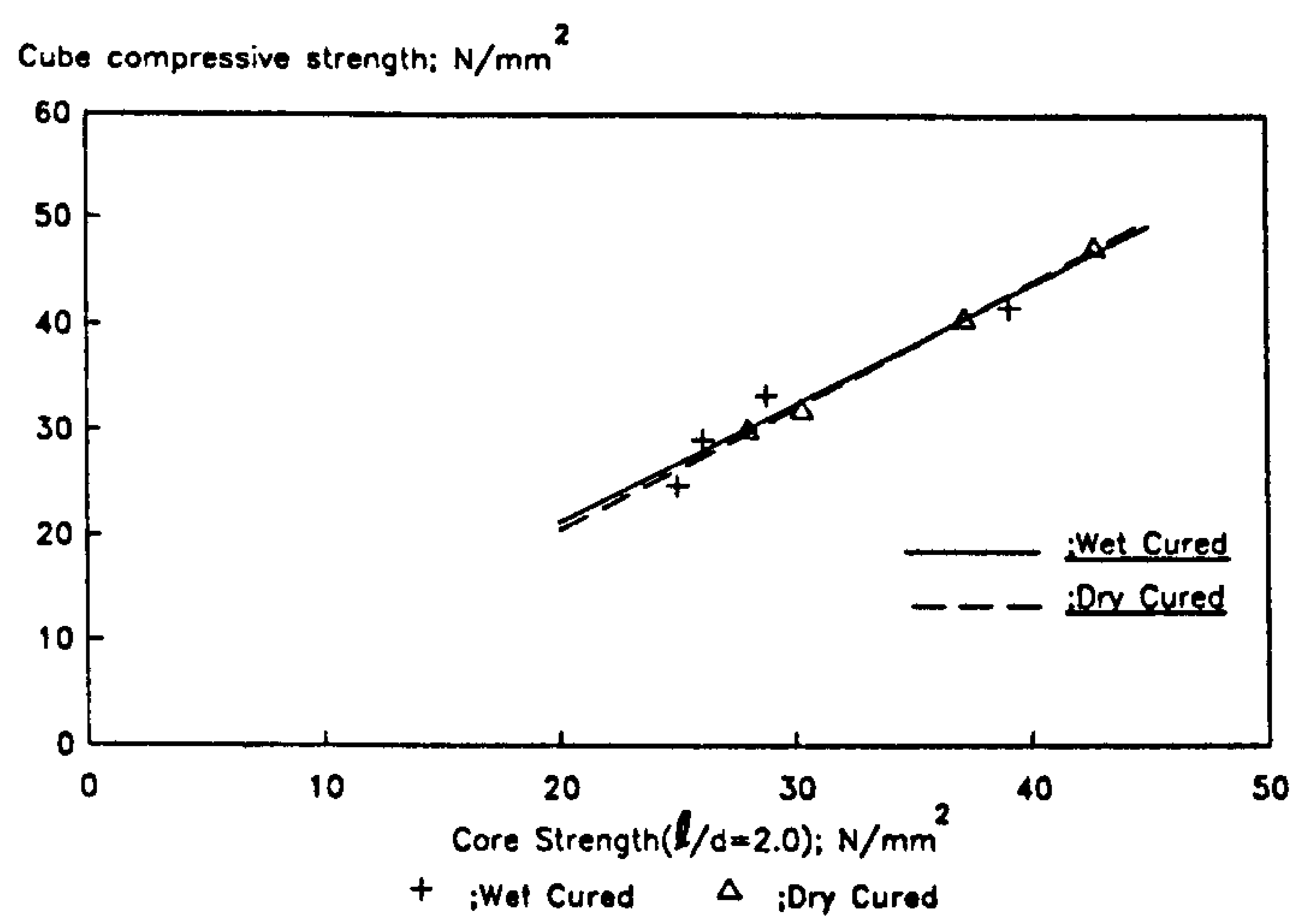


Figure 6.108:Relationship between core strength and cube strength of fully Lytag concrete under different curing conditions

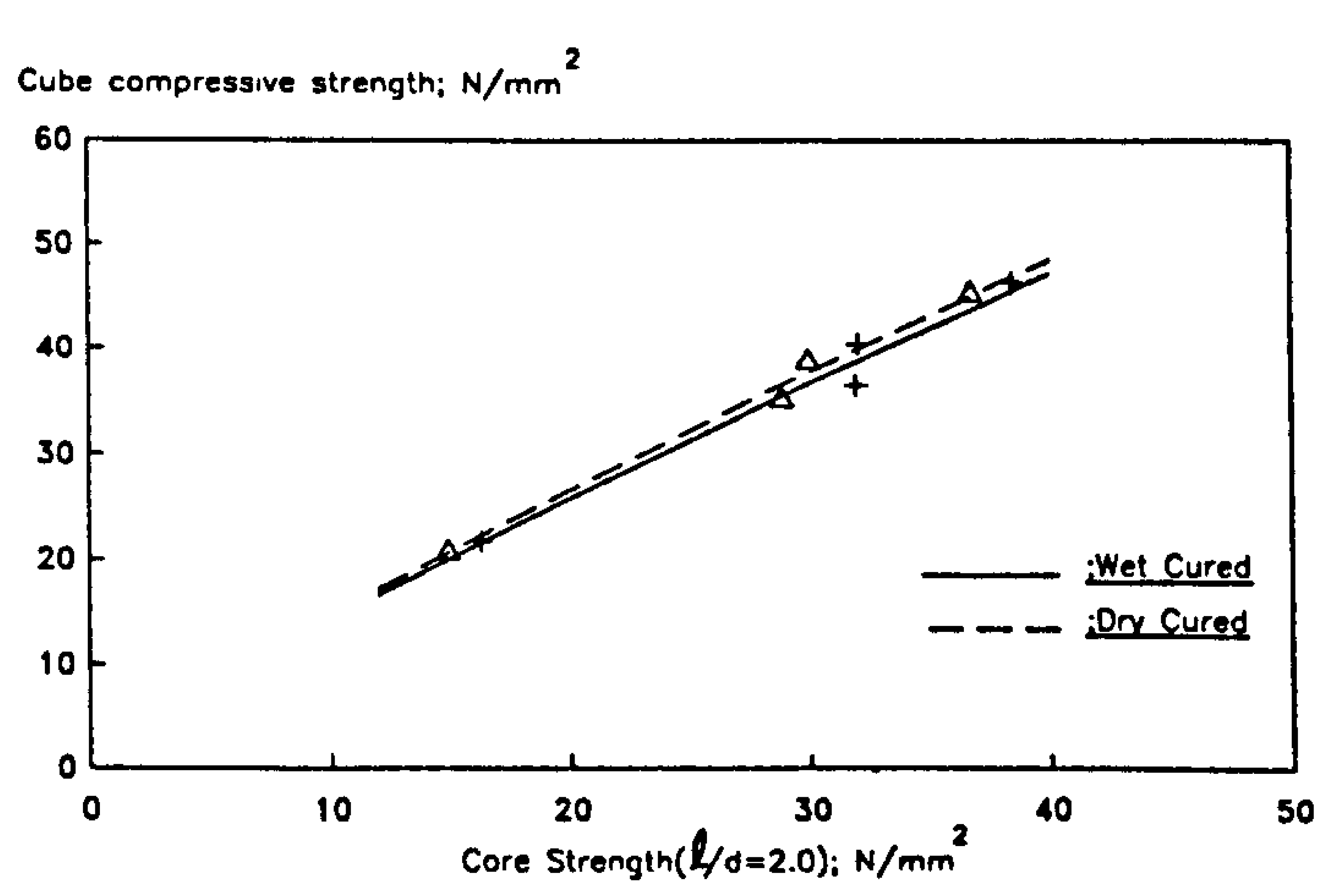


Figure 6.109:Relationship between core strength and cube strength of Pellite concrete under different curing conditions

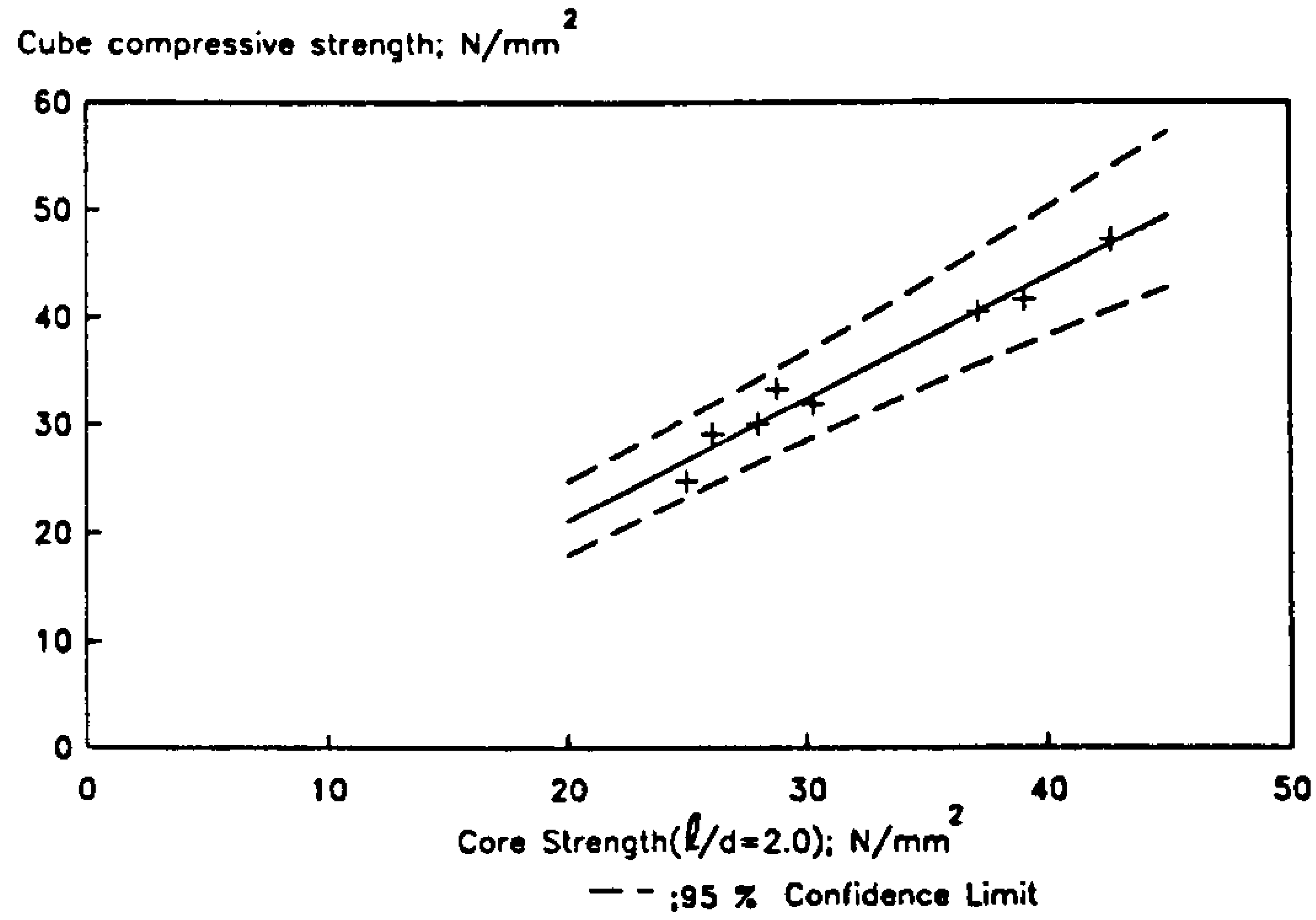


Figure 6.110:Relationship between core and cube strengths of fully Lytag concrete

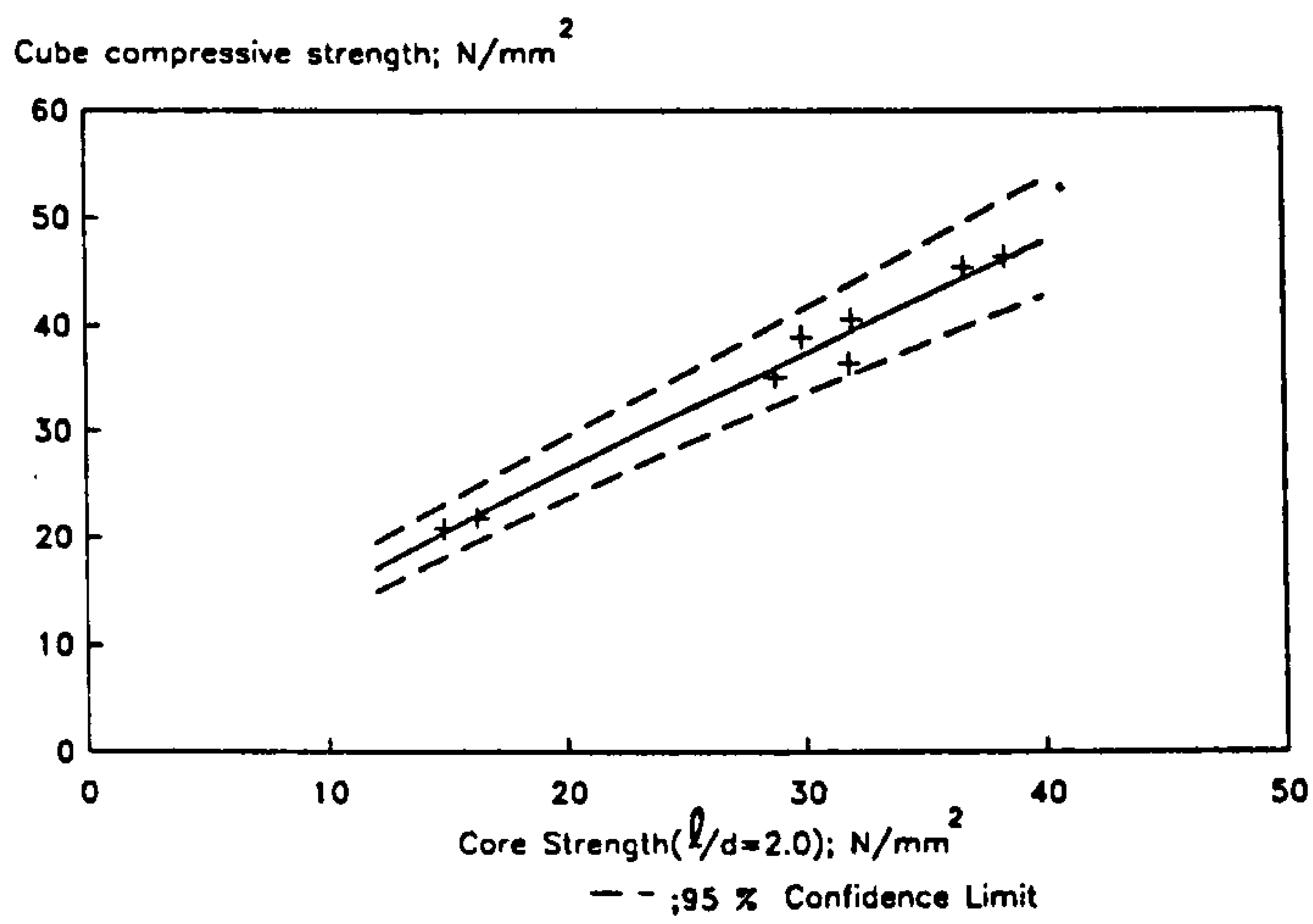


Figure 6.111:Relationship between core and cube strengths of Pellite concrete

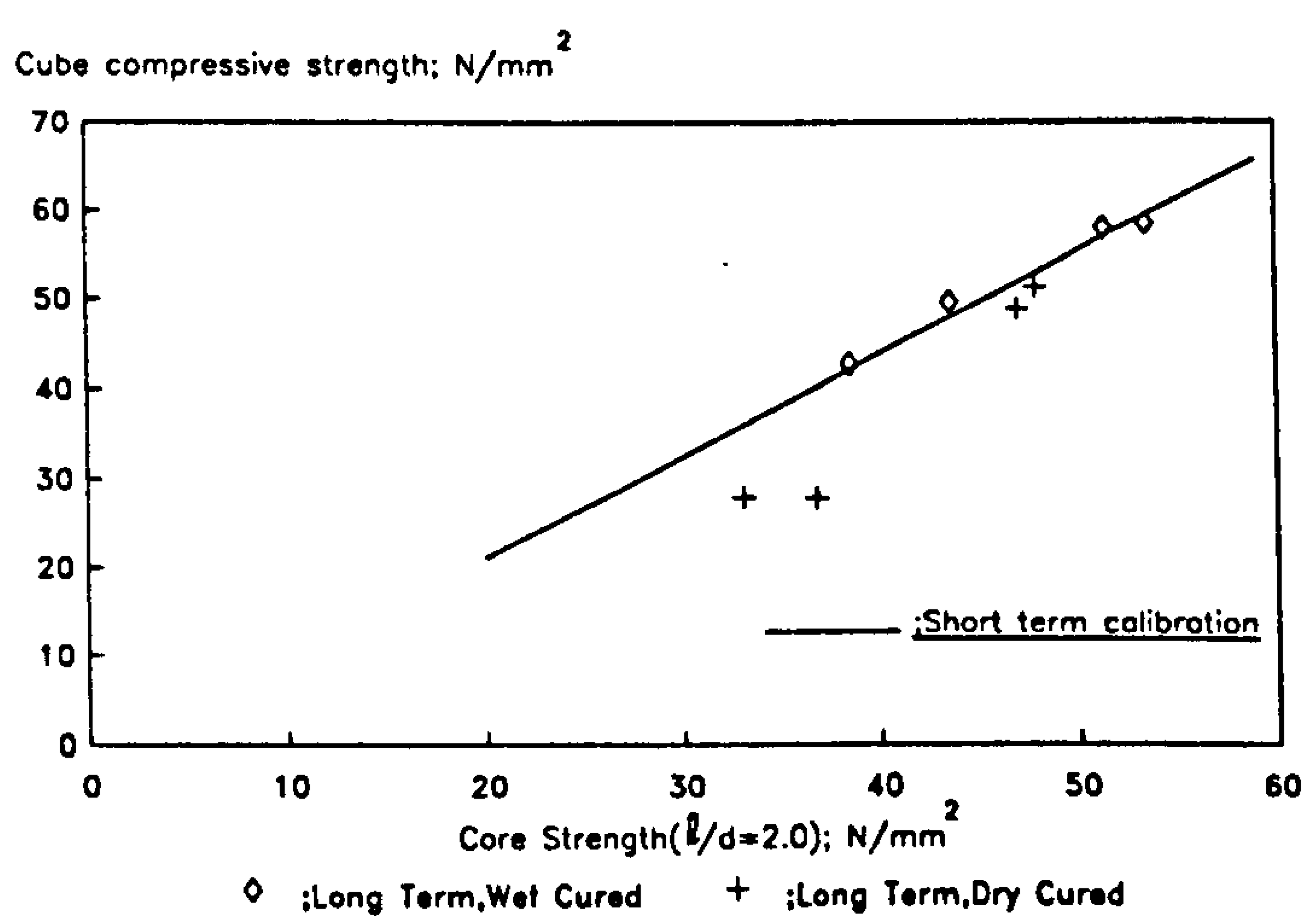


Figure 6.112:Reliabilty of short term core test calibration for assessing long term concrete strength of fully Lytag concrete

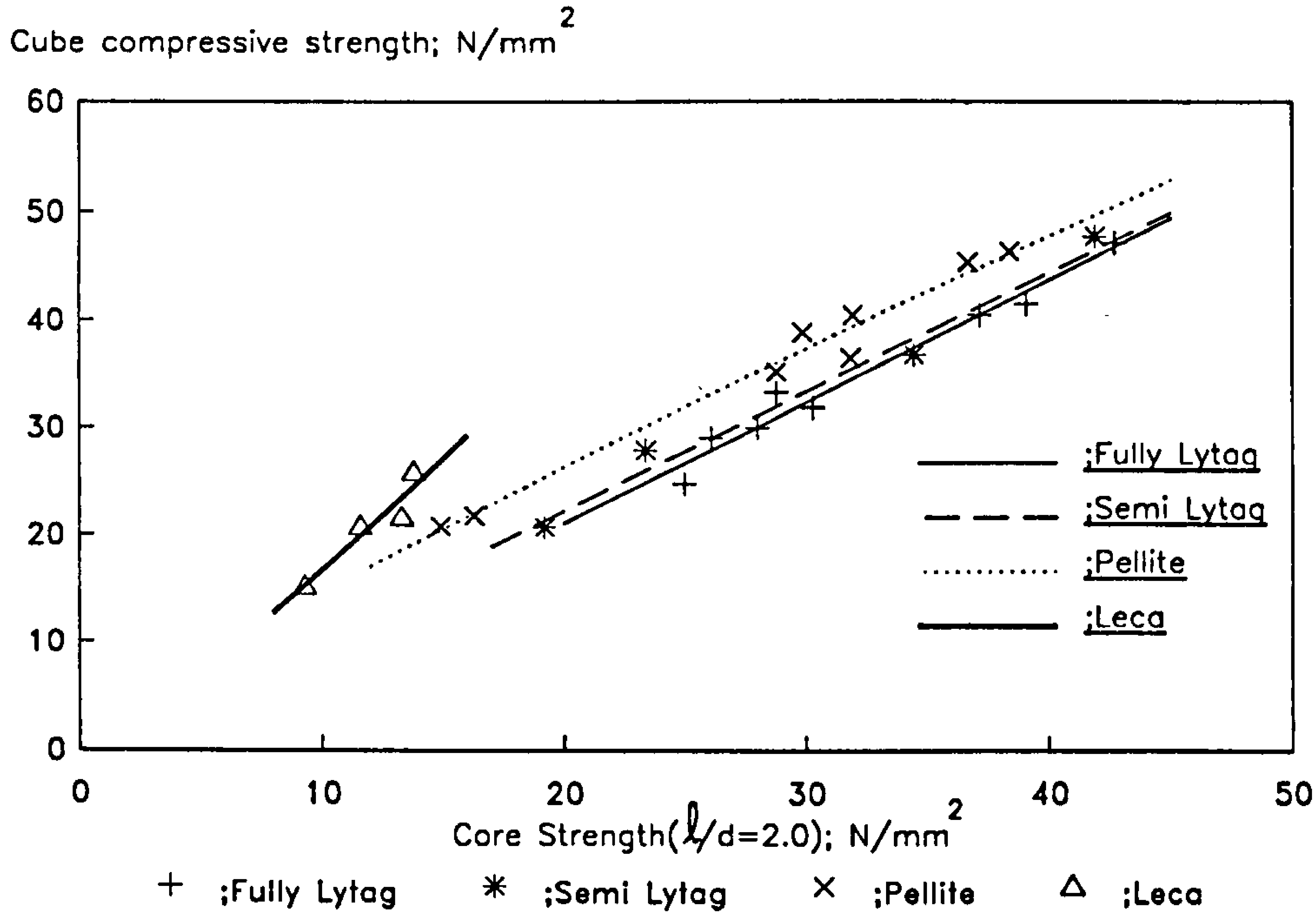


Figure 6.113:Comparison of relationships between core strength and cube strength of various lightweight concretes

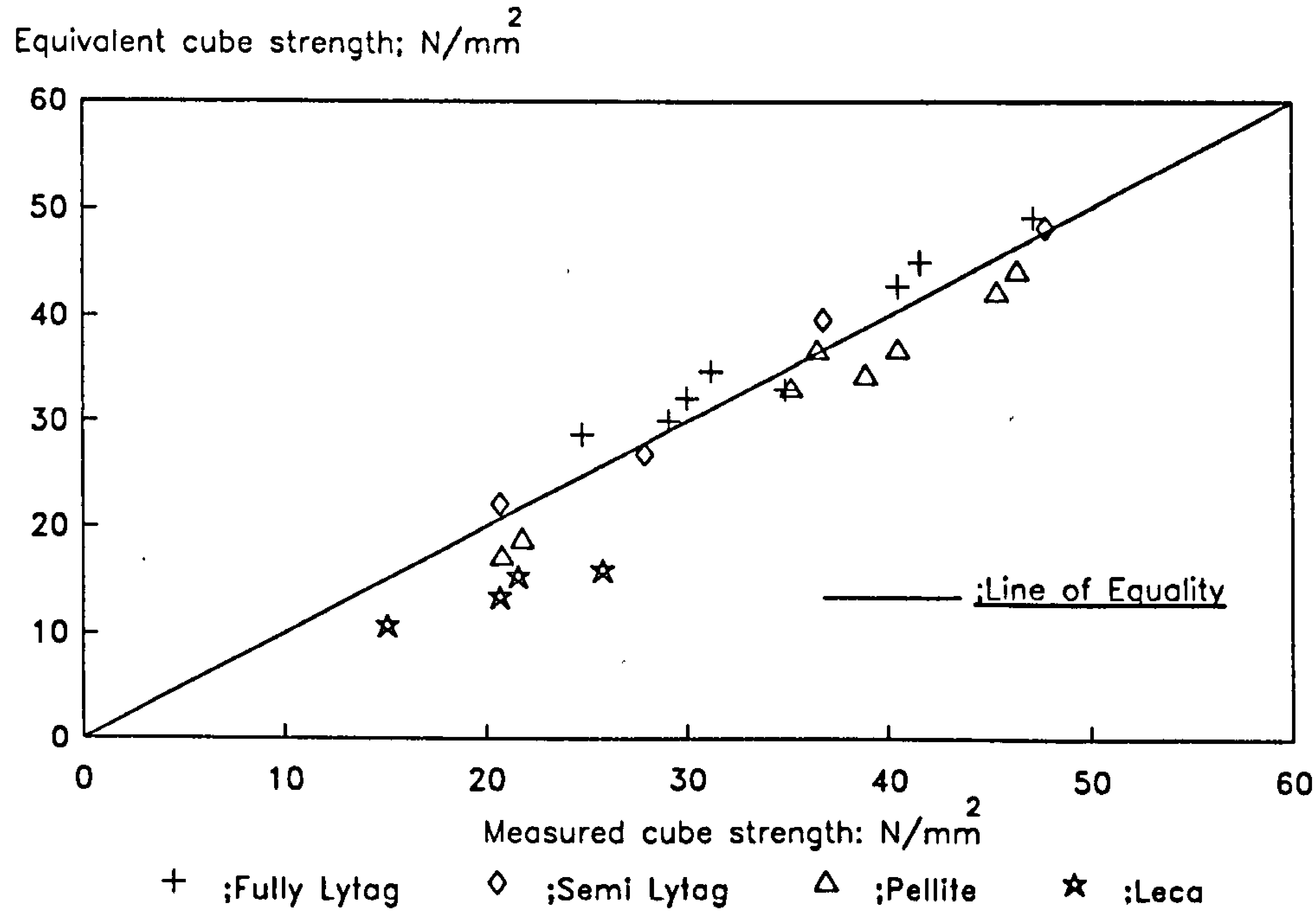


Figure 6.114:Relationship between measured cube strength and equivalent cube strength

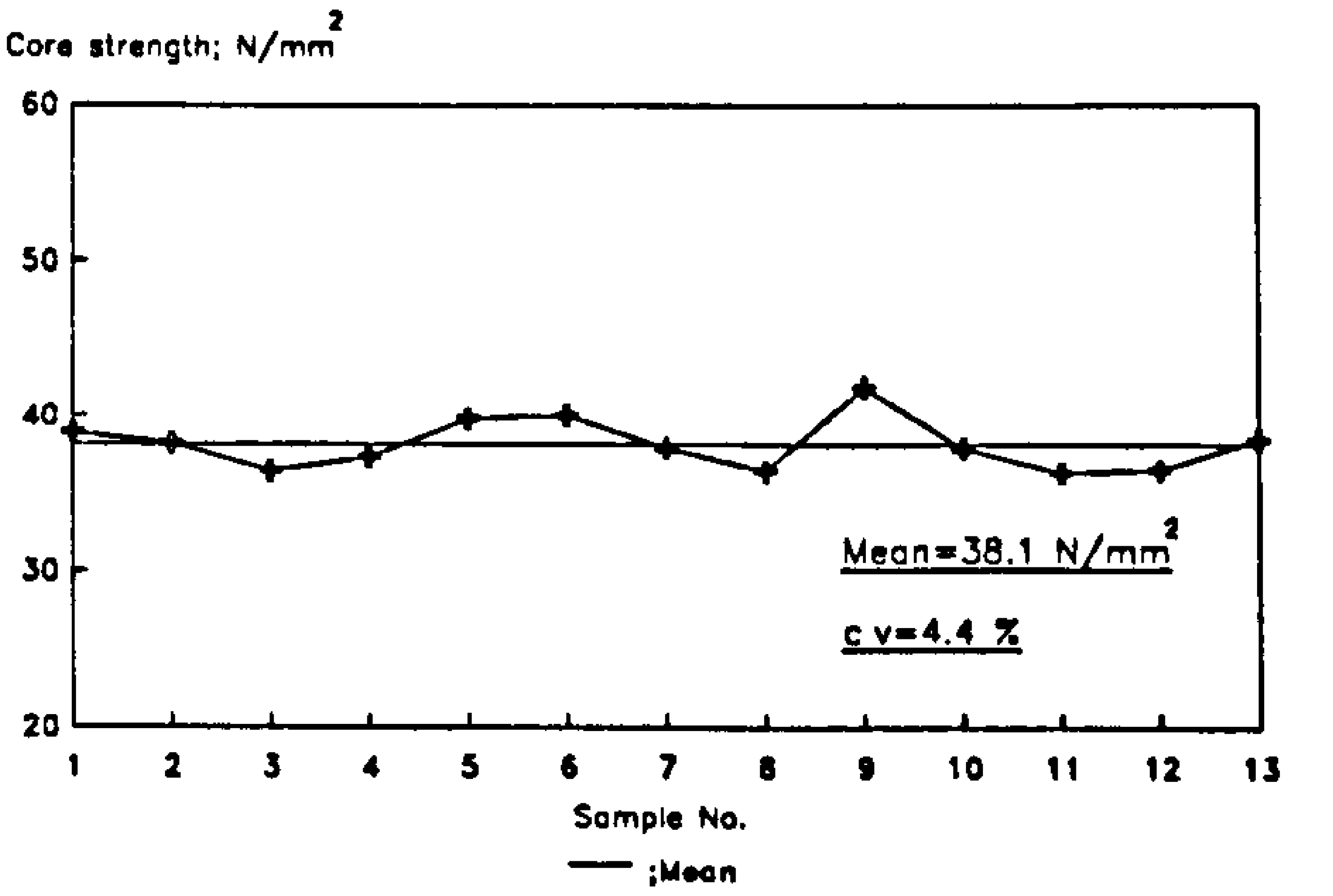


Figure 6.115:Typical variation of wet core strength of fully Lytag concrete with ℓ/d=1.0

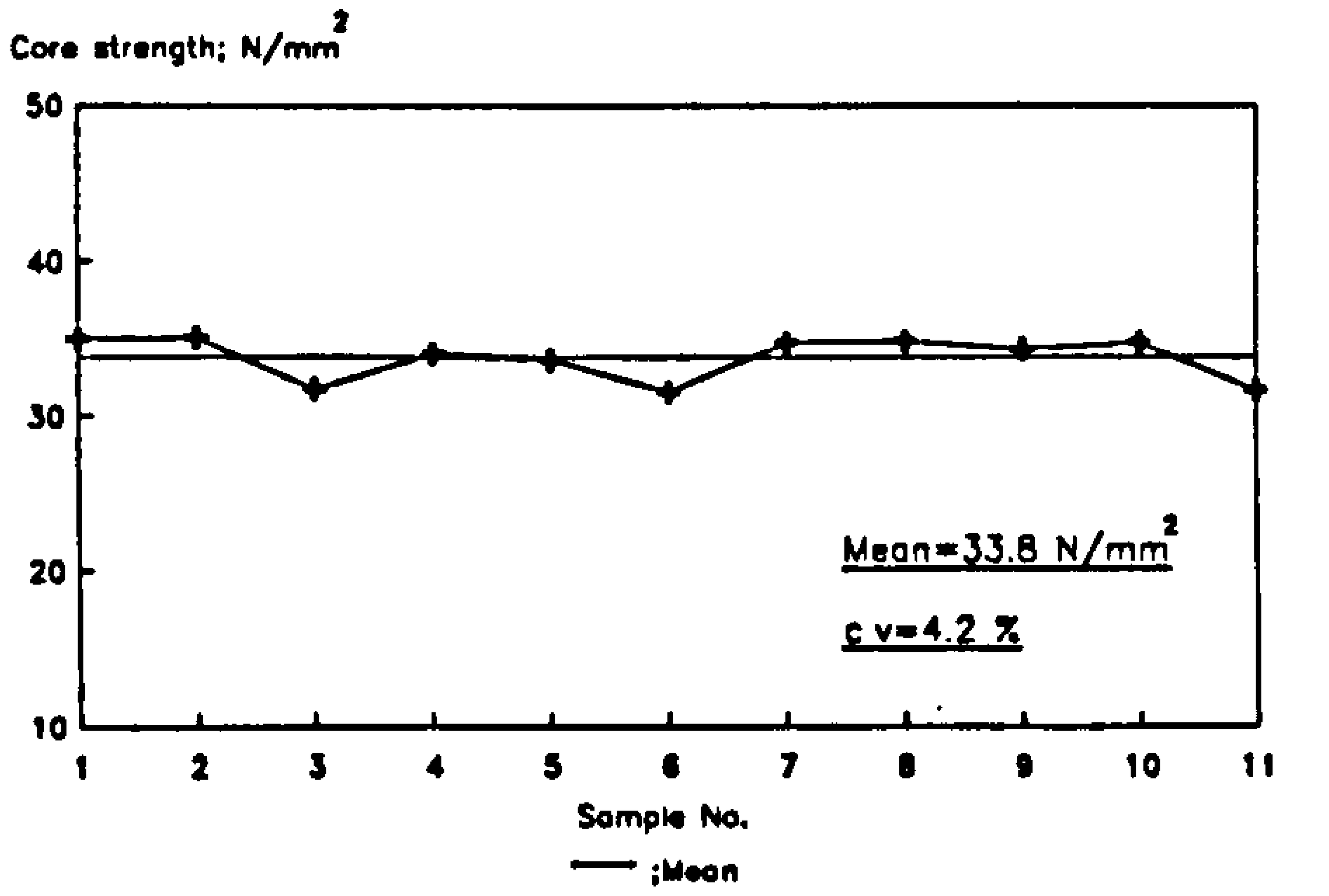


Figure 6.116:Typical variation of wet core strength of fully Lytag concrete with ℓ/d=2.0

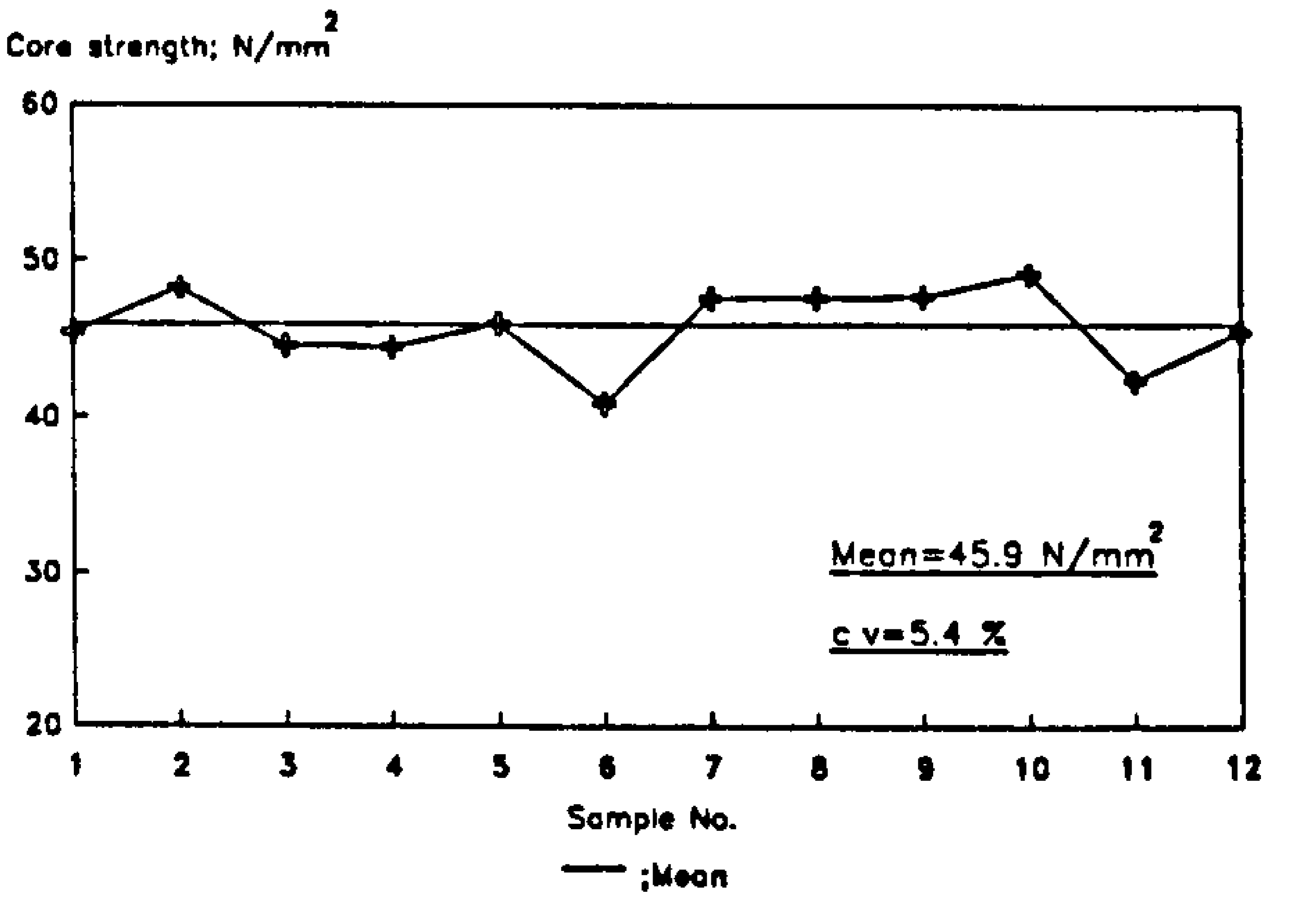


Figure 6.117:Typical variation of dry core strength of fully Lytag concrete with ℓ/d=1.0

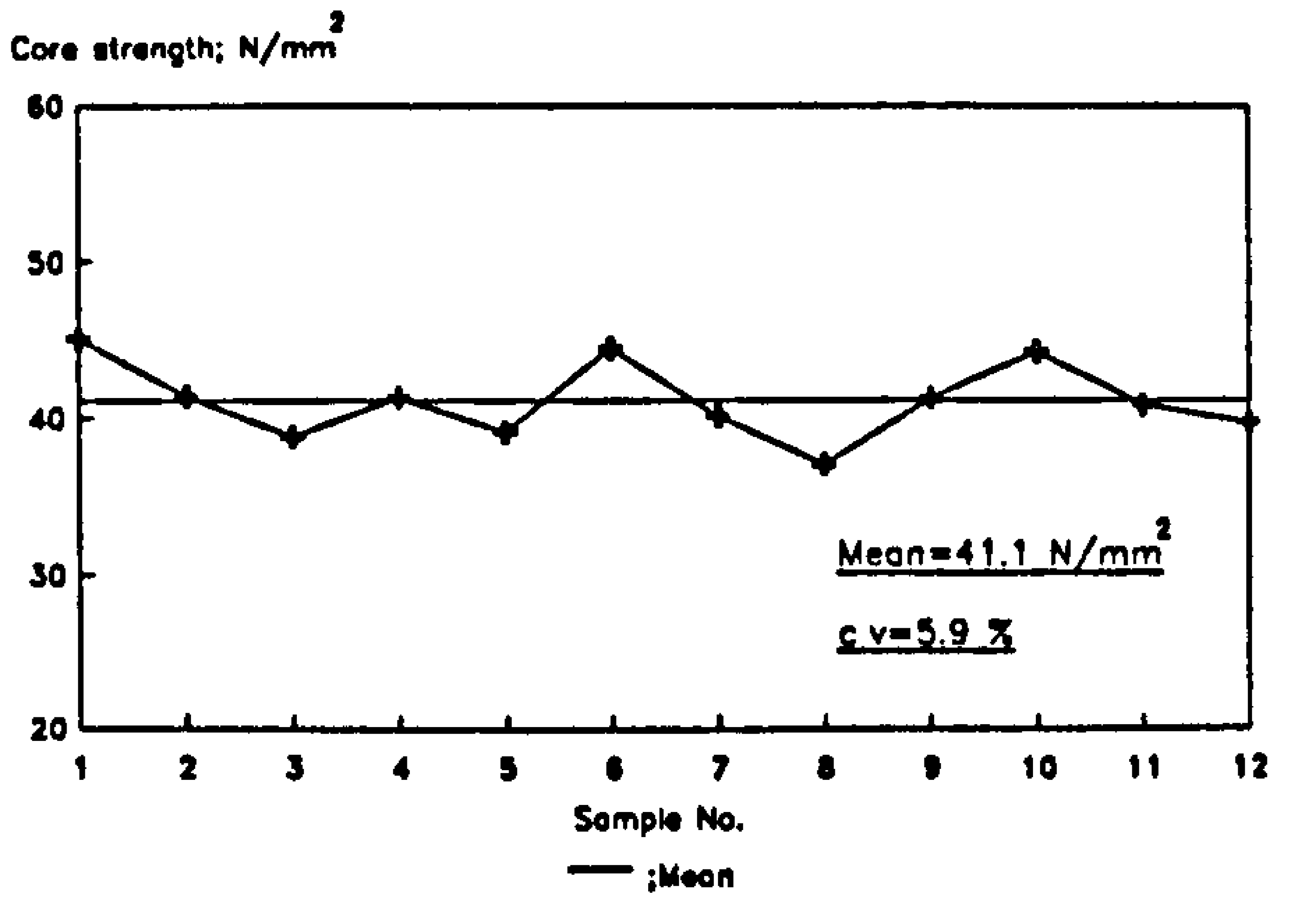


Figure 6.118:Typical variation of dry core strength of fully Lytag concrete with ℓ/d=2.0

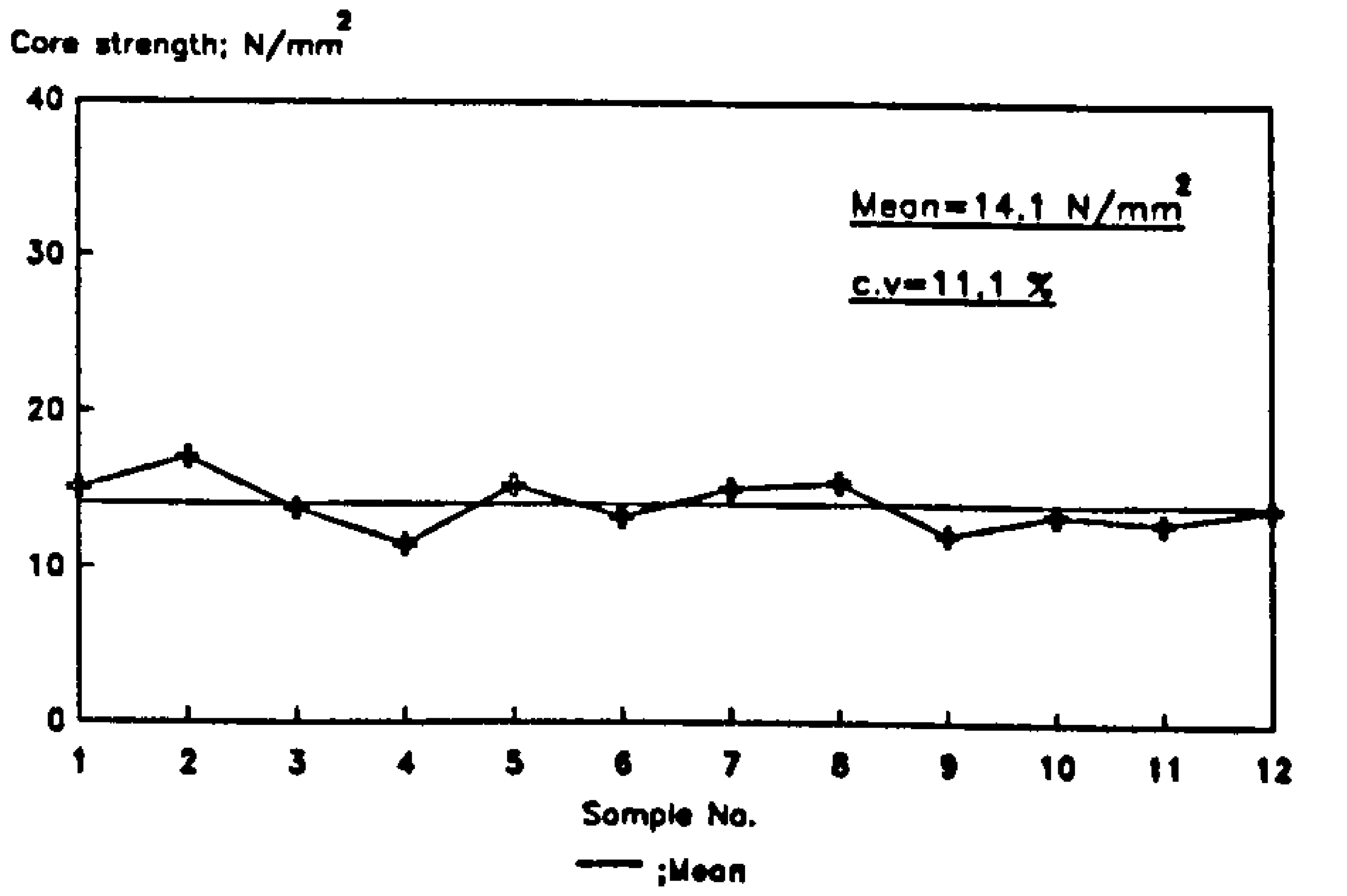


Figure 6.119:Typical variation of wet core strength of Leca concrete with ℓ/d=1.0

CHAPTER 7. FAILURE MECHANISMS OF PARTIALLY DESTRUCTIVE TESTS

7.1 INTRODUCTION

This Chapter presents an investigation of failure mechanisms of a number of partially destructive tests, namely Windsor Probe, pull out, direct pull internal fracture and pull off tests. In this study three different approaches were taken. In the Windsor Probe test, after the completion of tests, specimens were sectioned and examined for internal mode of fracture. Numerical analysis of the stresses for a linear elastic material has been carried out using a finite element system with pull out and pull off tests to find possible explanations for the influence of concrete type on the test measurements. Finally, an analytical solution using basic engineering mechanics has been applied to the direct pull internal fracture test to examine the possible mode of failure.

7.2 WINDSOR PROBE TEST

From a fundamental point of view, the Windsor Probe test is similar to the rebound hammer test, except that the probe impacts the concrete with much higher energy than the plunger of the rebound hammer. A theoretical analysis of this test is complex as it involves a combination of compressive, tensile, shear and friction forces (Bungey, 1989), but the essence of the test involves the initial kinetic energy of the probe and energy absorption by the concrete. The probe penetrates into the concrete to the distance required for the absorption of its initial kinetic energy.

In this study, the internal mode of fracture of fully Lytag concrete was examined after the completion of the test. The concrete element selected for examination was cut with a diamond saw after removing the probe using a wrench. The cut was placed so that it passed through the probe axis. The cut surface was coated with a low viscosity epoxy resin (CXL 600 epoxy resin formulated by Colebrand Ltd.) containing fluorescent dye. When the resin had fully hardened, the surface was ground and polished. The internal mode of fracture was examined by taking a photograph under ultraviolet light, and this may be seen in Figure 7.1a. This figure shows that the resin tended to accumulate in the cracks. Note that the low viscosity resin will also have accumulated in cracks caused by shrinkage as well as in those resulting from the impact of the probe.

On a previous specimen, fractured under identical conditions, it had been attempted to apply the resin to the bolt hole before cutting the concrete, but subsequently observing the cut surface under ultraviolet light showed that no resin had entered the cracks. However, when this cut surface was viewed under normal light, the concrete at the tip of the probe could be seen to be heavily crushed and this is shown in the photograph given in Figure 7.1b.

From the observation of the mode of fracture using the above photographs, it seems that when the probe is fired into the concrete, a stress wave travels through and tends to compress the concrete. This compression is unable to create room for the probe and the material is crushed around the shank. The shock waves associated with the impact will cause triangular fracture lines (Figure 7.1a). These fracture lines are propagated upwards until they reach the surface of the concrete at which time the spall areas become loose and they are displaced. Finally, when the probe is almost home, the stress falls below the level at which cracks will be formed. However enough energy is available to crush the concrete around the probe (Figure 7.1b). Penetration continues and energy is absorbed by the continuous crushing and by surface friction until penetration stops. From the above observation, it may be assumed that most of the holding power of the probe develops around of its tip. The features observed here support the general suggestions made by Kopf (1969).

7.2.1 The Significance of the Failure Mechanism of the Windsor Probe Penetration

In the previously discussed observations, it was shown that concretes made with lightweight aggregate crush more easily than those made with normal weight aggregate. Therefore the manufacturers supply different probes for use with different types of concrete (see Section 3.3.1).

The higher penetration of probes in lightweight concrete as opposed to normal weight concrete may also be related to stiffness of concrete where in lightweight concrete with lower stiffness, a deeper penetration of probes may be expected.

Higher probe penetration in fully lightweight concrete as compared to semi lightweight concrete (see Section 6.3.1.2) is probably related to the lower skin friction between probe and concrete resulting from use of fine lightweight aggregate, in addition to stiffness differences. This highlights the need for strength calibrations taking full account of the nature of both coarse and fine aggregates when using this test method with lightweight concretes.

7.3 PULL OUT TEST

Considerable effort has been directed towards determining what strength property of concrete is actually being measured when the pull out test is applied. Much research work has been previously carried out theoretically and experimentally to study this, including Jensen and Braestrup (1976), Ottosen (1981), Stone and Carino (1983, 1984), Yener and Ting (1984), Krenchel and Shah (1985), Thorpe et. al. (1987), and Hellier et. al. (1987). Among these investigations, it seems that the work in the National Bureau of Standard 'NBS' carried out by Stone and Carino (1983) is relatively important since it represents detailed measurements of internal strains at various stages of pull out behaviour. To facilitate measurements of strains in concrete, they enlarged the scale by 12:1. The main results which they obtained can be summarized as follows:

Three distinct phases of pull out behaviour were observed. 1) circumferential cracking near the upper edge of the disk initiates at 25 to 30% of the ultimate load and ends the elastic response. 2) The circumferential crack continues to propogate towards the reaction ring with an increase in load. The cracking is completed at about 65% of ultimate load. (Similar observation has also been reported by other investigators such as Ottosen using non-linear finite element analysis). 3) The load carrying mechanism beyond the load required to complete circumferential cracking is speculated to be due to aggregate interlock across the failure surface.

The results of the NBS study are in good agreement with other researchers up to the stage where the circumferential crack is completed, but beyond this stage there is some controversy. For instance, Ottosen (1981) states that large compressive forces run from the disk in a narrow band towards the support, and this constitutes the load carrying mechanism. Thorpe et. al. (1987), however, believed that the residual capacity is governed by crushing of the concrete around the support ring and below the juncture of the disk and the stem. It is also possible that scaling effects in the NBS study may have distorted the influence of coarse aggregate particles.

The experimental results obtained by the Author on lightweight concretes seems to support the hypothesis suggested by NBS since significantly lower pulling resistance was found compared to normal weight concrete of comparable strength. For instance, in fully Lytag concrete, the pulling resistance was on average only 70% of that for normal weight concrete. This suggests that, since in lightweight concrete the crack generally passes through the aggregate, the phenomenon of aggregate interlock is considerably reduced. This has been confirmed by other researchers such as Taylor (1970), Hamadi (1976) and

Arasteh (1988). Their investigations involved a range of aggregate types and all concluded that lightweight concretes have considerably less aggregate interlock than dense concretes and that the aggregate interlock action may be neglected in lightweight concretes.

7.3.1 Finite Element Analysis

Investigations have been undertaken by the Author to compare stress fields within normal and lightweight concretes subjected to pull out forces, at the pre-cracking stage. A finite element model was generated used the pre-processing facilities of the computer code ANSYS (Swanson Analysis Systems, 1989). For analysis, a linear-elastic, isotropic axisymmetric solid model was employed. Figure 7.2 shows a plot of a typical element mesh using 150x150mm cylindrical shape. Although this size and shape of specimen is different to that used during the experiments, it is believed that due to the localized nature of failure the size and shape of specimen may not have significant influence on stress fields provided there is sufficient edge distance.

From Figure 7.2, only half of the pull out (Lok-Test) geometry is shown since the model is axisymmetric. The mesh consisted of 383 elements and possessed a total of 435 nodes. These elements are referred to by the name STIF42 Isoparametric Quadrilateral Solid elements in ANSYS. There were four nodes per element having two degrees of freedom at each node; namely translation in the x and y directions.

Since the steel insert disk is typically lubricated before the concrete is cast, it was assumed that no bond exists between the bottom and sides of the insert head and the surrounding concrete by providing a small gap 0.2mm in width. On the other hand, perfect bond was assumed between the top of the steel disk head and the concrete due to the high bearing stresses in this region.

The bolt used for transmitting the load to the disk was not modelled as this was felt to be unnecessary. The applied load was distributed across the upper surface of the disk head in the zone where the bolt would screw into the head.

The surface reaction ring was assumed to be bonded to the concrete and to provide rigid restraint in the vertical and horizontal directions. The nodes along the axis of symmetry are also constrained not to move in the x-direction. These boundary constraints are also indicated in Figure 7.2.

The material properties associated with the 2-dimensional isoparametric solid elements are modulus of elasticity and Poisson's ratio. For lightweight and normal weight concretes, the modulus of elasticity was $17.3 \times 10^3 N/mm^2$ (for semi Lytag concrete) and $26 \times 10^3 N/mm^2$ respectively at a compressive strength of $30 N/mm^2$. The Poisson's ratio was 0.2 for both types of concrete. For the insert, a typical modulus of elasticity and Poisson's ratio of $200 \times 10^3 N/mm^2$ and 0.3 were used respectively.

7.3.1.1 Finite Element Results

The overall precracked state of stress was examined on the basis of principal stresses. In the discussion of the results, a positive stress is considered to be tension. Maximum and minimum principal stresses are presented in algebraic order in the following sections.

To investigate the influence of material properties on pulling resistance, two types of concrete namely semi Lytag and normal weight concretes were considered. For both concretes, the applied load was taken as 1.40kN, or 8.5% and 6% of ultimate load respectively. Stress analysis prior to cracking indicates that for a given strength level, the stiffer concrete (normal weight concrete) with $E = 26kN/mm^2$ achieved only slightly higher pulling resistance (about 5%) than less stiff concrete (semi Lytag concrete) with $E = 17.3kN/mm^2$. Thus the effect of stiffness of concrete is unlikely to be the key factor controlling differences in the ultimate pull out force between normal and lightweight concretes. Further investigation would be required using a non-linear finite element to clarify the effect of concrete stiffness on ultimate pulling resistance.

Circumferential crack initiation seems to be started at an early stage of loading (approximately 8% or 6% of ultimate loading in semi Lytag concrete or normal weight concrete respectively). This is similar to observations reported by Ottosen (1981) and Hellier et. al. (1987). However Stone and Carino (1983), using the large scale pull out experimental model indicated that the cracks initiate at some later stage such as

30% of ultimate load. Also, in their analytical analysis using the finite element method (Stone and Carino, 1984), they have applied 17% of ultimate load to the model without cracking. This difference in crack initiation has been further investigated by the Author by finite element modelling and it was found that different positioning of the applied load influences the crack initiation. In the Lok-Test model, the load is applied at the middle to the top of the disk head, while in the large scale pull out test, the load was applied to the periphery of the underside of the disk. Nevertheless the different positioning of the applied load in the large scale pull out test seems not to have any influence at the later stage of loading where it is shown that circumferential cracks were completed at around 65% of ultimate load which is in agreement with published theoretical results obtained for the Lok-Test.

Figures 7.3 and 7.4 present a typical principal stress distribution for part of the concrete elements using semi Lytag concrete at 8.5% of failure load. Figures 7.3 and 7.4 show the maximum and minimum principal stresses respectively based on average element stress. From Figure 7.3, the maximum principal stress indicates that high tensile stresses in the x-y plane (radial stress) occur in the concrete near the upper corner of the disk and the stress decays rapidly with distance from the disk. It is believed, therefore, that the initial cracking of the concrete would be at or near the upper corner of the disk. From the minimum principal stress distribution plot (Fig. 7.4), it is seen that high compressive stresses in the x-y plane occur just ahead of the upper surface of the disk and also beneath the reaction ring inner edge but with less magnitude. It is also revealed from Figure 7.4 that compressive stresses run from the disk towards the reaction ring as observed by Ottosen (1981) in his non-linear finite element analyses. He indicated that failure is initiated by crushing of the concrete within this compression band. However compressive strain measurements along the failure surface in the large scale pull out test (Stone and Carino, 1983) were shown to be insufficient to result in a compressive failure.

The existence of circumferential stresses was also observed at the precracking stage. Unlike the tensile principal stresses in the x-y plane, the tensile circumferential stresses are greatest at the intersection of the top concrete surface and the insert stem and can be seen in Figure 7.3. These tensile stresses cause the formation of radial cracks in the failure cone as observed during the experimental investigation.

7.3.2 The Significance of the Theoretical Study on Pull Out Resistance

The limited number of cases investigated on lightweight and normal weight concretes may be sufficient to confirm that, at the pre-cracking stage, similar behaviour may be expected for different types of concrete (i.e. lightweight and normal weight concretes) at each strength level. Hence the significant reduction in pull out resistance of lightweight concrete as compared to normal weight concrete is probably related to the stage after completion of crack formation. At this stage, the increase in pull out resistance of normal weight concrete seems to be controlled by aggregate interlock. In lightweight concrete, the absence or limited degree of effective aggregate interlock, as confirmed from previous investigations, would not be expected to significantly increase the pull out resistance beyond the stage of crack formation. This is supported by the significant calibration differences between lightweight and normal weight concretes illustrated in Figure 6.46. To fully confirm this mechanism, further analysis is required at the post-cracking stage taking into account of the influence of varying aggregate interlock effects.

7.4 DIRECT PULL INTERNAL FRACTURE

The experimental investigations discussed in Section 6.3.3.2., indicate tensile concrete failure with the direct pull internal fracture test. This mode of failure was previously confirmed analytically using the theory of plasticity by Salman (1979). A similar mode of failure has now also been demonstrated analytically with an alternative simple approach using basic engineering mechanics. This provides a better prediction of failure load than that obtained previously.

7.4.1 Derivation of Analytical Study

Consider the embedded wedge anchor bolt shown in Figure 3.8. The ultimate pull out force is normally governed by pull out of a portion of the concrete along a roughly conical failure surface. The failure zone has been assumed to be a truncated cone with a depth measured to be 17mm on average. For the purposes of analysis, the problem was treated in the two-dimensional case by taking an idealized element A within a sector (Figure 7.5a) with a unit width at the interface between the wedge anchor bolt and the concrete which is assumed to be homogenous and isotropic.

With reference to Figure 7.5(a), when the pull out force is applied, this would produce a radial expansion pressure and shear stress whose integration give the expansion force N and pull out force P in the region of engagement of the expanded portion (clip) with concrete. The state of stress in the concrete adjacent to the clip is given in Figure 7.5(b), and with it the principal stress on concrete (Figure 7.5(c)) can be deduced by means of Mohrs circle analysis (Figure 7.5(d)). The principal stresses from Figures 7.5(c and d) are:

$$\sigma_1 = -\frac{\sigma_x}{2} + \sqrt{\left(\frac{\sigma_x}{2}\right)^2 + \tau^2{}_{xy}} \tag{7.1}$$

$$\sigma_2 = -\frac{\sigma_x}{2} - \sqrt{\left(\frac{\sigma_x}{2}\right)^2 + \tau^2{}_{xy}} \tag{7.2}$$

which σ_1 (maximum) is positive as being tension and σ_2 (minimum) is negative as being compression, and the direction of principal stresses is given by

$$\tan 2\theta = \frac{2\,\tau_{xy}}{\sigma_x} \tag{7.3}$$

Using Equation 7.3, the tensile principal stress can be defined in respect to σ_x and θ as

$$\sigma_1 = -\frac{\sigma_x}{2} + \sqrt{\left(\frac{\sigma_x}{2}\right)^2 + \left(\frac{\sigma_x}{2}\right)^2 \tan^2 2\theta}$$

This equation can be simplified using trigonometric identities to

$$\sigma_1 = \sigma_x \frac{\sin^2\theta}{\cos 2\theta} \tag{7.4}$$

If failure is assumed to be tensile a tensile crack is initiated at right angles to the direction of the tensile principal stress, that is along the line of the compressive principal stress. Hence the planes of failure and of principal stress will coincide. The combination of N and P produces a resultant Q which acts at an angle α, say. This force along with other forces such as tensile force T acting at right angles to the failure surface, and force H corresponding to reaction through the hinge which is assumed to occur at point C are shown in Figure 7.6. These three forces, Q, T, and H, are in equilibrium and Q is most easily determined if H is eliminated by taking moments about C.

$$Q\ell_c \cos(\theta + \alpha) = T\,\bar{\bar{x}} \tag{7.5}$$

First of all it is necessary to know T. In order that a tensile crack shall propogate, the tensile stress at right angles to the failure surface must be equal to the tensile strength of concrete, f_t, in the vicinity of A, i.e.

$$\sigma_1 = f_t \tag{7.6}$$

Further from A, the tensile stress will probably be less than f_t, and at C it will be zero. However between A and C an exact distribution of tensile stress is not known, but for simplicity it may be possible to assume a linear distribution, i.e.

$$\sigma_n = f_t \; (x/\ell_c) \left| \begin{array}{l} x = 0 \rightarrow \sigma_n = 0 \\ x = \ell_c \rightarrow \sigma_n = f_t \end{array} \right.$$

In Equation 7.5, T can be replaced in terms of f_t, using the above equation and integrating over the length ℓ_c

$$Q\ell_c \cos(\theta+\alpha) = \int_0^{\ell_c} \left[f_t \left(\frac{x}{\ell_c} \right) S' \; dx \right] x \tag{7.7}$$

S' can be determined from Figure 7.7 as:

$$S' = \frac{(S - 1)(\ell_c - x)}{\ell_c} + 1 \tag{7.8}$$

where from Figure 7.7

$$S = \frac{3 + \ell_c \cos\theta}{3}$$

Substituting Equation 7.8 into 7.7, would lead to

$$Q\ \ell_c\ \cos(\theta+\alpha) = \frac{f_t}{\ell_c} \int_0^{\ell_c} x^2 \left[\frac{\left(\frac{3 + \ell_c \cos\theta}{3} - 1\right)(\ell_c - x)}{\ell_c} + 1 \right] dx$$

which can be simplified as:

$$Q \cos(\theta+\alpha) = \frac{f_t\ \ell_c}{3} \left(\frac{\ell_c}{12} \cos\theta + 1\right)$$

From Figure 7.6, since $Q = P/\cos\alpha$ and $\ell_c = D/\sin\theta$ this reduces to:

$$P = \frac{f_t\ D}{3}\ \frac{\cos\alpha}{\sin\theta\ \cos(\theta+\alpha)} \left(\frac{D \cos\theta}{12 \sin\theta} + 1\right) \qquad (7.9)$$

The correct solution for P would be the one corresponding to a minimum of the trigonometric function in Equation 7.9 by taking account of the fact that α and θ are related to each other. From Figure 7.6

$$\cot\alpha = \frac{P}{N} = \frac{\tau_{xy}}{\sigma_x}$$

but from Equation 7.3, the right hand side of the above equation is equal to ½ $\tan 2\theta$.

Hence

$$\cot\alpha = \tfrac{1}{2} \tan 2\theta \qquad (7.10)$$

Recalling the test mechanism, a lateral force is first exerted to maintain the expanding wedge anchor bolt in place in the hole, and then the pulling force is exerted until failure takes place. Thus σ_x is an invariable quantity in the test. An attempt was made to set up an expression for P that involves σ_x and θ.

Substituting Equation 7.6 into 7.4 and the resulting equation into Equation 7.9 would lead to

$$P = \frac{\sigma_x\ D}{36} \ \frac{\cos\alpha\ \sin^2\theta}{\sin\theta\ \cos(\theta+\alpha)\ \cos 2\theta} \left[\frac{D\ \cos\theta + 12\ \sin\theta}{\sin\theta}\right] \qquad (7.11)$$

α can be converted to θ using Equation 7.10. Subsequently Equation 7.11 can be rewritten using trigonometric identities:

$$P = \frac{\sigma_x\ D}{36} \ \frac{\cos\theta\ (D\ \cos\theta + 12\ \sin\theta)}{\cos 2\theta\ \sin^2\theta} \qquad (7.12)$$

In order to obtain the minimum P required for failure to be occurred, the derivative of Equation 7.12 with respect to θ must be set to zero:

$$\frac{\partial P}{\partial \theta} = 0$$

That is:

$$[-\sin\theta\ (D\cos\theta + 12\sin\theta) + (-D\sin\theta + 12\cos\theta)\cos\theta]\cos 2\theta\ \sin^2\theta$$
$$-[(-2\sin 2\theta\ \sin^2\theta + 2\sin\theta\cos\theta\cos 2\theta)(D\cos^2\theta + 12\sin\theta\cos\theta] = 0$$

This can be simplified using trigonometric identities to:

$$-\sin 2\theta\ \cos 2\theta(D + 6\sin 2\theta) + 2\sin^2\theta\ (6 + D\sin 2\theta\ \cos^2\theta) = 0$$

By trial and error, θ can be solved by knowing D = 17mm. This was found to give $\theta = 31°$.

The predicted of angle of failure, 31° is reasonably close to the corresponding measured angle as shown in Table 7.1 found in this investigation and elsewhere. This therefore supports the concept of tensile failure.

The success of the tensile theory in predicting the angle of failure raises the hope for predicting the pull out force. Recalling that Equation 7.9 gives the pull out force per unit length of expanding clip. The total pull out force may be determined from:

$$F = P(2\pi r) \tag{7.13}$$

where r is the radius of expanded clip.

Substituting Equation 7.9 into the Equation 7.13, the following expression for F can be found

$$F = \frac{2\pi r D\ f_t}{3} \frac{\cos\alpha}{\sin\theta\ \cos(\theta+\alpha)} \left[\frac{D\ \cos\theta}{12\ \sin\theta} + 1\right] \qquad (7.14)$$

knowing D = 17mm, r = 3mm, θ = 31° and α = 46.8° (from Equation 7.10), the pull out force can be evaluated from the linear relationship

$$F = 2.26\ f_t \qquad (7.15)$$

where F is in kN and f_t in N/mm².

A similar linear relationship was also obtained by Salman as:

$$F = 2.54\ f_t \qquad (7.16)$$

using the theory of plasticity where the slope of the line is 12% higher than that obtained from this investigation.

Pull out force can be expressed in terms of compressive strength using the correlations between tensile and compressive strengths shown in Table 5.2 for various types of lightweight concrete. The pull out force/compressive strength relationships based on using these with Equation 7.15 are presented in Table 7.2. Figure 7.8 illustrates the relations between pull out force and compressive strength obtained from theoretical and experimental investigations on a typical lightweight concrete such as semi Lytag concrete. From this figure, the theoretical solution developed above, assuming a simple tensile separation, indicates a very good agreement with the experimental results. At the strength level of $30N/mm^2$, the predicted pull out force from theoretical analysis overestimates the measured pull out force from experimental analysis by only 1.5%. Similar patterns were also found for other types of lightweight concrete.

The theoretical solution obtained by Salman (1979) using the theory of plasticity is also shown in Figure 7.8. f_t, tensile strength, in Equation 7.16 was converted to f_c, compressive strength using the tensile/compressive strength relationship for semi Lytag concrete shown in Table 5.2. It appears that using the theory of plasticity would yield a relatively larger overestimate of measured pull out force from experimental analysis. The overestimation was found to be about 15% at strength level of $30N/mm^2$. This higher deviation may possibly be related to reduced validity of the theory of plasticity to a brittle material, especially when tensile stresses are present (Stone, Carino, 1983).

7.4.2 The Significance of the Theoretical Study on Direct Pull Internal Fracture Resistance

Using basic engineering mechanics, the Author has shown that the state of stress on the assumed failure surface would result in a tensile separation failure. This leads on to the fact that the pull out force/compressive strength ratio for different types of concrete (i.e. lightweight and normal weight concretes) is mainly related to the tensile strength of concrete. This has been confirmed experimentally throughout this research. It must be borne in mind that the theoretical work presented here considers concrete as a homogenous material and does not take into account the characteristics of aggregate itself. It has been demonstrated previously in Section 6.3.3.1 that for some types of aggregate such as Leca this assumed mechanism of failure cannot be developed. Hence the theoretical prediction must be used in conjunction with a knowledge of aggregate characteristics.

7.5 PULL OFF TEST

Experimental investigations carried out on surface and partial cored pull off tests (see Section 6.3.4) proved that the pull off strength for a given compressive strength varies from one type of concrete to another. The aim of the present investigation is to attain clearer insight into the behaviour of various types of concrete under pull off forces. The investigation has also considered some other relevant factors including size of concrete specimen and disk material type and size. A theoretical analysis of stresses for a linear, isotropic axisymmetric solid model using the finite element system, ANSYS as described in Section 7.3.1 was used. A finite element mesh used for the surface pull off test based on a 150x150mm cylinder specimen and 50mm diameter x 20mm thick disk is shown in Figure 7.9 with half of the pull off test geometry. The mesh consisted of 304 elements and possessed a total of 329 nodes. Similar meshes were also used for other purposes by either removing or adding elements to the original mesh shown in Figure 7.9. The concrete and disk elements used in the analysis were based on STIF42 as used in Section 7.3.1. Material properties (modulus of elasticity, E, and Poisson's ratio, ν) for concretes used in the analysis are given in Table 7.3 whilst for aluminium disks $E = 7\ 3\times10^3 N/mm^2$ and ν=0.33, and for steel disks $E = 200\times10^3 N/mm^2$ and $\nu = 0.3$ were assumed.

The adhesive bond layer between disk and concrete was simulated using a spring element. This is because the very thin layer of glue would require a large number of elements which makes it impractical to model. The spring element referred as STIF14 in ANSYS was used along horizontal and vertical directions. The lack of information about glue properties as well as the unknown rigidity of glue along horizontal and vertical directions resulted in difficulty in precise allocation of the stiffness of spring. A value of $1\times10^6 N/mm$ was used for the stiffness of spring along the vertical direction based on assuming a modulus of elasticity of glue, $E = 7000 N/mm^2$, and Poisson's ratio, $\nu = 0.3$ with the thickness of glue taken as 0.02mm. The corresponding value for spring stiffness along x direction was taken to be $1\times10^5 N/mm$ as it is likely to acquire less rigidity along this direction. No significant change in results was observed in the stress analysis for changing the spring stiffness in both directions between $K=1\times10^5$ to $1\times10^{10} N/mm$, however.

The applied load was uniformly distributed at the top face of disk in the region where the bolt (transmitting the force to the disk) is connected to the disk.

The support ring was modelled by providing restraint against vertical and horizontal directions. The boundary constraints are shown in Figure 7.9.

7.5.1 Results from Finite Element Analysis

The applied load in the model was varied, but was restricted to a maximum value of 50% of the expected failure load, to take into account the possible nonlinear behaviour of concrete in tension as described by Raphael (1984). Relative pull off stresses were considered based on concrete type, test type (whether surface or partial cored pull off tests), and disk type. Assuming that the corresponding relative pull off stress at the failure stage is similar to that in the linear zone, then the corresponding relative pull off stress in the linear zone may be compared with that obtained in the experiment at the failure stage.

The maximum principal stresses were considered in the study based on average element stress. They are presented algebrically according to the sign convention outlined previously in Section 7.3.1.1.

7.5.1.1 Surface Pull Off Tests

Stress distributions within different types of concrete are shown in Figures 7.10 to 7.14 and the stress elements are tabulated in Appendix C (Table C-1 to C-5). Similar load (taken as 50% of ultimate pull off force on normal weight concrete as 2.47kN at cube strength of 25N/mm^2) was applied in all cases. The material properties of each type of concrete were taken at a cube strength of 25N/mm^2. The linear elastic stress analysis confirms the dependency of pull off resistance on the type of concrete with lower pull off resistance for normal weight concrete. From the theoretical analysis, it is suggested that the rigidity of concrete is the key factor in pulling resistance. Considering the extreme cases, in normal weight concrete with relatively high rigidity, a high localised stress was created in the concrete beneath the axis of loading (Figure 7.10) whereas in Leca concrete with relatively low rigidity uniform stress exists in concrete along the interface (Figure 7.14). This phenomenon may explain the lower pull off resistance for normal weight concrete. Table 7.4 compares the theoretical and experimental ratios of the pull off stress of lightweight concretes with respect to normal weight concrete at the 25N/mm^2 strength level. The ranking is the same and reasonable agreement exists between the corresponding ratios with a maximum 9% difference in Leca concrete. This might be due to the greater effect of excess glue around the disk which has not been considered in the theoretical analysis. Another observation which can be seen from the above figures is that in lightweight concretes larger stresses are created adjacent to the edges of the disk, with the maximum occurring in Leca concrete. These high edge stresses may explain the increased overbreaking in lightweight concretes already mentioned in Section 6.3.4.1.

7.5.1.1.1 Influence of Specimen Size on Pull Off Stress

The lack of influence of specimen size on pull off strength was shown experimentally in Section 6.3.4.2. Here further investigation was undertaken to assess the maximum principal stress distribution within a 100×100mm cylinder specimen. The results indicate an identical stress distribution as in the 150×150mm cylinder specimen. Stress distributions in two different size of concrete specimens made from fully Lytag concrete can be compared in Figures 7.13 and 7.15.

7.5.1.1.2 Influence of Disk Material on Pull Off Stress

As previously indicated, aluminium disks were used throughout the investigation. A comparison between calibration obtained using aluminium disks with published calibrations achieved using steel disks on normal weight concrete shows some dependency on disk material. Figure 7.16 compares the calibration obtained through this investigation on normal weight concrete using aluminium disks with the corresponding calibrations using steel disks of identical dimensions obtained by other investigators (Stehno, 1975), (Long, 1983). From this figure it is indicated that higher pulling resistances are generally obtained using steel disks, with the exception of some low strength concrete tests. This phenomenon was examined theoretically to study the stress distribution within the concrete. The study was based on the use of 50mm diameter × 20mm thick disks to conform to the sizes used by the Author and other investigators. From the analysis, it was found that the high rigidity of steel disk (approximately 3 times that of aluminium) ensured that the stress transmitted to the concrete is relatively more uniform across the interface between concrete and disk with less intensity for a given load than was evident using aluminium disks. This is demonstrated in Figure 7.17 by the maximum principal stress distribution in concrete across the interface between disk and concrete.

In reference to Figure 7.16, it is apparent that the increase in pull off resistance using a steel disk seems to be dependent on the type of concrete used by the investigators. This may perhaps be related to the types of natural aggregate and their stiffnesses. BS 8110 : Part 2 (1985) states that for normal weight concrete at a given compressive strength, the modulus of elasticity of concrete varies within the range as tabulated in Table 7.2 of the British Standard. Pull off stresses using aluminium and steel disks have also been examined using the above approach for normal weight concretes with moduli at the extremes of the range quoted for each strength level. Figure 7.18 shows the theoretical upper and lower limits of the ratio of pull off stress using steel disks to pull off stress using aluminium disks. The corresponding ratios of the experimental analyses are also shown in the above figure as obtained through this investigation using aluminium disks linked with the published data using steel disks. From this figure, the experimental results almost fit within the theoretical limits. It can be seen that the results of Stehno (1975)/Author mostly fit within the theoretical range with the exception of those for high strength concrete, whereas the results of Long (1983)/Author show more deviation. The reasons for these discrepancies are not immediately clear, due to the absence of the detailed material properties of the concretes used by Stehno (1975) and Long (1983). This suggests that further investigation is required on the effect of disk material type on the pull off resistance of concrete.

7.5.1.1.3 Influence of Disk Size on Pull Off Stress

As described in a previous section, a disk with 50mm diameter × 20mm thick is most common in U.K. practice. However different sizes of disk such as 75mm diameter × 10mm thick are in use in elsewhere (Hindo, 1990).

Although pull off strength is measured from the force required to cause failure divided by the cross sectional area of the disk, it is believed that the thickness of the disk may have a secondary effect on measurements due to stiffness changes. Theoretical investigation has thus been undertaken to consider the effect of disk thickness. The study was based on using a 50mm diameter disk with different thicknesses namely, 10, 20 and 30mm. The results obtained from linear elastic stress analysis indicate that for a given load the maximum stress transmitted into the concrete through a 10mm thick disk is approximately 45% higher than through a 20mm thick disk. There is, however, a less significant change between a 20mm thick disk and a 30mm thick disk, with the difference approximately 15%. Figure 7.19 (a,b,c) shows the theoretical displacement of the aluminium disks using 10, 20 and 30mm thicknesses. It is apparent from the above figure that in the case of 10mm thick disk, the vertical upward displacement of nodes near the axis of loading is significantly higher than those nodes near the edge of the disk. This finding implies that the concrete at the interface with the disk is subject to localised stress concentrations and this may lead to a lower overall pull off force, and is compatible with the findings of section 7.5.1.1.2.

The investigation was extended to consider a larger aluminium disk, such as 70mm in diameter, to obtain the required disk thickness to achieve similar pull off stress as obtained with 50mm diameter × 20mm thick aluminium disk. From the finite element analysis, it was noted that a 30mm thick disk would create a similar pull off stress as previously obtained using 50mm diameter × 20mm thick aluminum disk. Figure 7.20 shows the maximum principal stress in normal weight concrete with a cube strength of 25N/mm^2 using 70mm diameter × 30mm thick aluminium disk. Clearly this figure shows the same maximum tensile stress as that indicated in Figure 7.10 for an equal pull off stress. This observation supports the speculative British Standard recommendation (BS 1881 : Part 207, 1991) that requires the thickness/diameter ratio of disk to be not less than 0.4.

7.5.1.2 Partial Cored Pull Off Tests

Partial cored pull off tests covered in Section 6.3.4.3(b) showed a reduction in pull off resistance when compared to surface pull off tests. The degree of reduction was found to be dependent on the type of concrete and this influence can be seen in Figures 6.97 to 6.100. A theoretical investigation was carried out to find a possible explanation for this reduction in pull off strength with particular emphasis on the influence of concrete type.

Stress analysis in surface pull off tests show that those elements around the edge are contributing substantially to the carrying of the applied load (see Tables C-1 to C-5). This is more pronounced in lightweight concretes being greatest in Leca concrete. Partial coring results in the removal of these edge elements. This creates a stress concentration at the base of the partial core and the net result is a reduction in pull off resistance. The magnitude of this reduction in pull off resistance is largely dependent on the type of concrete as the load carrying capacity of edge elements is dependent on concrete type. Table 7.5 shows the theoretical percentage reduction in pull off stress in partial cored pull off tests (20mm core length) when compared with surface pull off tests. A comparison between experimental results and those theoretical predictions is evident in the table and shows a good agreement for lightweight concrete with the exception of Leca concrete. In normal weight concrete, the theoretical prediction agrees very well with experimental observation using formed cores. This further demonstrates the influence of the drilling action in weakening the concrete and hence further reducing the pull off strength. The high reduction in partial cored pull off stress of Leca concrete as obtained during the experiment is likely to be related to aggregate weakness and non-uniform distribution of aggregate as explained previously in Section 6.4.1.3. These parameters obviously would not be considered in the finite element analysis which assumed the concrete to be homogenous within each element. Figure 7.21 demonstrates the typical stress distribution in partial cored pull off tests (20mm core length using Pellite concrete). This figure clearly shows the existence of high tensile stresses at the edge interface between the base of the partial core and the remaining body of concrete. Similar high stresses also occur beneath the aluminium disk. This is borne out by the observation that the fracture location varies in position, suggesting that failure occurs where the concrete is weakest.

Reduction in partial cored pull off strength with increasing core length has been investigated experimentally on fully Lytag concrete (see Section 6.3.4.3(b)) which showed that the reduction is significant only up to 20mm in core length. To provide better understanding of the above observation, stress analyses were taken into account. As indicated earlier in this section the edge elements are substantially contributing to carrying the applied load. However, this contribution as shown in Tables C-1 to C-5 gradually reduces with depth. It is apparent from these tables that most of the stress is concentrated in those edge elements within a depth of approximately 20mm from the surface. Hence any concrete removed from this region may be expected to have a significant effect on pull off strength. Figure 7.22 shows the variation in pull off stress with core length on fully Lytag concrete, and indicates the significance of increase in core length up to 20mm. Both experimental results and theoretical predictions are shown, and are generally in good agreement.

7.5.2 The Significance of the Theoretical Study of the Pull Off Resistance

The linear elastic stress analysis shown the significance of the rigidity of concrete in the pull off resistance. In the surface test, a higher pull off stress results from a lower concrete stiffness, and this is confirmed by the experimental findings. Increased overbreaking, as observed in low stiffness concrete, seems to be linked with edge stresses, as it was greatest with the lower concrete stiffnesses. British Standard (BS 1881 : Part 207, 1991) recommends the use of a metal disk in the test, but the theoretical analysis shows that the type of metal needs to be specified, as disk rigidity has some influence on pull off resistance. This phenomenon is evident if the results of the work described here using aluminium disks on normal weight concrete are compared with the results of others using steel disks on normal weight concrete. Furthermore, limited stress analysis carried out using different thickness/diameter ratios of the disk shows that this ratio has a significant effect on the pull off resistance. The results obtained in this theoretical analysis support the minimum thickness/diameter ratio of 0.4 as proposed by the new British Standard (BS 1881 : Part 207, 1991). These results further suggest that the thickness/diameter ratio should be standardized to a specific single value.

The reduction in pull off stress in partial cored tests as compared to surface tests, has been shown by means of stress analysis to be related to the removal of edge elements, which contribute substantially to the carrying of the applied load, and stress concentration created at the base of the partial core by forming the core. This reduction has been shown to be dependent on the core length, but the effect of change in core length will be negligible for core length greater than 20mm. It is thus recommended that if partial coring is specified, this should not be less than 20mm in length.

The development of the finite element system used here assumed a linear elastic behaviour for concrete under tension, and the analyses were carried out at the stage up to 50% of failure load. The possible non-linear behaviour of the concrete in tension, and the presence of particles of various sizes, shapes and stiffnesses, could be factors limiting the application of the technique here. Further investigation is required to take into account the above factors to obtain more accurate analyses and to extend these to the ultimate stage. Nonetheless, the theoretical results given here highlight the significance of concrete type on pull off tests and support the use of different calibrations as illustrated in Section 6.3.4.3.

Table 7.1:Typical measurements of angle of failure(θ) for various concretes

Concrete Type	Angle of Failure(θ)
Fully Lytag Concrete	27,30,26
Semi Lytag Concrete	27,28,31
Pellite Concrete	30,27,28
Normal Weight Concrete Salman(1979)	27

Table 7.2:Predicted relationship between pull out force and compressive strength in direct pull internal fracture test

Concrete Type	Pull out force/compressive strength relationship	
	Wet Cured	Dry Cured
Fully Lytag Concrete	f_c=1.91 $F^{1.56}$	f_c=0.01 $F^{4.76}$
Semi Lytag Concrete	f_c=2.41 $F^{1.43}$	f_c=0.13 $F^{3.13}$
Pellite Concrete	f_c=2.44 $F^{1.45}$	f_c=1.80 $F^{1.79}$

Table 7.3:Material properties of various types of concrete

Concrete Type	Compressive Strength N/mm^2	Modulus of Elasticity N/mm^2	Assumed Poisson's Ratio
Fully Lytag Concrete	25 40	14500 16000	0.2 0.2
Semi Lytag Concrete	25 40	16300 20000	0.2 0.2
Leca Concrete	25	11800	0.2
Pellite Concrete	25 40	19200 23000	0.2 0.2
Normal Weight Concrete	25 40	25000 28000	0.2 0.2

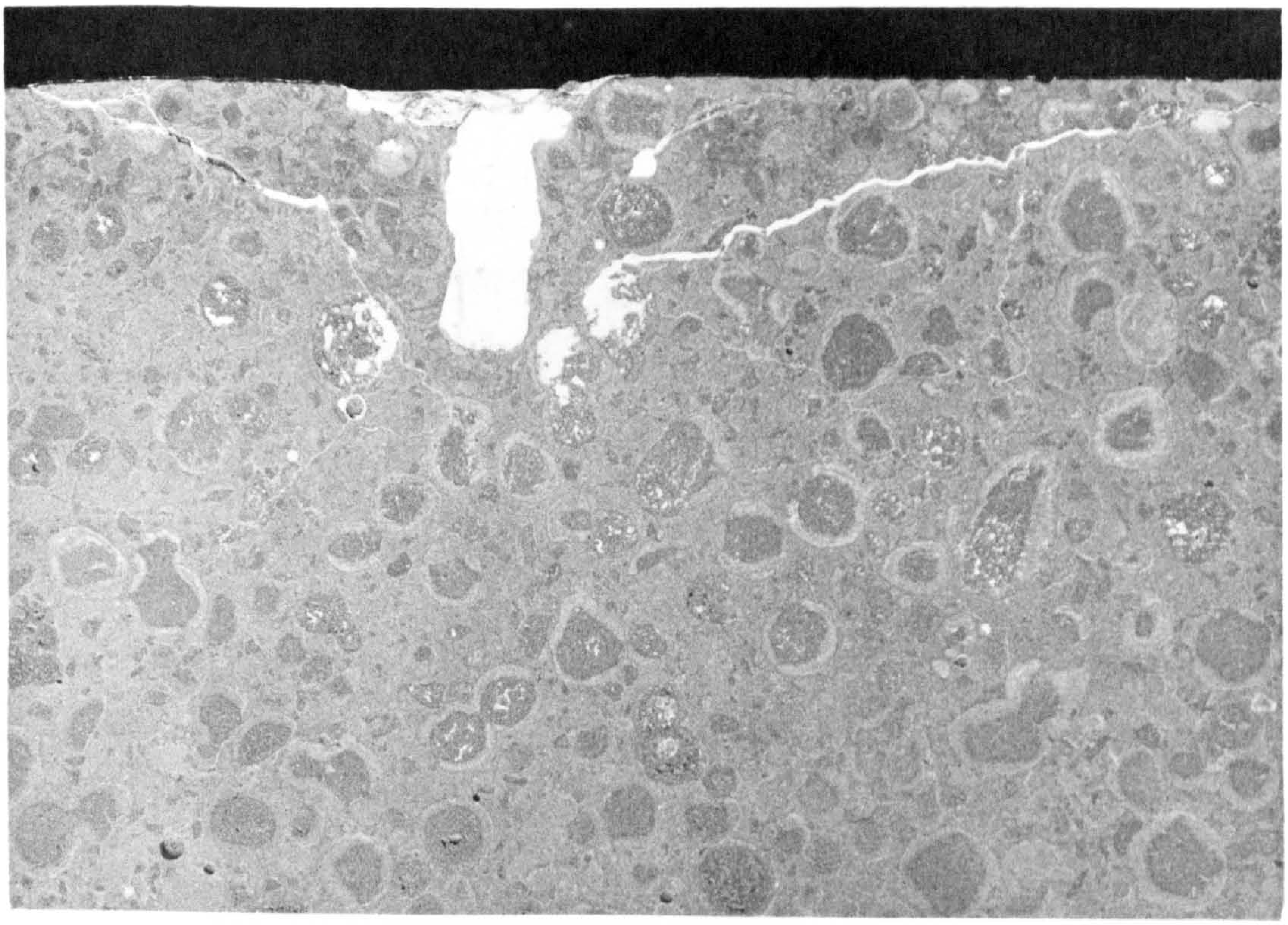
Table 7.4:Comparison of ratios of pull off stress in lightweight concrete to pull off stress in normal weight concrete

Concrete Type	Pull Off Stress Ratio	
	Theory	Experiment
Pellite Concrete	1.10	1.06
Semi Lytag Concrete	1.18	1.11
Fully Lytag Concrete	1.20	1.21
Leca Concrete	1.24	1.33

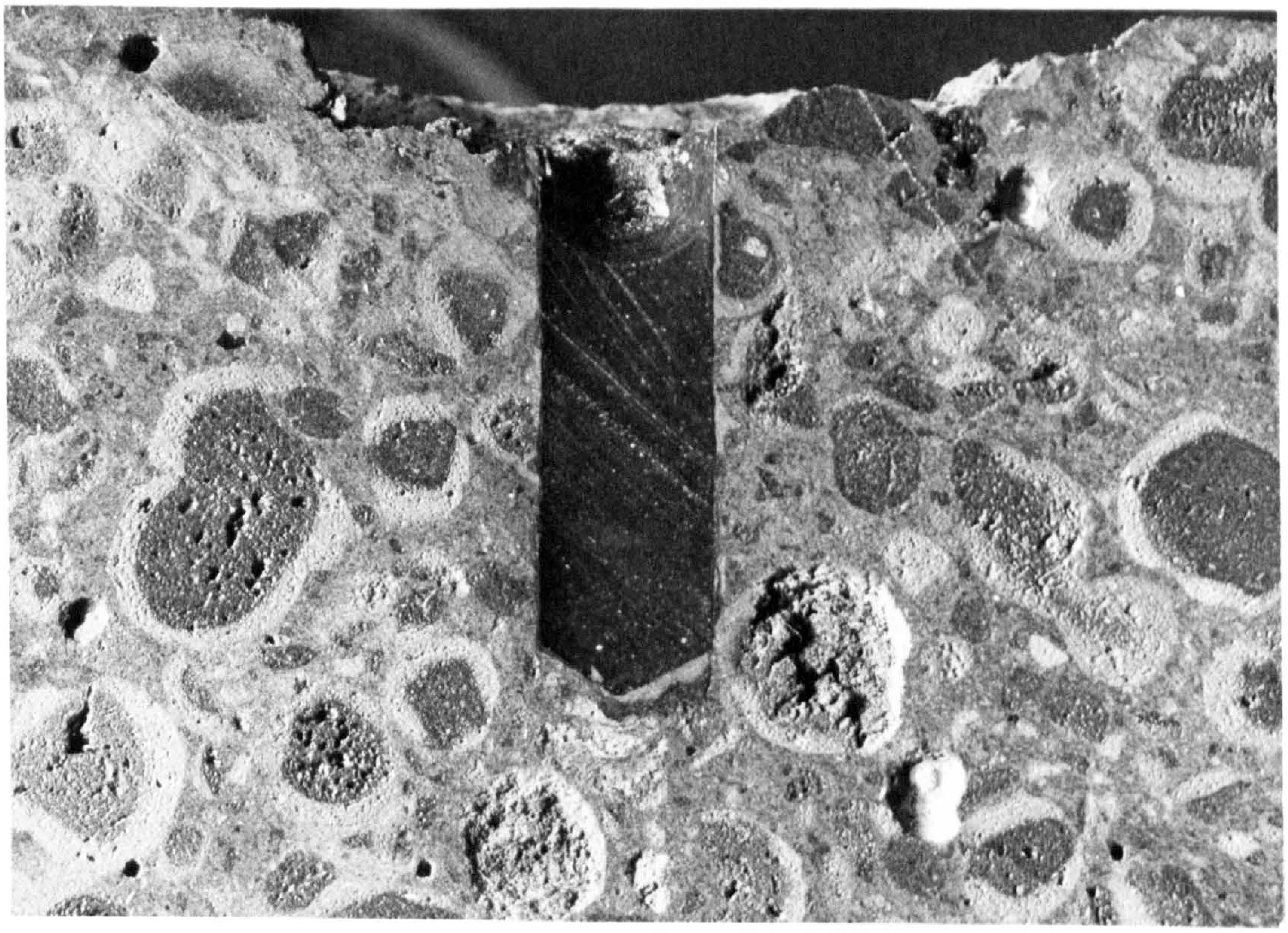
Table 7.5:Percentage reduction in partial cored pull off stress(core length of 20 mm) compared to surface pull off stress

Concrete Type	Compressive Strength N/mm^2	% of Reduction in Pull Off Stress	
		Theory	Experiment
Normal Weight Concrete	40	20	27(20$^+$)
Pellite Concrete	40	25	24
Semi Lytag Concrete	40	30	26
Fully Lytag Concrete	40	35	32
Leca Concrete	25	33	52

+;Formed core using split former(see section 6.3.4.3(b))



(a)



(b)

Figure 7.1: Internal mode of fracture caused by the Windsor Probe impact

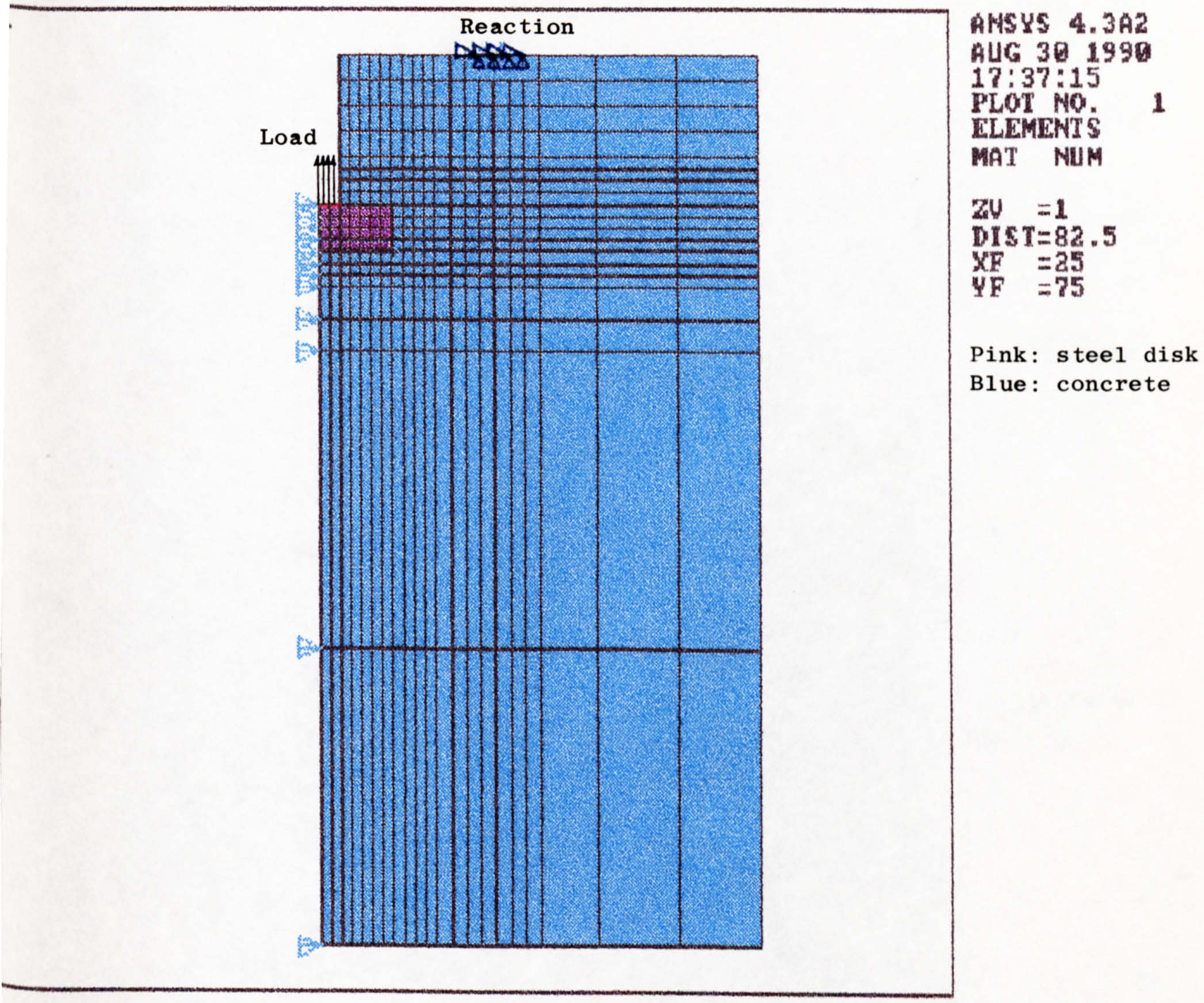
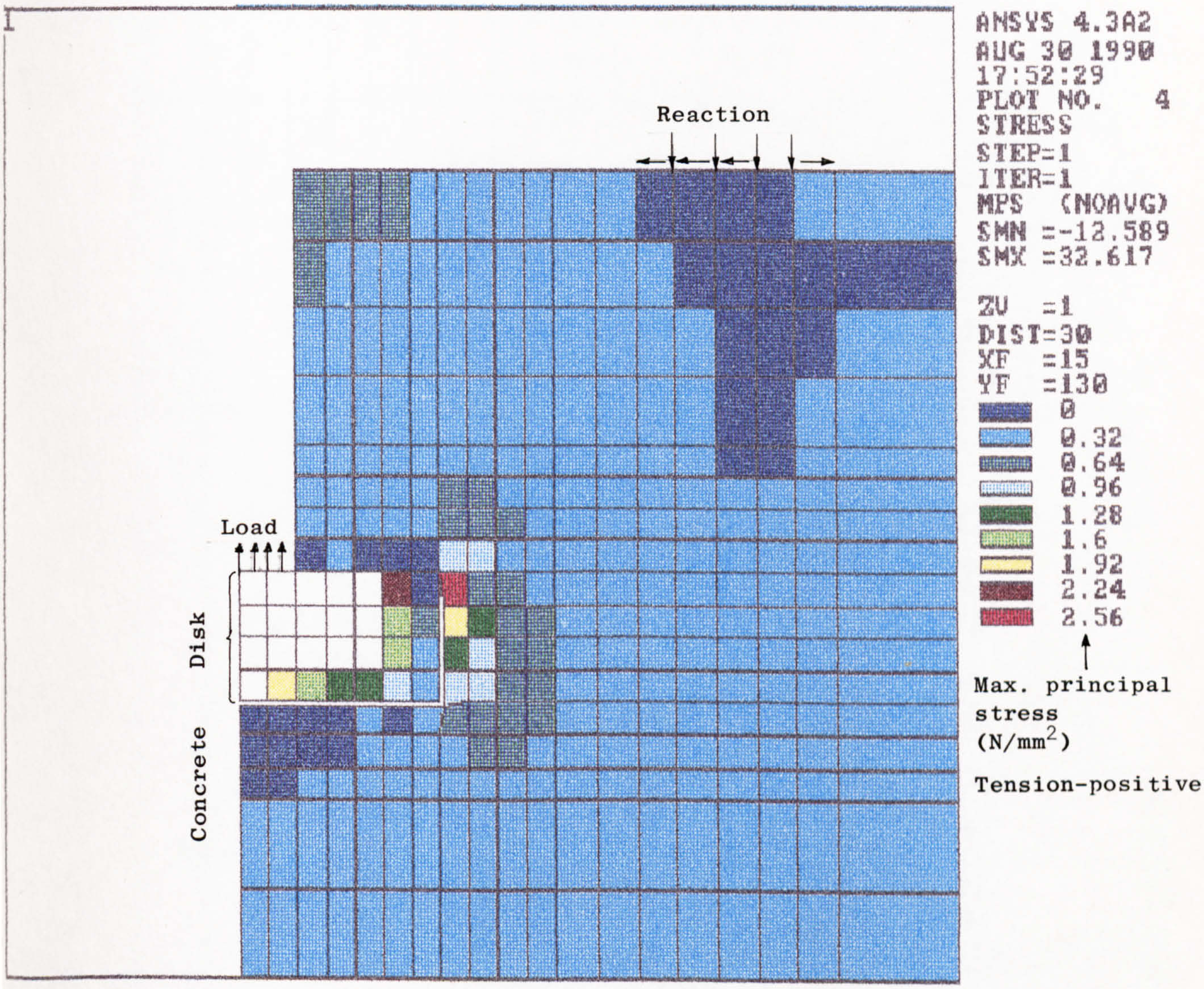
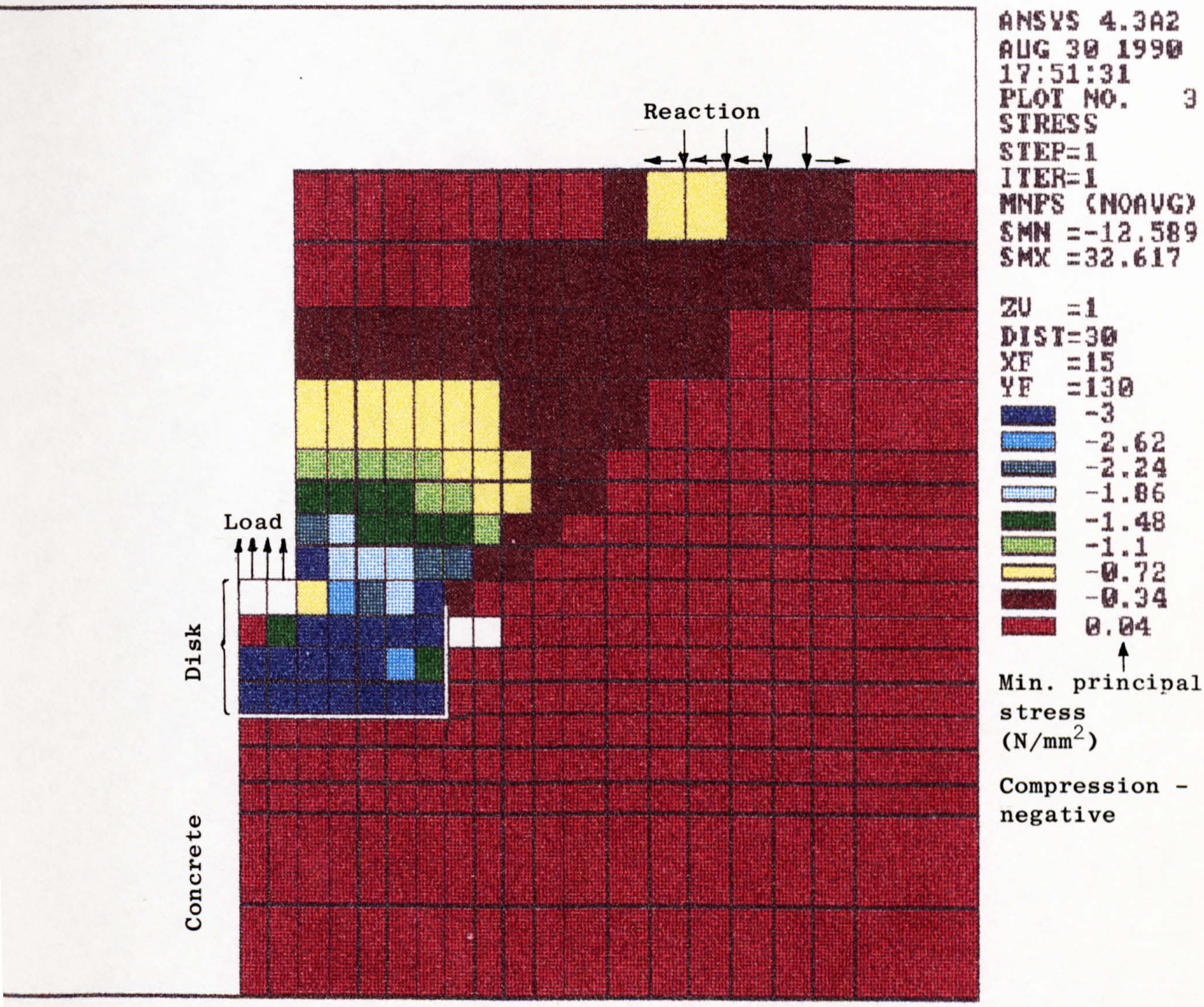


Figure 7.2: Finite element mesh for pull out test



N.B. The stress level in the elements coloured white is greater than 2.56 N/mm^2

Figure 7.3: Maximum principal average element stress distribution in semi Lytag concrete under pull out test



N.B. The stress level in the elements coloured white is greater than 0.04 N/mm^2

Figure 7.4: Minimum principal average element stress distribution in semi Lytag concrete under pull out test

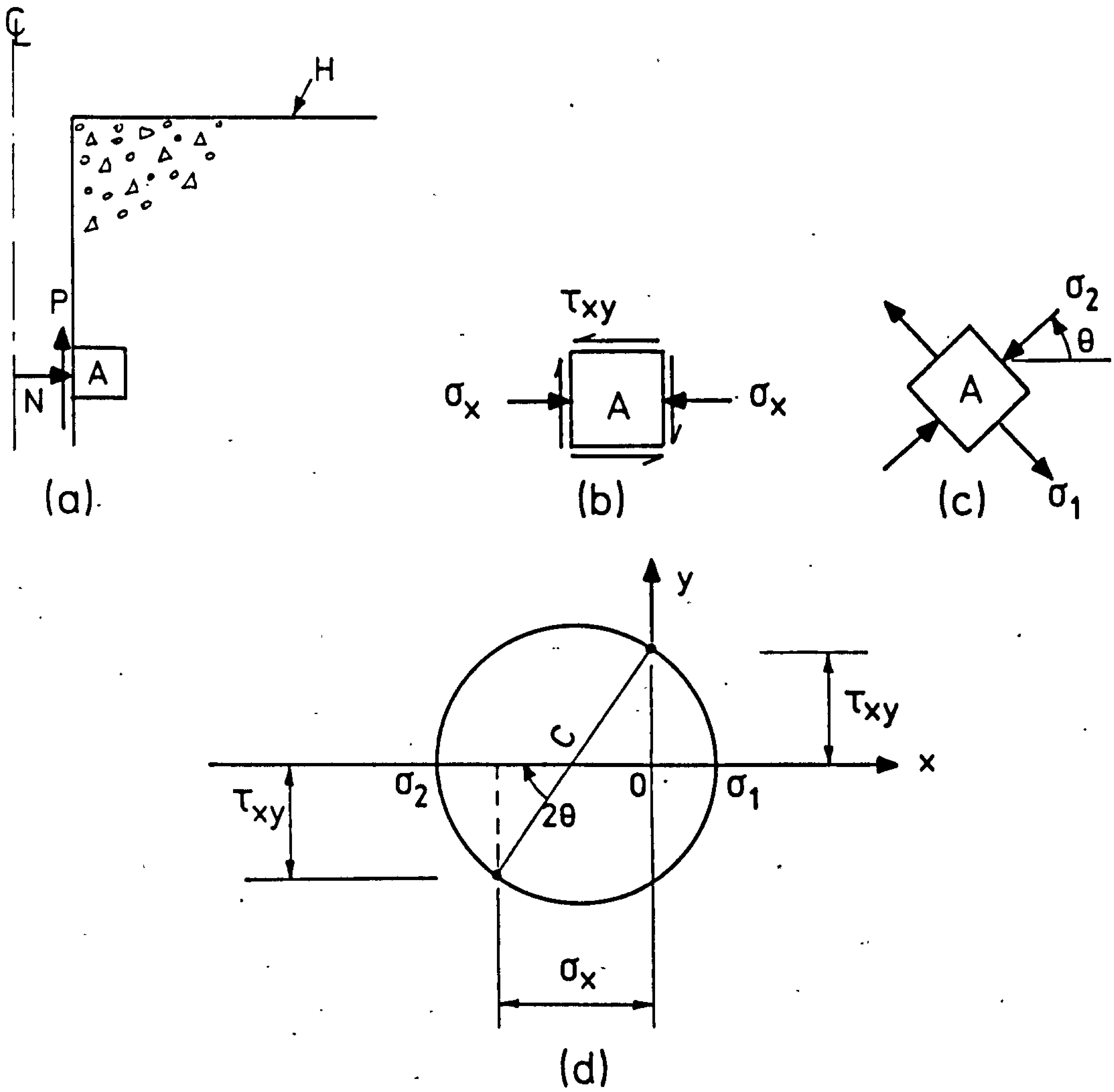


Figure 7.5: Two dimensional stress analysis at the interface between wedge anchor bolt and concrete

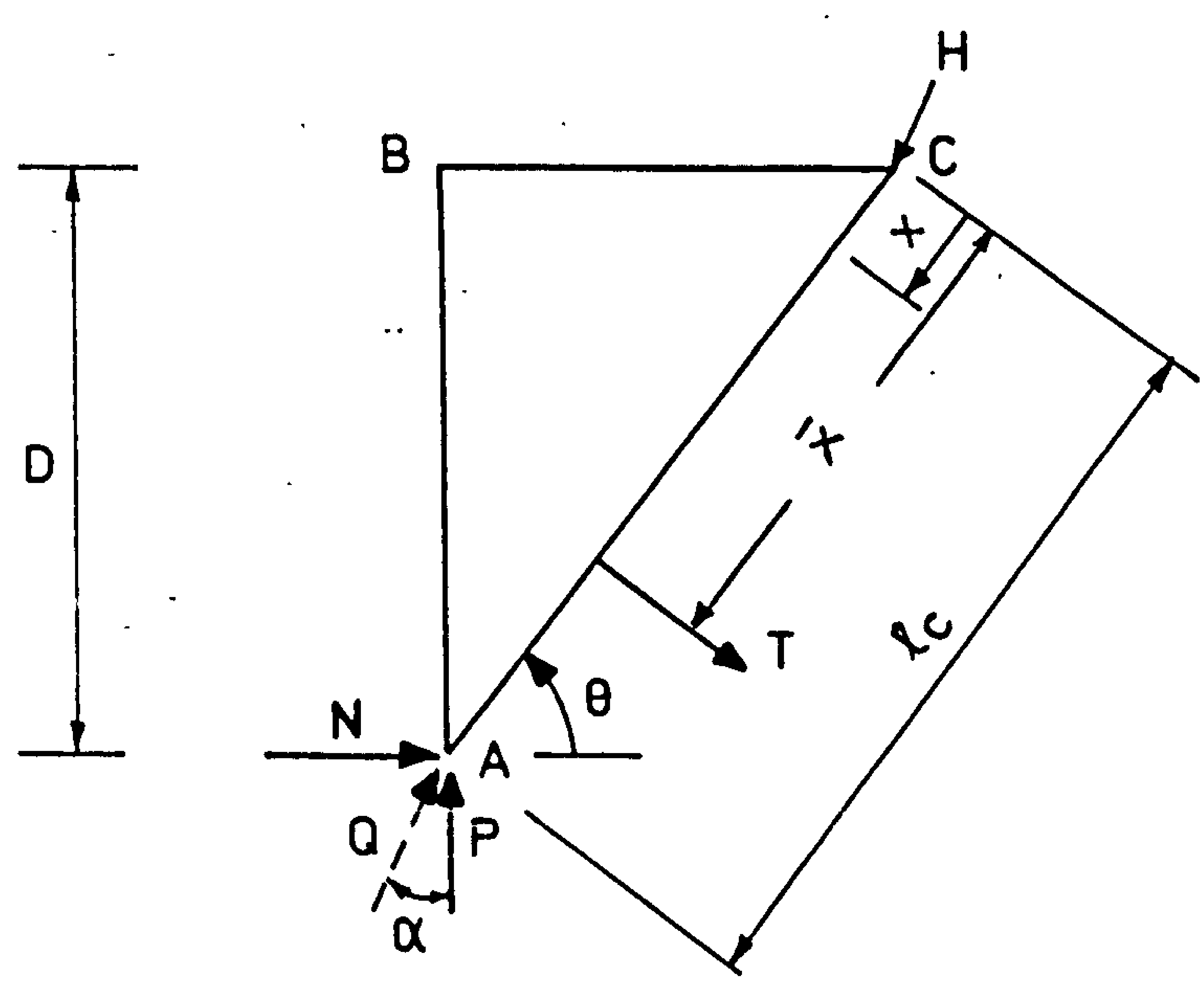


Figure 7.6: Forces acting on a sector of the truncated concrete cone

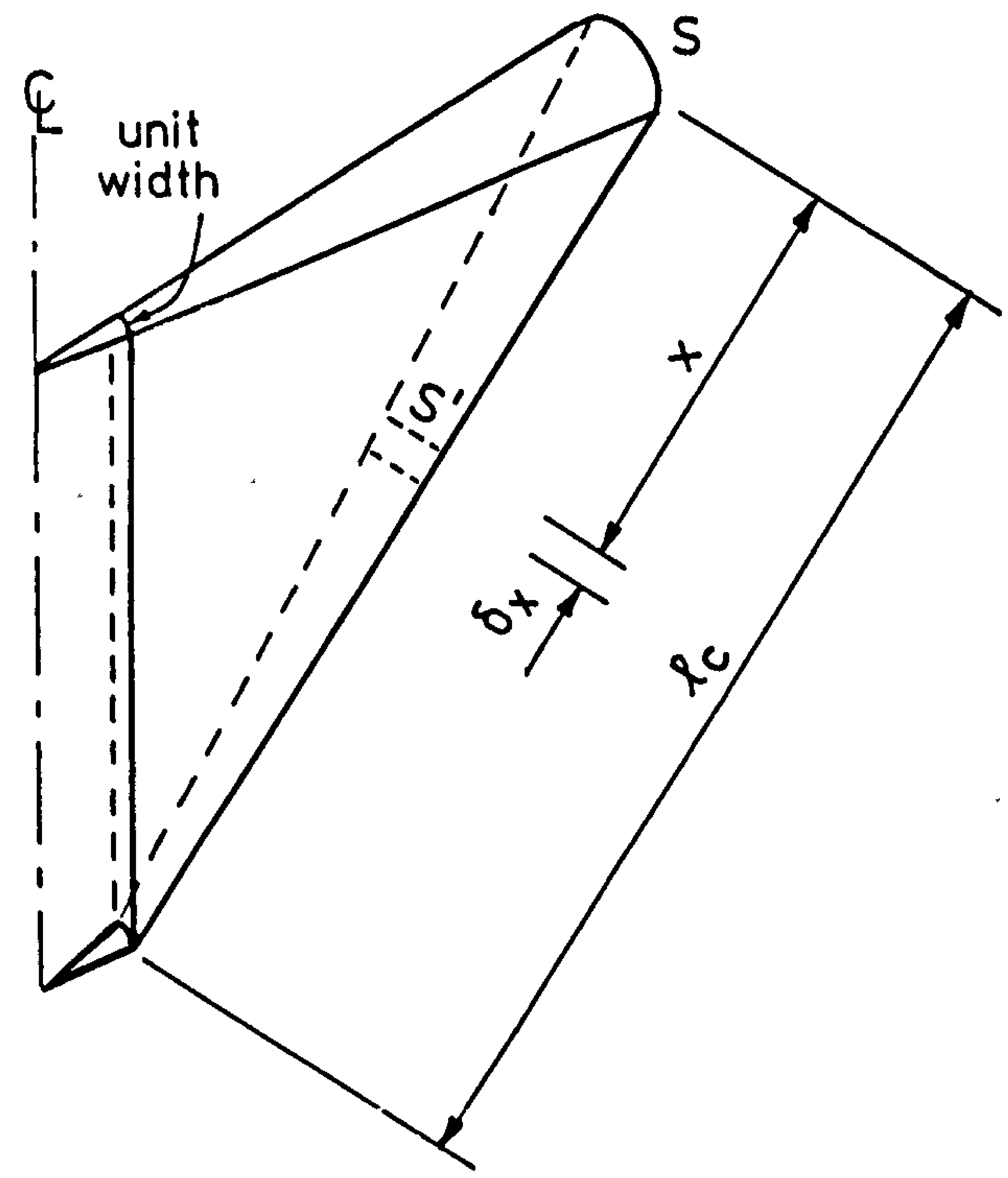


Figure 7.7: Ilustration of the sector of the truncated concrete cone used for analysis

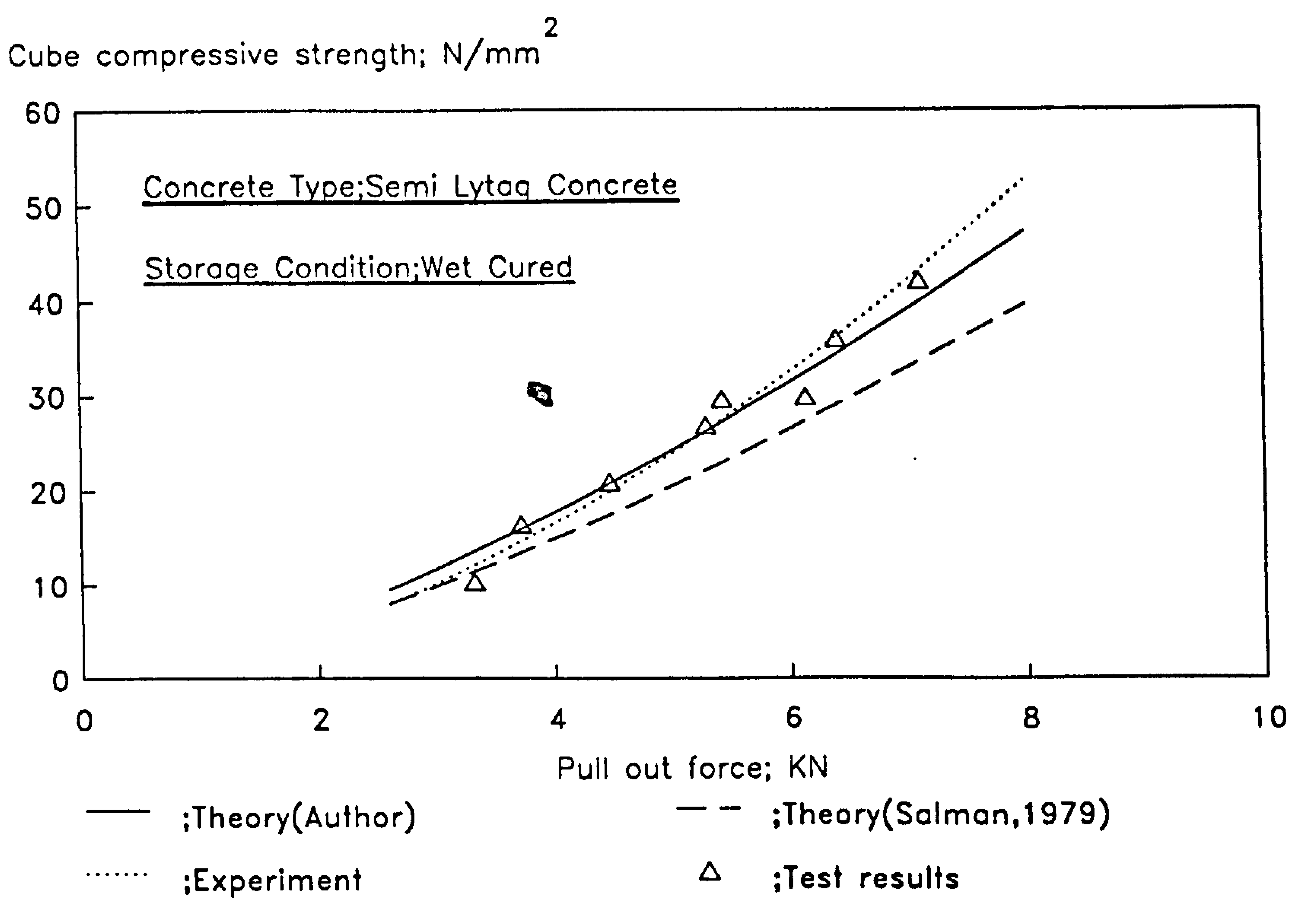


Figure 7.8:Typical comparison of theoretical and experimental relations of direct pull internal fracture force and compressive strength

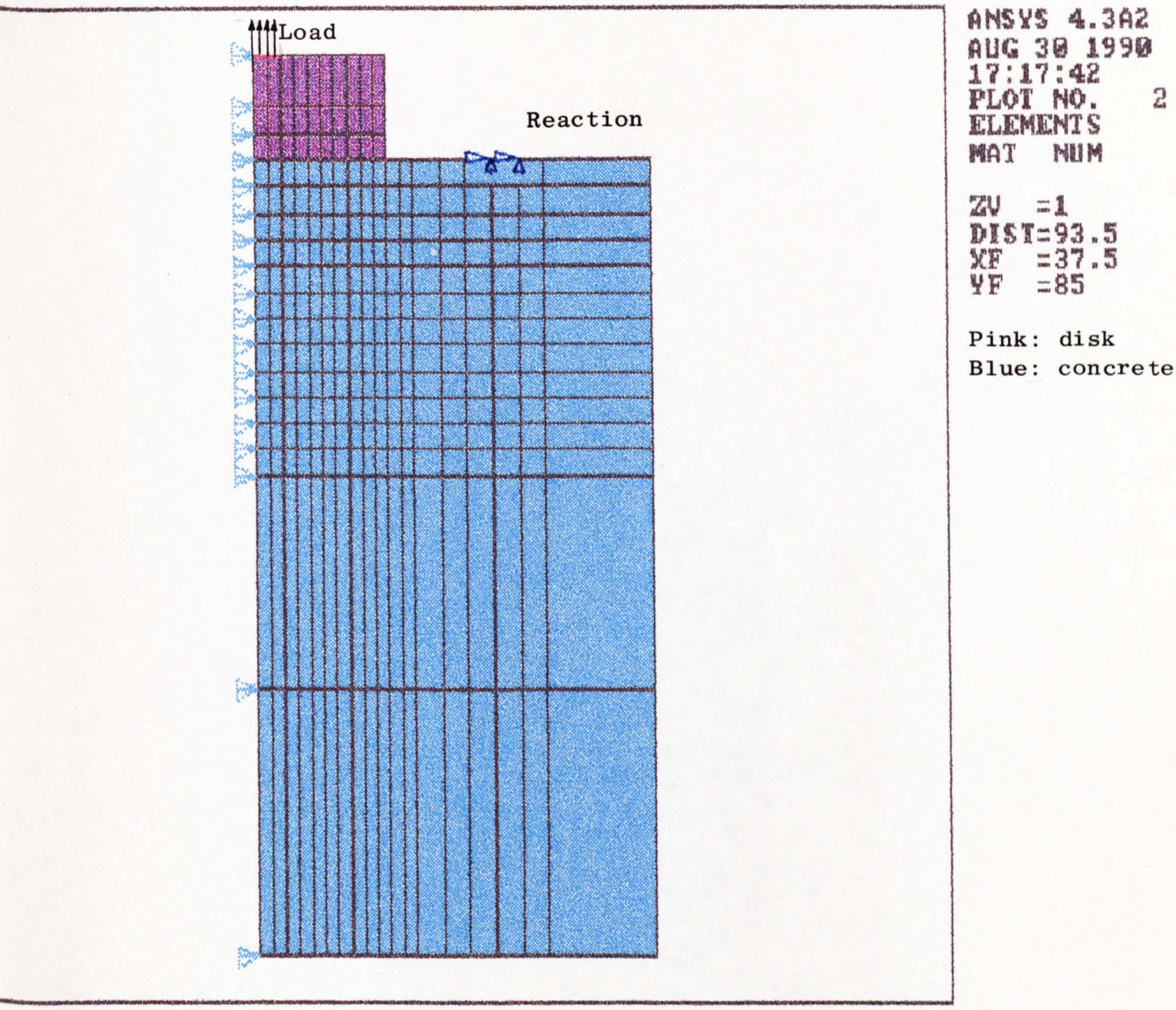
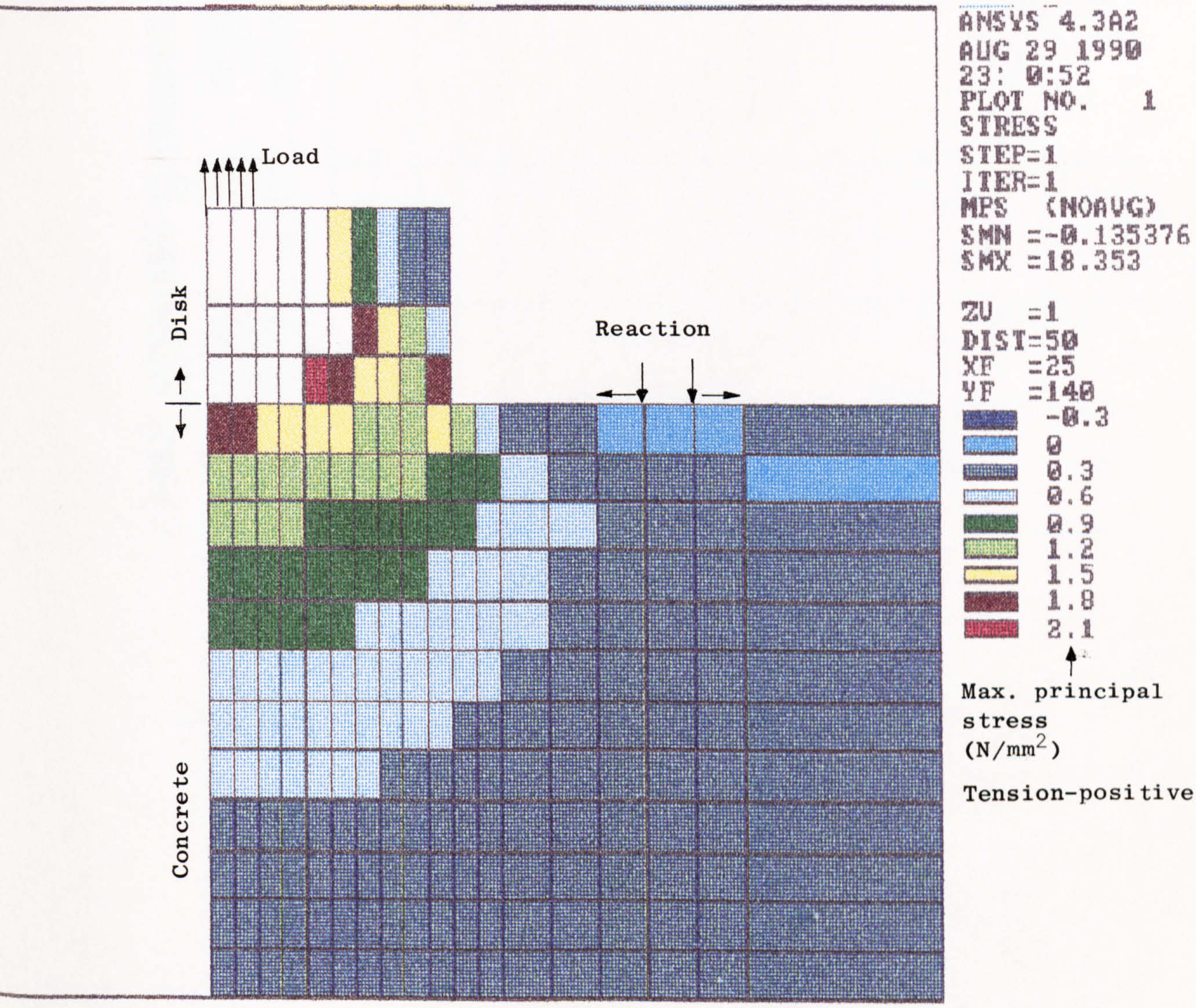
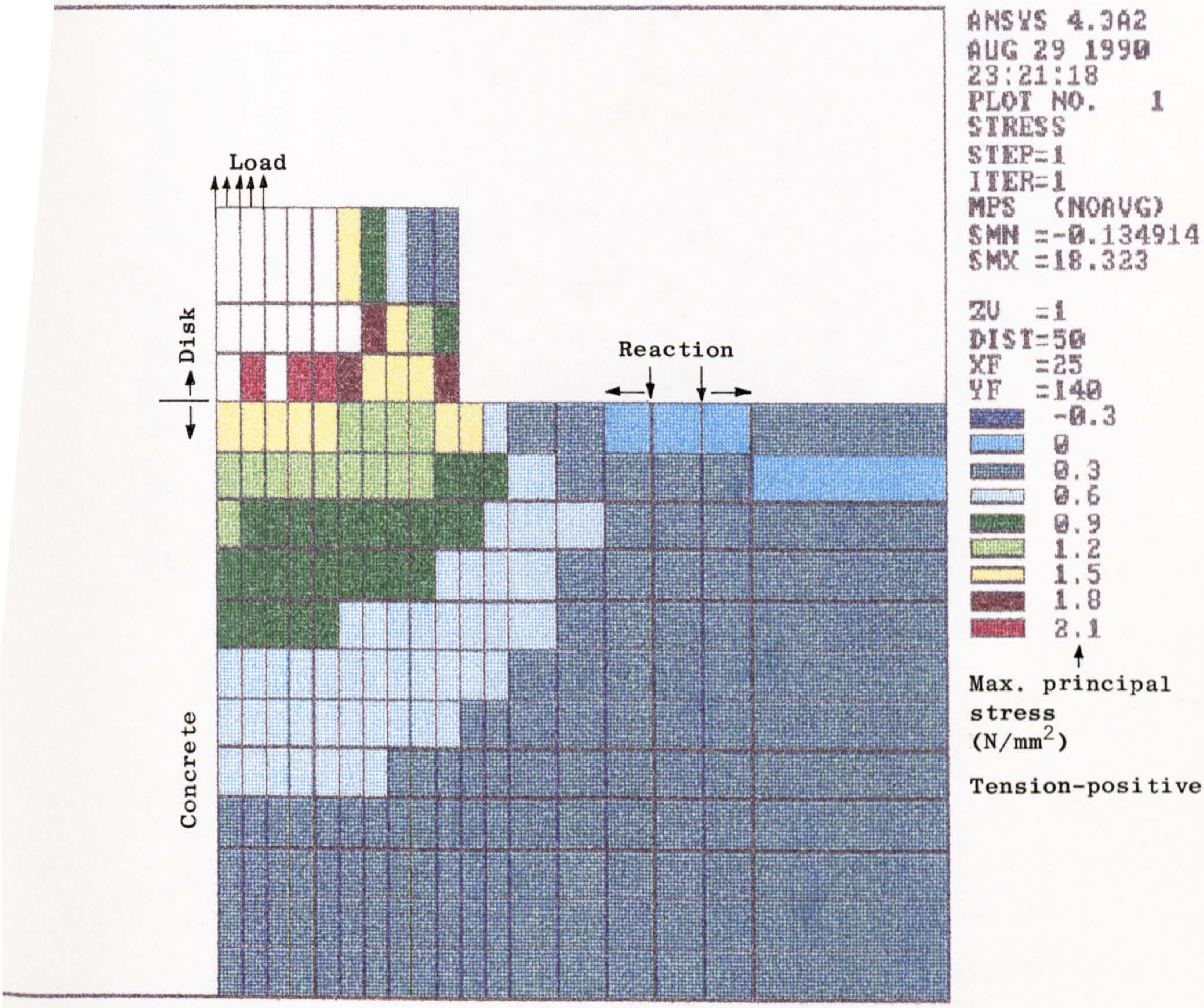


Figure 7.9: Finite element mesh for surface pull off tests



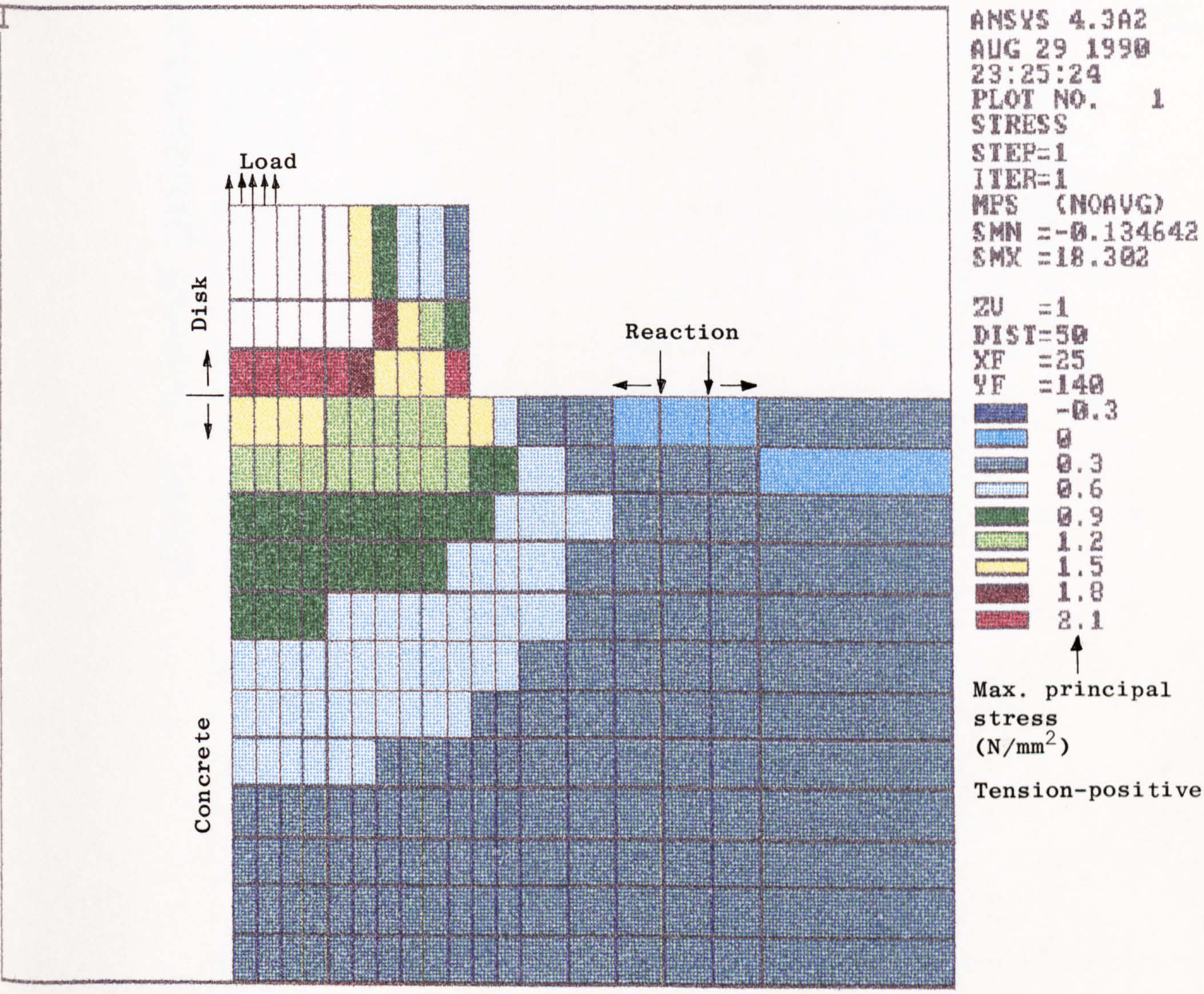
N.B. The stress level in the elements coloured white is greater than 2.1 N/mm^2

Figure 7.10: Maximum principal average element stress distribution for surface pull off test on normal weight concrete



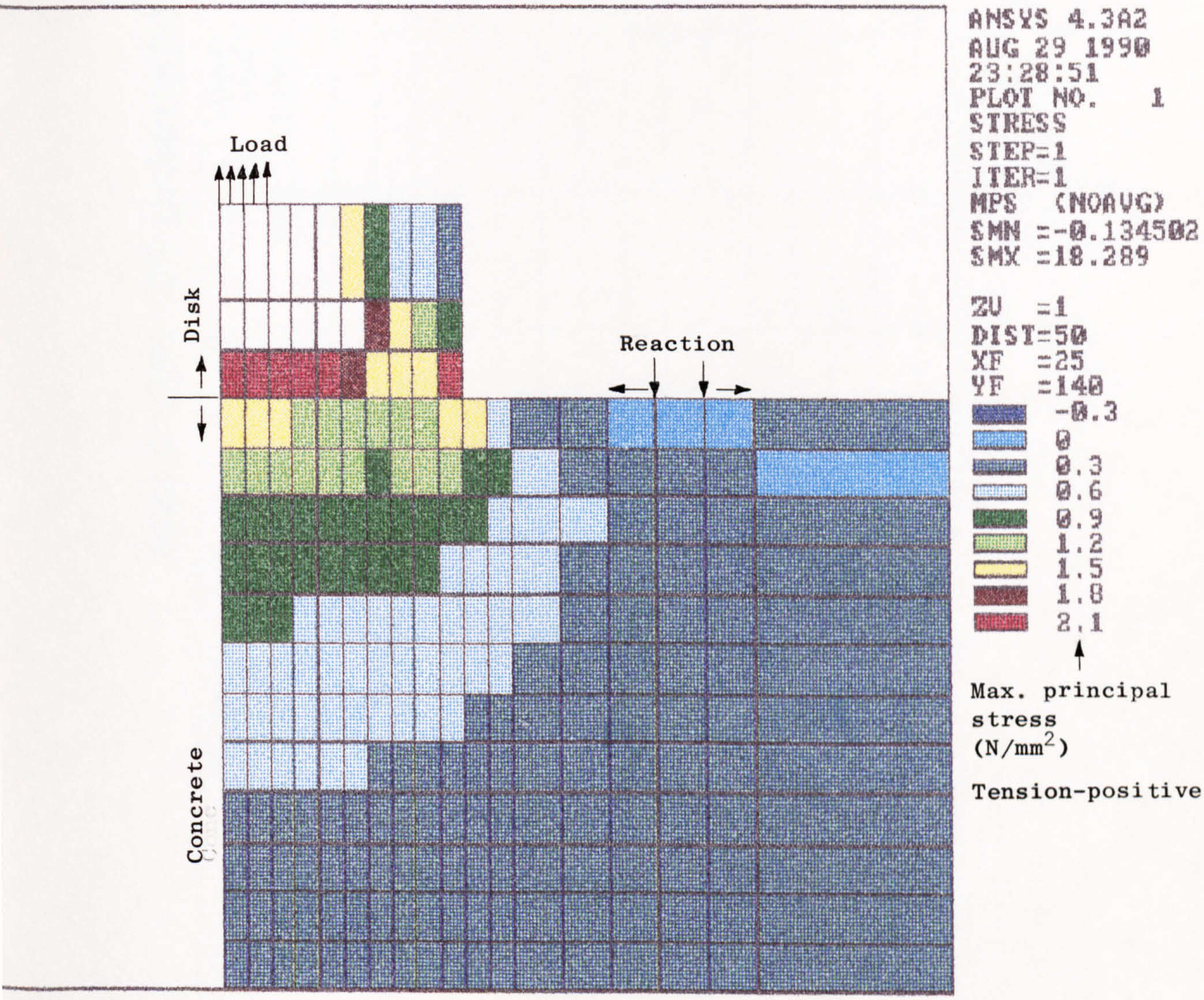
N.B. The stress level in the element coloured white is greater than 2.1 N/mm^2

Figure 7.11: Maximum principal average element stress distribution for surface pull off test on Pellite concrete.



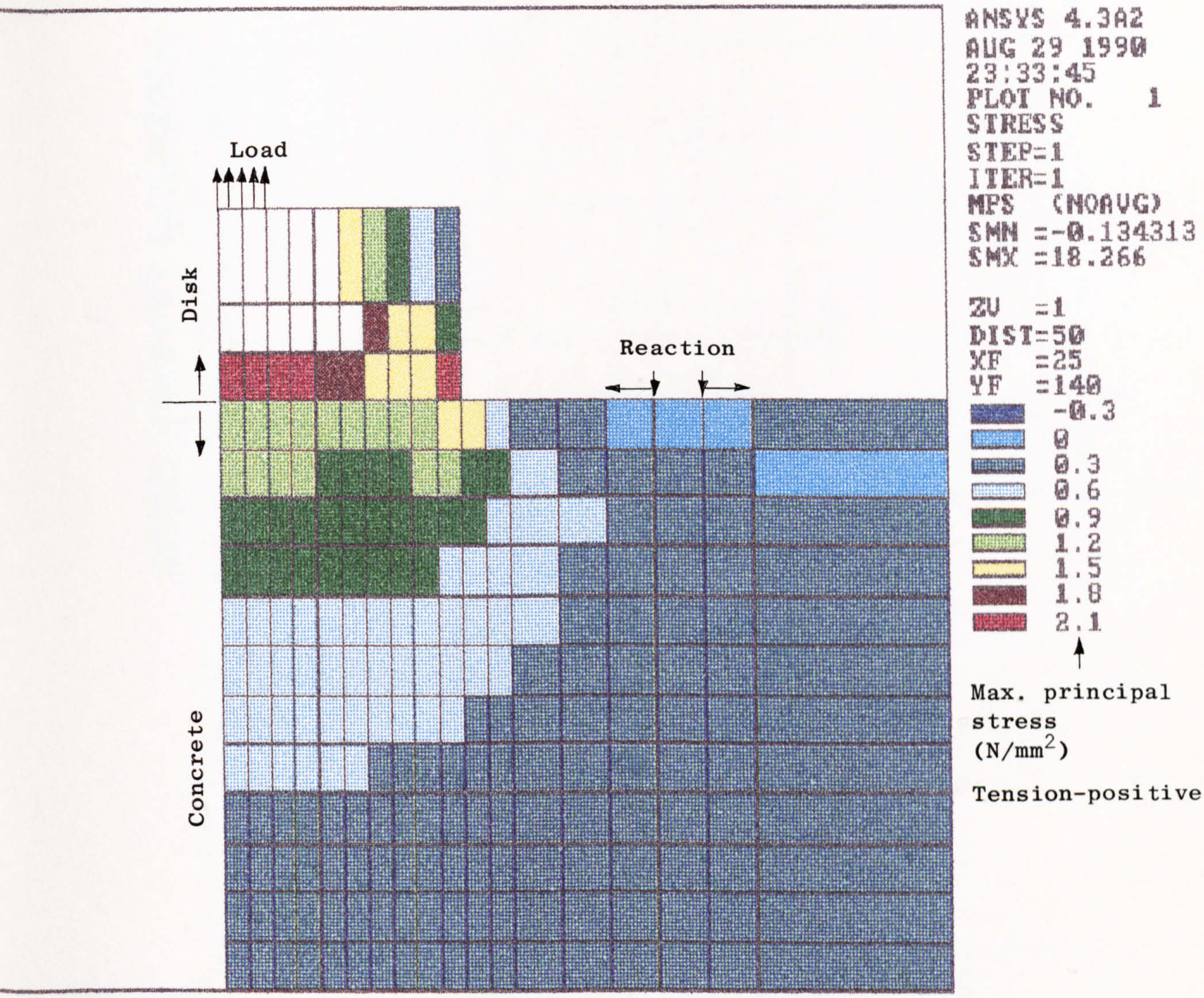
N.B. The stress level in the elements coloured white is greater than 2.1 N/mm^2

Figure 7.12: Maximum principal average element stress distribution for surface pull off test on semi Lytag concrete



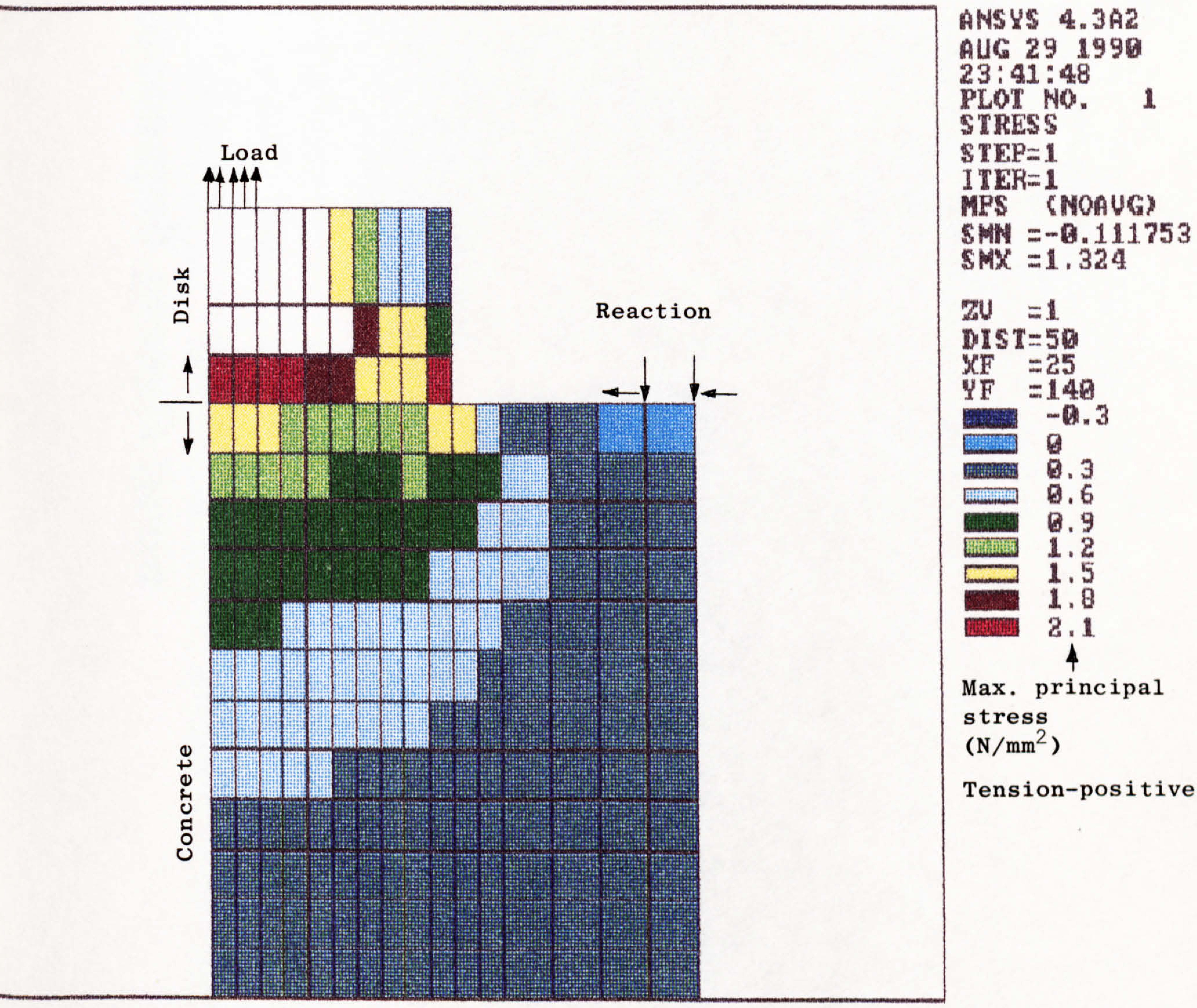
N.B. The stress level in the elements coloured white is greater than 2.1 N/mm^2

Figure 7.13: Maximum principal average element stress distribution for surface pull off test on fully Lytag concrete



N.B. The stress level in the elements coloured white is greater than 2.1 N/mm^2

Figure 7.14: Maximum principal average element stress distribution for surface pull off test on Leca concrete



N.B. The stress level in the elements coloured white is greater than 2.1 N/mm^2

Figure 7.15: Maximum principal average element stress distribution for surface pull off test on fully Lytag concrete using a 100x100mm cylinder specimen

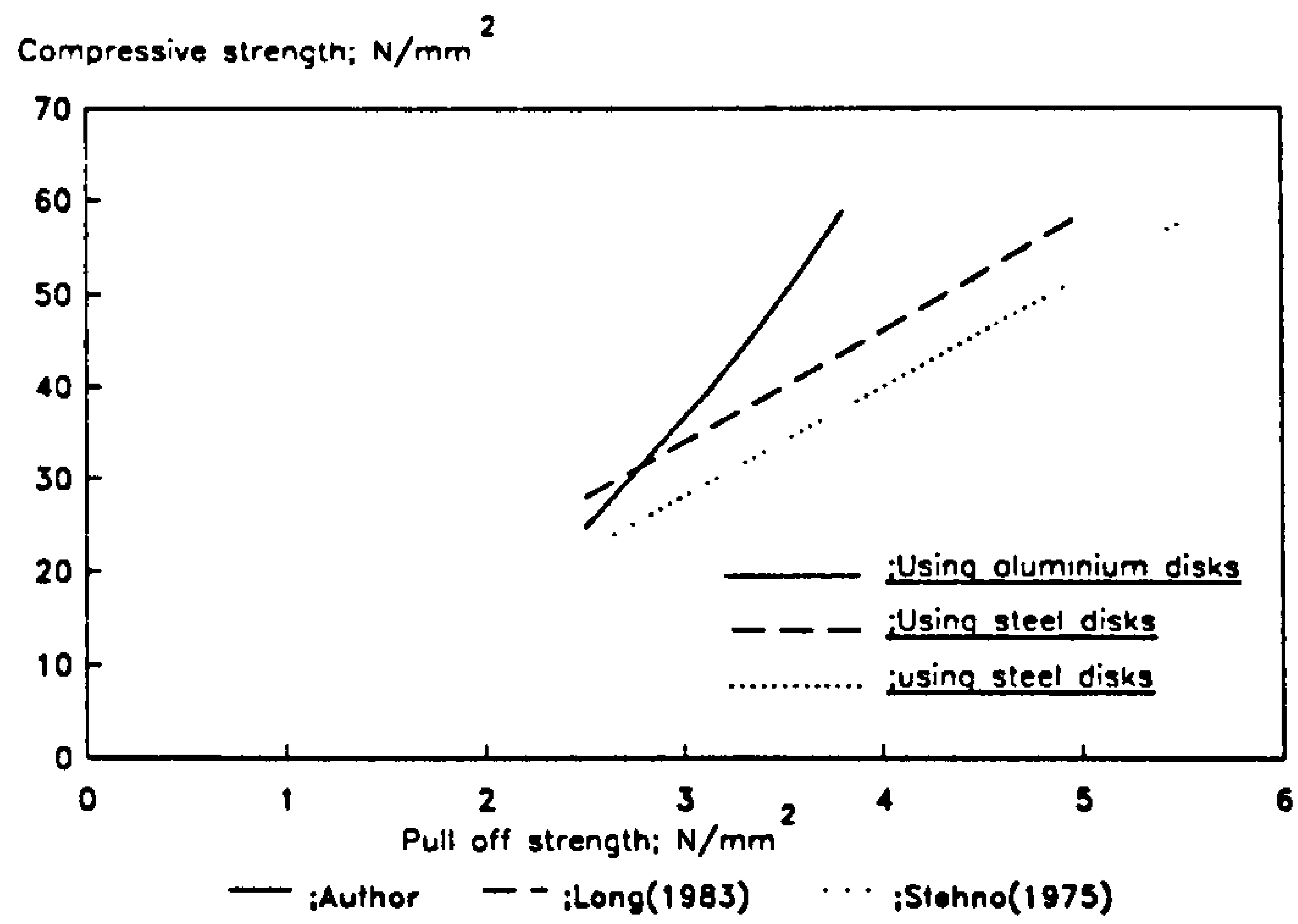


Figure 7.16:Empirical relationships between pull off strength and cube compressive strength on normal weight concrete

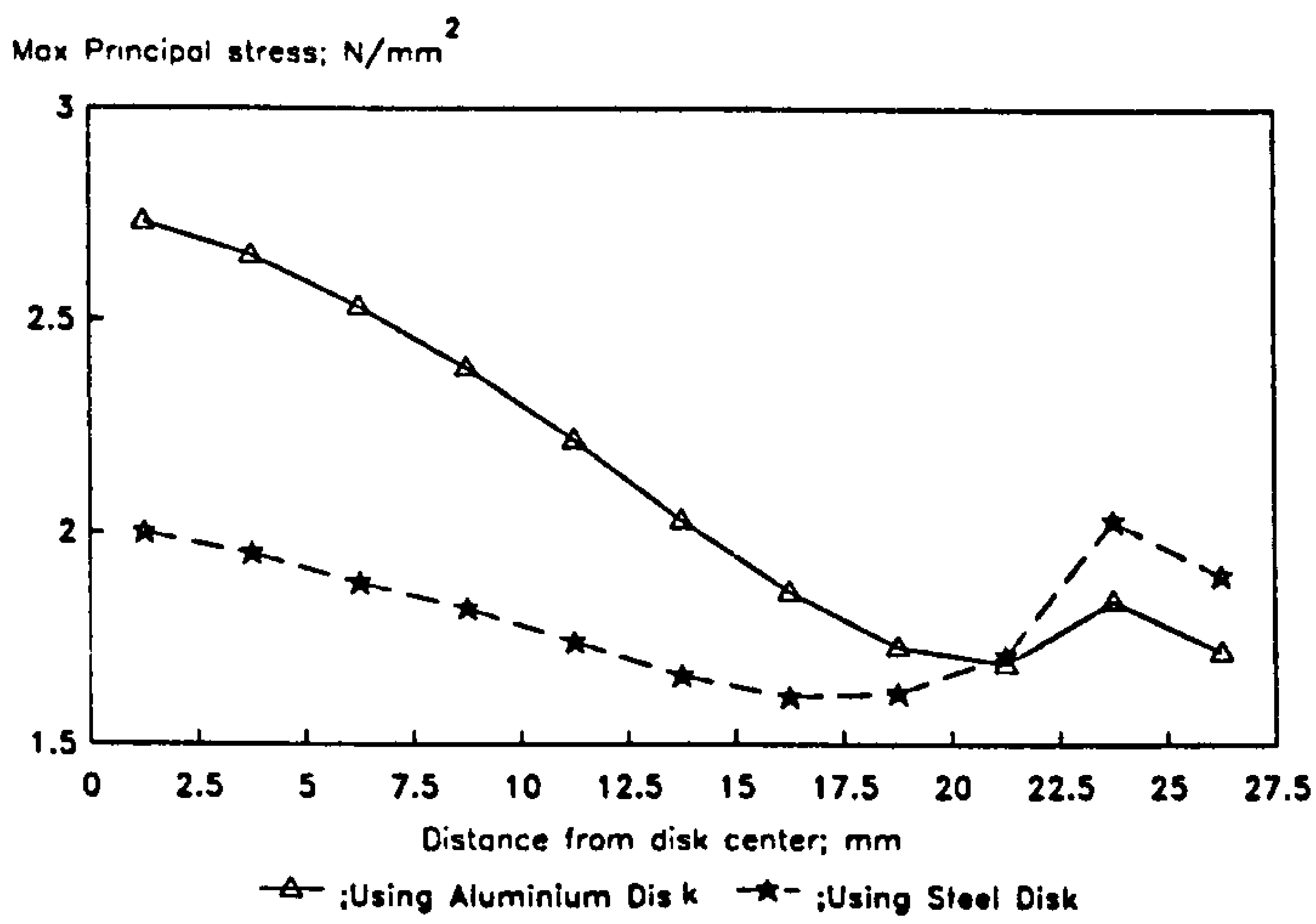


Figure 7.17:Distribution of max principal stress in concrete at the interface between concrete and disk

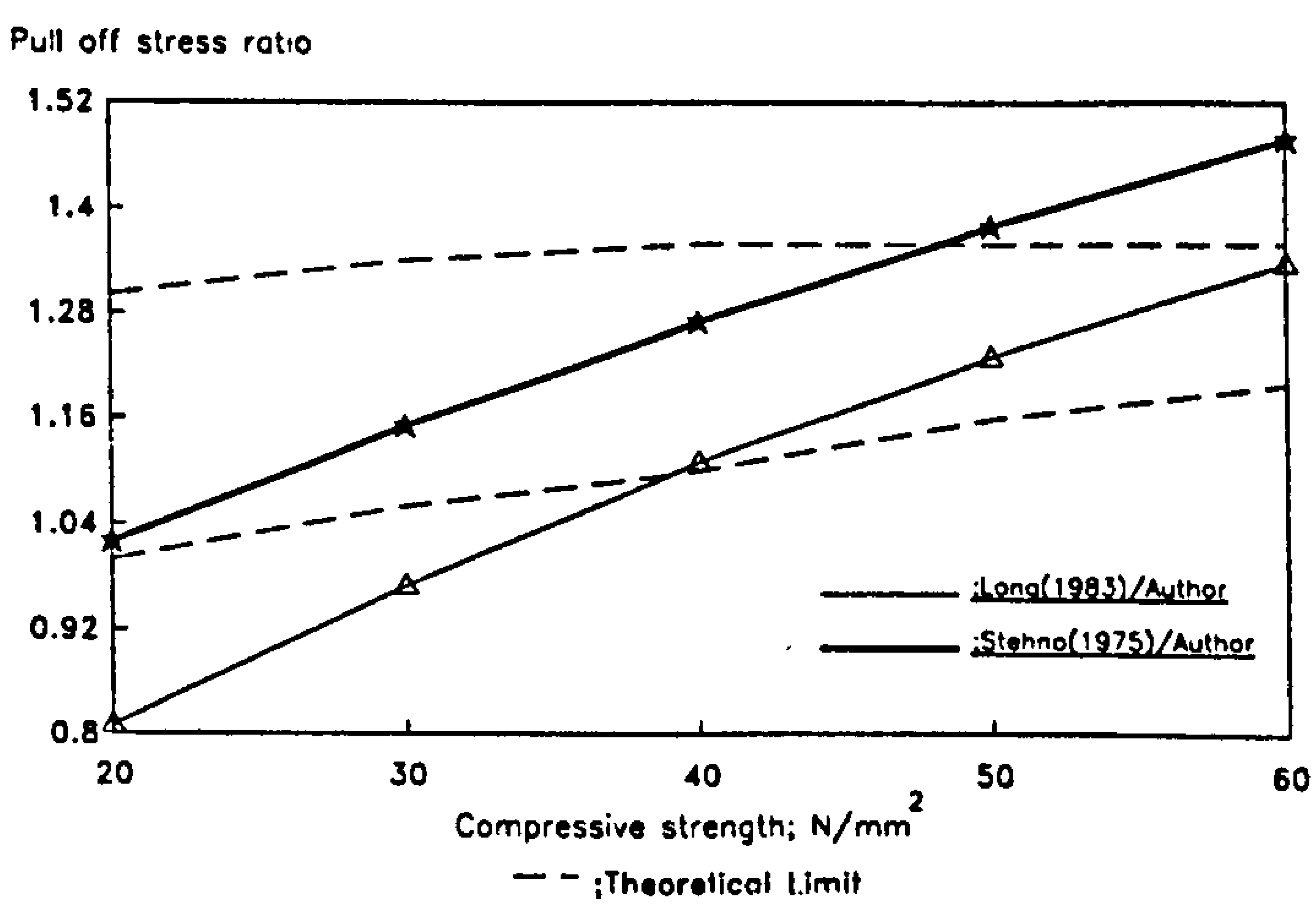


Figure 7.18:Pull off stress using steel disk/pull off stress using aluminium disk on normal weight concrete

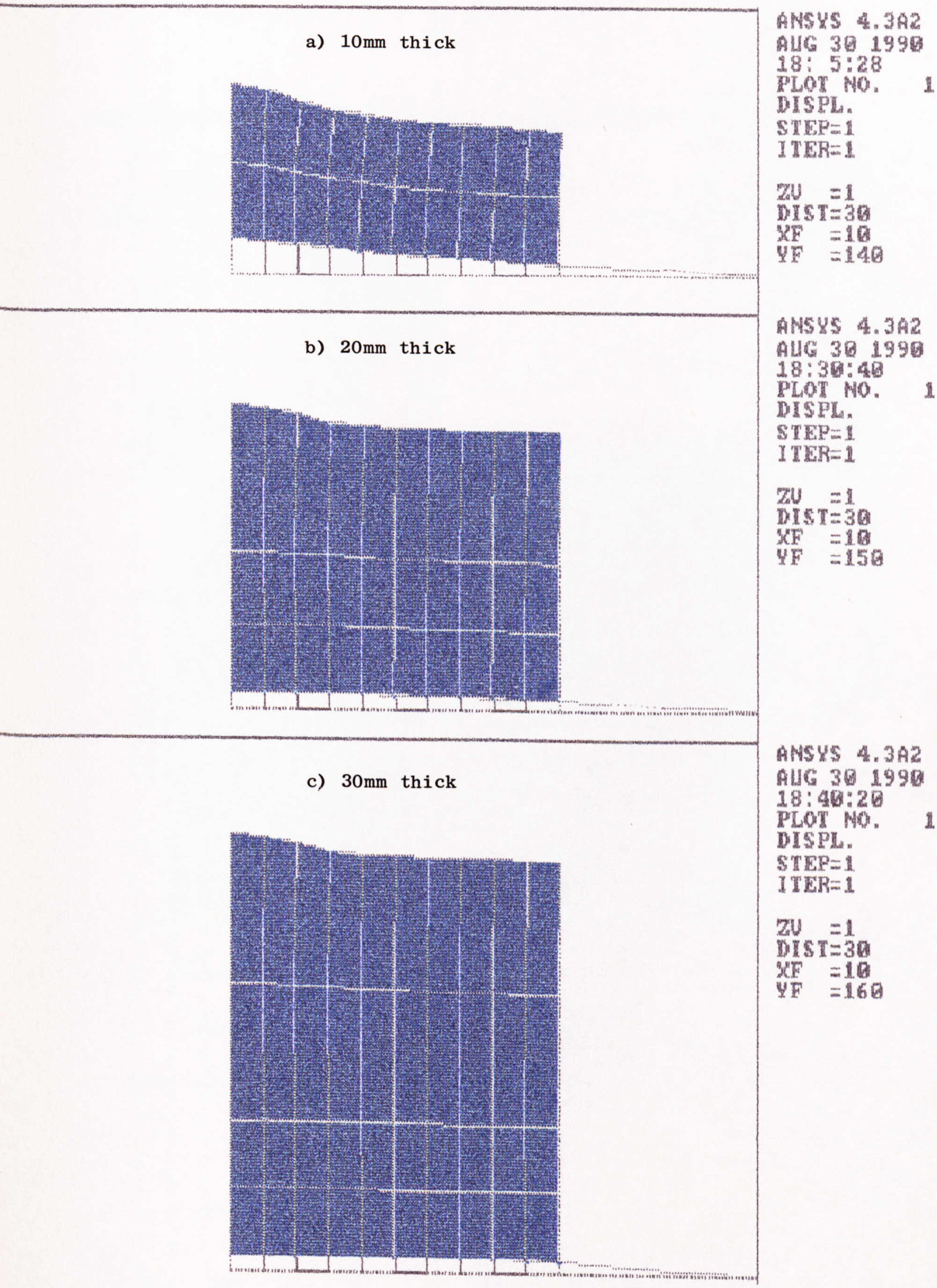
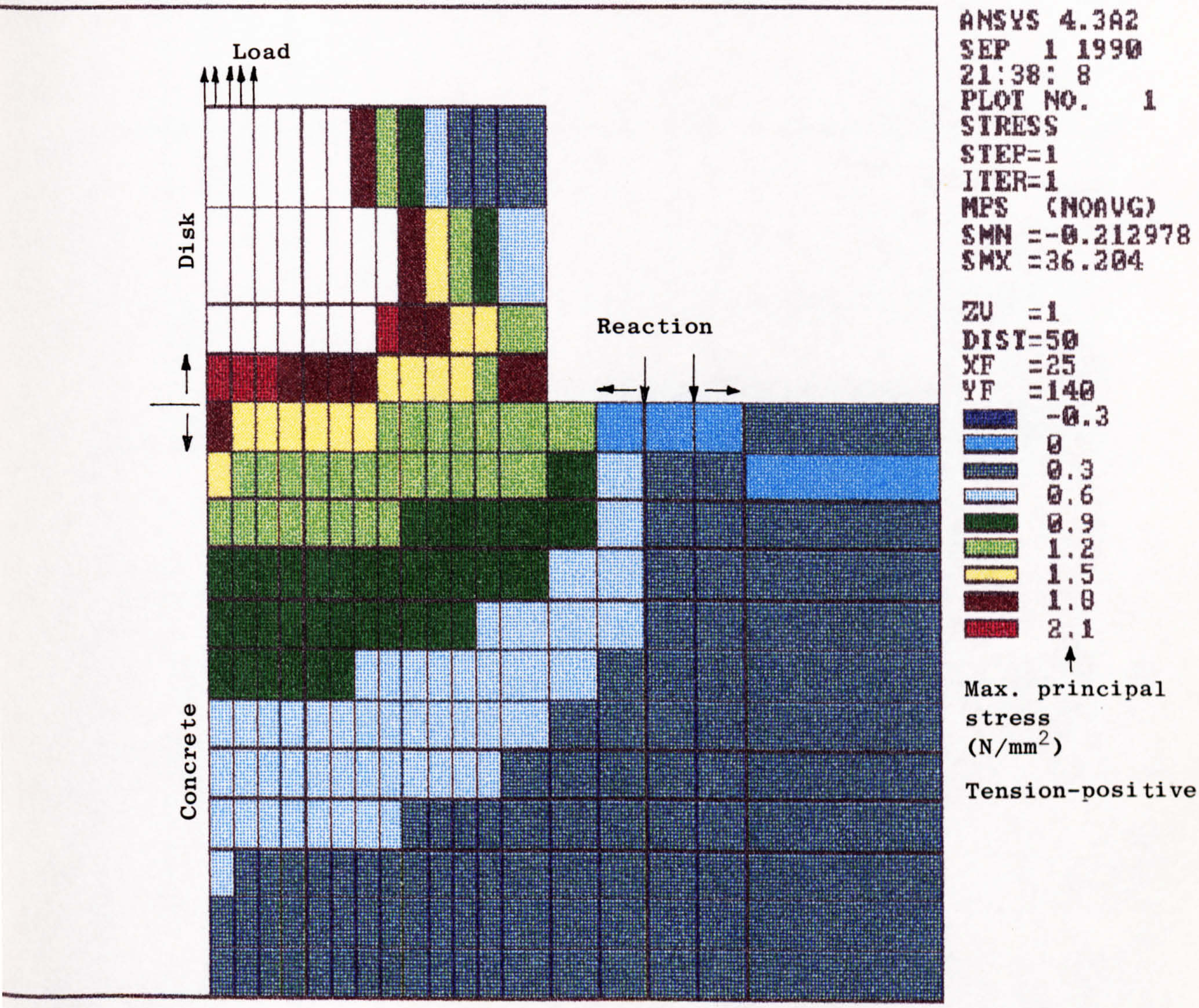
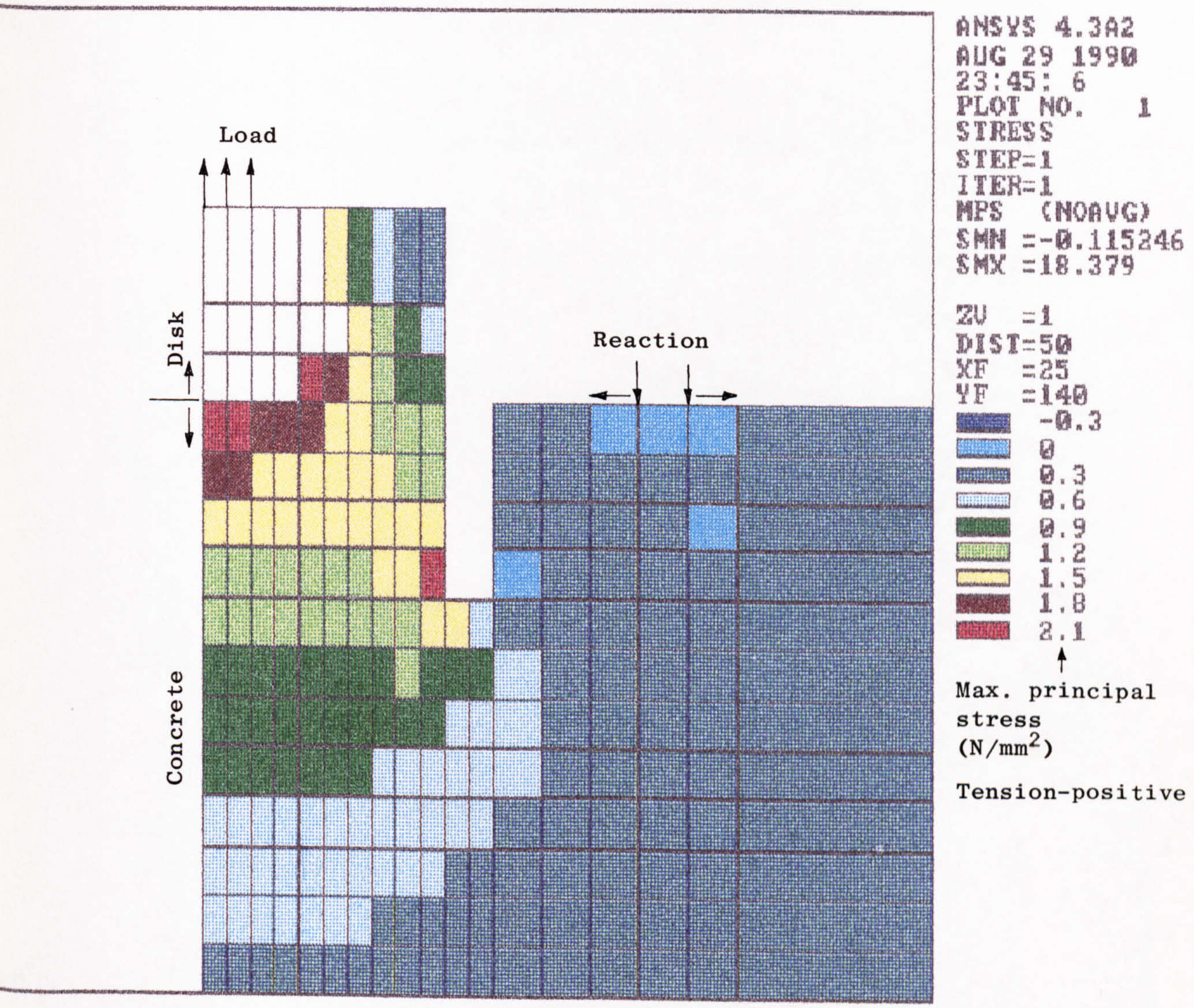


Figure 7.19: Displacement of aluminium disk with 50mm diameter using different thicknesses



N.B. The stress level in the elements coloured white is greater than 2.1 N/mm^2

Figure 7.20: Maximum principal average element stress distribution for surface pull off test on normal weight concrete using aluminium disk with 70mm diameter x 30mm thick



N.B. The stress level in the elements coloured white is greater than 2.1 N/mm^2

Figure 7.21: Maximum principal average element stress distribution for partial cored pull off test on Pellite concrete

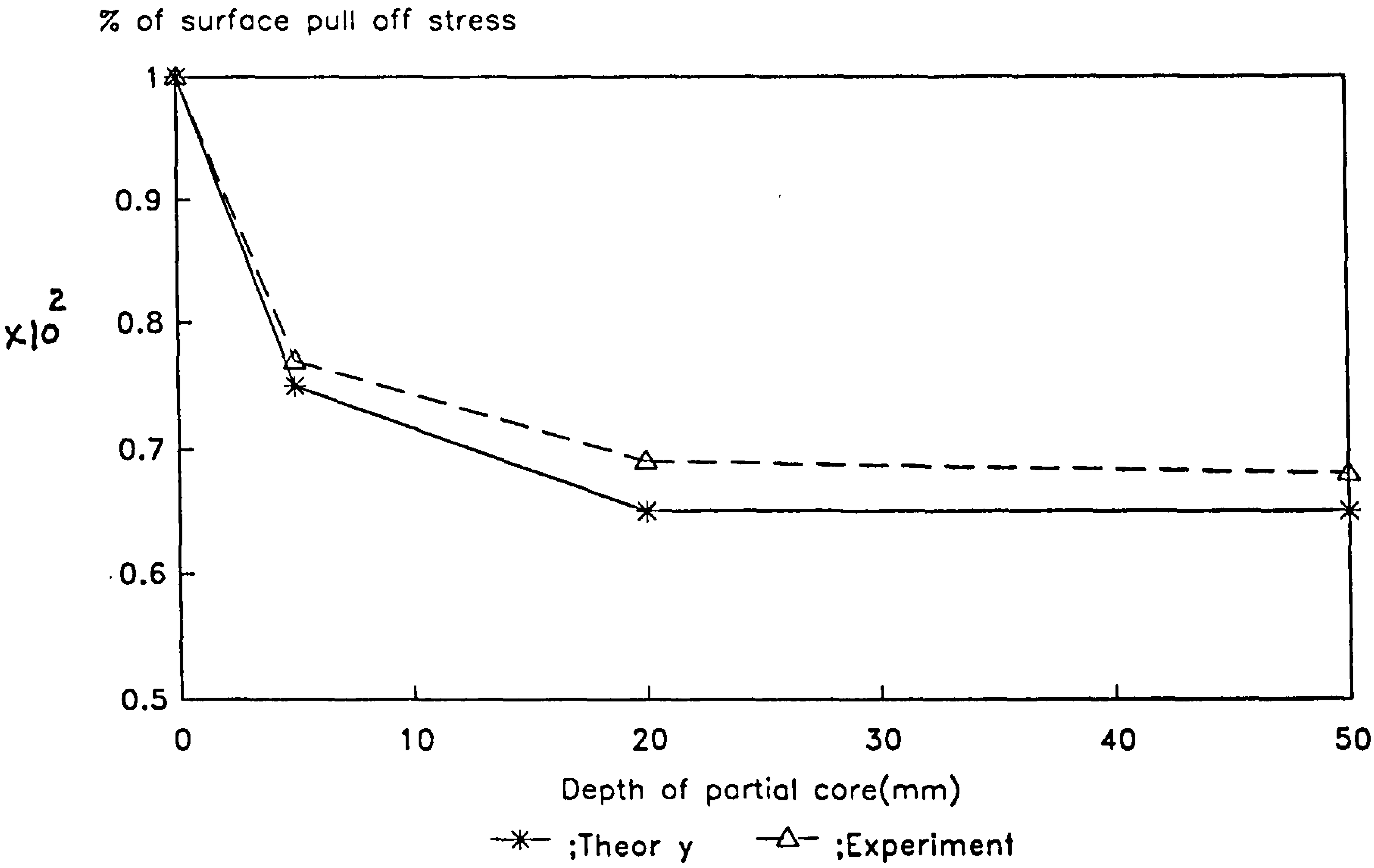


Figure 7.22:Variation of partial cored pull off stress with depth in fully Lytag concrete

CHAPTER 8. CONCRETE STRENGTH AND ITS VARIATION IN BEAMS

8.1 INTRODUCTION

An investigation was made on full-sized beams to study the strength of insitu concrete and its variation within typical structural elements. Experiments were carried out on five different types of concrete covering a range of lightweight concretes and normal weight concrete for comparison. A selection of insitu testing techniques were used to assess insitu concrete quality. Full details of the experimental programme have been given in Chapter four.

8.2 NATURE OF INSITU CONCRETE VARIABILITY

There have been numerous investigations in the past regarding evaluation of the quality of concrete in full-scale structural elements, including Petersons (1971), Maynard and Davis (1974), Bungey (1981b) and Murray and Long (1987). These investigations were mainly on normal weight concrete. Two important features observed in these investigations are

i) There was a variation in the strength of concrete in the structural elements across the height with the weakest concrete at the top.

ii) The strength of concrete in the structural element was generally lower than the strength of control test specimens.

The causes of the differences between the strengths of concrete in a structure and control specimens are principally due to differences in compaction and curing. These factors, as well as lack of uniformity of concrete supply, are also likely to influence strength variations across the depth of the elements.

Variations in concrete supply will be due to differences in materials, batching, transportation and handling techniques. These are usually controlled by the quality control of production, and compliance testing of control specimens, and are obviously not related to the element type involved.

Compaction and curing differences are two important factors affecting the strength of concrete in a structure and are related to member types and location within the member. For instance, the lower strength in top portions of columns is believed to be related to compaction effects and the process of water gain, which is due to the tendency of water to rise and the aggregates to sink to the lowest position. This will cause an increased water/cement ratio in upper zones and Gikley (1927) found evidence of a distinct porosity in such zones caused by the accumulation of air and water bubbles, which rose as far as the underside of the aggregates and were blocked there. However in slabs, the strength differences are thought to be dominated by curing effects, since top surfaces of slabs are generally more exposed to atmospheric conditions and vulnerable to poor curing. Figure 8.1 shows typical relative strength variations across the height according to member type (Bungey, 1989).

8.3 OUTLINE OF TESTS

Five large scale beams (2.2x0.3x0.5m) along with control cube specimens were cast, one from each of the different types of concrete. Full details of casting and curing procedures were given in Chapter four. As already indicated in Chapter four, particular attempts were made to minimize variations in concrete supply by weighing the materials the day before casting, and dry (fine and coarse) aggregates were used for mixing. Each beam was manufactured from five batches of concrete and consistency of the first concrete batch with the remaining batches was achieved as described in Section 4.4. However these facilities may not be available under site conditions and higher variations might be expected than those found in this investigation.

Non-destructive tests (rebound hammer and pulse velocity) and the partially destructive Lok-Test were used to assess the concrete beams, mainly at the age of 28 days. The layout of test positions has been given previously in Figures 4.15 to 4.17 and was designed to provide measurements at different levels within the member depth. 28 day cube compressive strengths were measured from fifteen control cube specimens cast (three cubes for each batch) from the same concrete mix used in each beam and subjected to similar curing. A limited number of tests were also carried out at the age of six months on fully Lytag concrete only. These were Pulse velocity tests, Capo-Tests and 50mm core tests and were performed as shown in Figure 4.18. Each core passed horizontally completely through the beam and the central 200mm portion was split into two specimens which, after capping, gave a total length of 100mm each. At the locations of A-10 and

E-10 (Figure 4.18) the exterior portions of cores from both ends were also tested with overall length of 50mm after capping to compare the exterior and interior strength of concrete in the beam. In all cases, the core tests were performed in a dry state.

8.4 PRESENTATION OF TEST RESULTS

All the individual insitu test results are presented in Appendix D (Table D-1 to D-11), and 28 day control cube compressive strength measurements are shown in Table 8.1. Insitu equivalent cube strengths in the beams were predicted from the insitu tests using suitable calibrations. For 28 day non-destructive tests, calibrations were built up from testing, at various ages, on cubes cast from one batch of the same mix used in the beam (see Tables 6.1 and 6.5). For six month tests, the pulse velocity results were correlated with the measured core strengths ($\ell/d = 2.0$) to yield the following exponential function

$$f_{cc} = 2.65 \times 10^{-2} e^{2.04V} \qquad (8.1)$$

The calculated equivalent core strengths were subsequently converted to equivalent cube strength using the relevant regression equation given in Table 6.16. For partially destructive tests, the Lok force results were converted to equivalent cube strength using pre-determined calibrations for lightweight concretes given in Table 6.7 or established calibration for normal weight concrete using Equation 3.2. Lok-Test calibrations were also used for predicting equivalent cube strengths from Capo-Test measurements, as already reported to be applicable (see

Section 3.3.3.2). For destructive tests, the 50mm core test results with $\ell/d = 2.0$ were used to estimate equivalent cube strength from pre-determined calibrations given in Table 6.16. Summaries of insitu test results along with average equivalent cube strengths at each level are listed in Table 8.2 and 8.3. Statistical variation analyses of insitu tests based on coefficient of variation have been summarized in Table 8.4. Estimations of population mean concrete strength in the beams within 95% confidence limits are presented in Table 8.5. Figures 8.2 to 8.6 show the variation in strength throughout the height of the beams based on the three different insitu tests. Relative strength distributions within the body of the concrete beams for different types of concrete are plotted in Figures 8.7a-e on the basis of pulse velocity test results.

8.5 ANALYSIS AND DISCUSSION OF TEST RESULTS

It is apparent from the summarized results given in Tables 8.2 and 8.3 that the strength of concrete in the beams generally varied across the height with lowest value at the top. The average concrete strength obtained at mid-height was typically shown to be the same as the overall average strength in the beam. This suggests that limited numbers of insitu tests on structures, should be accomplished at the mid-height to assess the average concrete strength. This confirms the findings of Munday and Dhir (1984).

Variations in strength across the depths of the beams shown in Figures 8.2 to 8.6 for 28 day old concrete, indicate that trends based on pulse velocity and Lok-Test results are generally similar. The only exception was for fully Lytag concrete which is discussed later in this section. The strengths predicted from rebound hammer tests, however, are relatively higher than those from the other tests, and in addition, indicated strength variation across the depth did not always follow the same patterns. This is most noticeable for fully Lytag concrete (Figure 8.2) and Leca concrete (Figure 8.4). The higher strengths predicted by the rebound hammer may be coupled with mould type, the latter being different than the steel moulds used for acquiring the calibration. This has also been observed elsewhere (Bellander, 1979), (Bungey, 1989). The difficulties encountered in detecting strength changes across the depth of some members by rebound hammer may possibly be related to the nature of this test method, as this is purely a surface test. This is unlike the pulse velocity test which is affected by the interior body of the concrete, or the pull out test which is affected by the concrete up to 25mm below the surface. In view of these uncertainties, the rebound hammer test results have been ignored in all subsequent analyses.

In an attempt to detect differences in concrete strength distribution between beams of different concretes, relative strength contours based on pulse velocity tests were compared (see Figures 8.7a-e). The influence of concrete type on the homogeneity is apparent. Fully Lytag concrete shows the least variation in strength development with average strength variation between top and bottom layers of only approximately 10%. Pellite concrete was found to be the second most homogenous concrete with an average strength variation between top and bottom layers of about 20%. In semi Lytag concrete high variation in strength (approximately 35%) was detected across the depth of the beam. One possible explanation for this might be due to particularly high bleeding which was observed after this concrete had been compacted. In Leca concrete, strength distribution surprisingly came out to be similar to semi Lytag and normal weight concretes. This might be due to the large mass of concrete and the use of poker for compaction, as opposed to the vibration table used for cubes, which may cause the concrete to suffer less from seggregation. It must be remembered also that the test results given here are taken from side faces, and the results for the bottom layer were taken 100mm above the base of the beam. Figure 8.8 shows concrete cores drilled vertically from each of the beams which illustrate typical aggregate distributions within the matrix. For Leca concrete, a more uniform distribution of aggregate can be seen than that previously obtained in cylinder specimens (Figure 6.2).

During the testing on the fully Lytag concrete beam, shrinkage cracks appeared on all faces of the concrete beam. These shrinkage cracks may have played a part in lower pull out resistance which led to estimated equivalent cube strengths approximately 20% lower than those obtained from pulse velocity tests. This discrepancy in strength estimation was further examined on the fully Lytag concrete beam at the age of six months using the Capo-Test along with pulse velocity and core tests. Once again from the results shown in Tables D-11 and 8.3, similar disparity was derived on cube strength estimation from surface zone Capo-Tests compared to those related to the interior zone of concrete namely pulse velocity and core tests. It also needs to be mentioned that during some of the Capo-Tests, splitting of the Capo ring insert occurred which raises some doubts as to the reliability of measured pull out forces. To try to account for this problem, additional tests were accomplished by testing some cores (50mm in length) from the exterior portion of the concrete beam as described earlier in Section 8.3 and results are tabulated in Table D-11. After conversion of core strength from $\ell/d=1.0$ to $\ell/d=2.0$ from correction factors given in Table 6.14, results indicate that concrete in the exterior zone is 19.6% on average weaker than the interior. This finding may confirm the lower cube strength predicted from the surface zone tests of Lok-Test and Capo-Test.

8.5.1 Variability of Insitu Test Measurements on Beams

The variability of insitu test measurements may be assessed from the percentage coefficient of variation and this is shown in Table 8.4 for pulse velocity and Lok-Test. The variation analyses were carried out at different levels and the results based on top, middle and bottom layer are presented in the above table for different types of concrete.

The measured coefficients of variation based on Lok-Test show that the variations are similar at different levels with the exception of semi Lytag and Leca concretes where high variations were found at the top level. The high variation at the top level of semi Lytag concrete may be due to the formation of water channels created by the excessive bleeding which was observed at the surface of the concrete beam in contact with the mould. this was predominant at the top layer of the beam. In Leca concrete, the relatively greater accumulation of aggregate at the top and the major differences in physical characteristics between the aggregate and the matrix may be responsible. The coefficients of variation at the top level of the two latter types of concrete were higher than those anticipated from cube specimens reported in Section 6.3.2.3. However, in the remaining cases, shown in Table 8.4 the coefficients of variation of Lok-Test results are of the same order as those obtained with cube specimens.

The pulse velocity test results shown in Table 8.4 indicate that the coefficients of variation in many cases are not significantly different at various levels. The interesting observation that can be seen from Table 8.4 is that in most cases the coefficients of variation for pulse velocity are less than those obtained from cube specimens given in Section 6.2.2.3. This seems to be related to the path length, which was 300mm (taken across the width of the beam) as opposed to 100mm in the cube specimens. Clearly, heterogeneity in concrete causes variations between measurements of the transit time through different paths of identical length. Facaoaru (1968) showed however that the extent of variation depends upon the distance travelled by the pulse in relation to the size of the inhomogeneities. He noticed that as the path length increases the coefficient of variation decreases tending to a limiting value and may explain the observed phenomenon.

Overall coefficients of variation of pulse velocity and Lok-Test measurements have also been given in Table 8.4 for different types of concrete. The results were found to be considerably higher than those given at each level as expected. This is reflected by the strength variations across the depths of beams which have been discussed earlier in this chapter.

8.5.2 Prediction of the Population Mean Concrete Strength

Only Lok-Test results have been considered for this analysis since this is regarded as the most practicable insitu strength assessment technique of those used on the beam specimens. In practice, normally, a limited number of samples from a population is available and there is a need to transform statistical characteristics of the sample into statistical characteristics of the population. Additionally, whenever statistical characteristics of the sample are obtained by an indirect method, as is the case with evaluation of concrete strength through partially or non-destructive tests, these transformations must also take into account the correlation of indirect methods with direct observations. Here an approach to the estimation of mean concrete strength in the beams within prescribed confidence limits is presented.

The relationship between the sample and population mean is given by the theory of confidence limits (Kennedy and Neville, 1976)

$$\mu = \overline{x} \pm t_{\alpha,m} \frac{\overline{\sigma}}{\sqrt{n}} \tag{8.2}$$

where

μ = population mean strength

$\bar{x}$ = sample mean strength

$\bar{\sigma}$ = sample standard deviation

n = number of observation

$t_{\alpha,m}$ = random variable of the student's t distribution at confidence limit α for a sample with m=n-1 degrees of freedom.

In Equation 8.2, $\bar{\sigma}$ can be substituted by $\bar{v}$ (sample coefficient of variation) knowing;

$$\bar{v} = \frac{\bar{\sigma}}{\bar{x}} \tag{8.3}$$

hence

$$\mu = \bar{x}\left(1 \pm t_{\alpha,m}\frac{\bar{v}}{\sqrt{n}}\right) \tag{8.4}$$

In Equation 8.4 it is assumed that the sample mean strengths are closely distributed with a normal curve. Hindo and Bergstrom (1985) stated that for concrete with excellent quality control, the strength distribution can be assumed as a normal distribution. For this study the beams were cast and cured under laboratory conditions and the low coefficients of variation of control cube specimens, shown in Table 8.1 suggest that it may be reasonable to assume a normal distribution for concrete strength.

The mean concrete strength of the population is calculated by Equation 8.4 at 95% confidence limits ($t_{95\%,29}$ = 2.05). Calculation to predict the population mean strength for semi Lytag concrete is shown below and similar procedures were adopted for other types of concrete with results shown in Table 8.5.

By reference to Equation 8.4, $\bar{v}$, sample coefficient of variation of concrete strength is not known, but it may be calculated from the coefficient of variation of Lok-Test results using an approach described elsewhere (Murray, 1984).

In an attempt to determine how much of the variation in Lok-Test values was due to the method of testing, results from the tests on semi Lytag concrete described in Section 6.3.2.3 were examined. It was shown that the coefficient of variation for Lok-Test values, from cubes prepared from a single batch of concrete, was approximately 7%.

It was decided that the variation in concrete quality could be approximated by determining the variation between measured compressive strength of cubes. The coefficient of variation of measured cube strengths from a single batch of the concrete used for the Lok-Test, was determined to be 3.5%. It needs to be indicated however that the cube strength measurements were carried out on 100mm cube specimens as were different from those 200mm cube specimens used for the Lok-Test. This was because of the limited number of 200mm cube specimens available and it was assumed that the variation in coefficient of variation would not be critical.

The coefficient of variation of Lok-Test result, v_L, due to factors inherent in the method, was calculated using the equation

$$v_T = \sqrt{v_L^2 + v_C^2} \tag{8.5}$$

where

v_T = the coefficient of variation of Lok-Test results due to factors inherent in the method and concrete variability = 7% (Table D-12)

v_C = the coefficient of variation of the strength of sample cubes = 3.5% (Table D-13)

The coefficient of variation of Lok-Test results due to factors inherent in the method was thus estimated to be;

$$v_L = \sqrt{7.0^2 - 3.5^2}$$

$$v_L = 6\%$$

Having obtained the coefficient of variation inherent in the Lok-Test as 6% and knowing the total variation measured from Lok-Test results on large scale beam, as 18.2% using semi Lytag concrete, the sample coefficient of variation of concrete strength in the beam may be predicted from Equation 8.5 as

$$\bar{v}_C = \sqrt{18.2^2 - 6^2}$$

$$\bar{v}_C = 17.2\%$$

Knowing the sample mean value of the compressive strength of semi Lytag concrete as 34.2N/mm^2 based on Lok-Test results (Table 8.2) and the calibration given in Table 6.7, the population mean semi Lytag concrete strength could be calculated from Equation 8.4;

$$\mu = 34.2\left[1 \pm 2.05 \frac{0.172}{\sqrt{30}}\right]$$

Accepting the lower limit value of μ;

$$\mu_L = 34.2 - 2.2 = 32 \text{ N/mm}^2$$

Corresponding values for population mean equivalent concrete cube strength for other types of concrete based on Lok-Test results are summarized in Table 8.5.

For measured cube strengths given in Table 8.1 for different types of concrete, the population mean cube strengths can be predicted from Equation 8.4 knowing the sample cube strengths and their coefficients of variation given in Table 8.1. The population mean cube strengths based on lower limits are presented in Table 8.5 for different types of concrete.

From Table 8.5, it can be seen that the concrete strengths in beams (equivalent cube strengths) are relatively smaller than the cube strengths and this has been observed by other investigators as described earlier in Section 8.2. Table 8.5 shows the ratio of the cube strength divided by the concrete strength in beam for various types of concrete. In Leca concrete, the reduction in concrete strength in the beam was found to be smaller than for other types of concrete. This may be due to the fact that the cubes had reached the ceiling strength for the material thus indicating an apparently lower reduction in concrete strength in the beam. Apart from Leca concrete, the percentage reduction of concrete strength in beams are shown to be nearly similar in lightweight and normal weight concretes. This confirms that the partial materials safety for concrete strength, γ_m=1.5, as defined in BS 8110 : Part 1 (1985), which expresses the ratio of standard cube strength to that in the member, is suitable for use with lightweight concretes as well as normal weight concrete. The ratio given in the present investigation is however shown to be smaller than γ_m=1.5. This is probably related to a number of factors. Firstly, the ratio used in this investigation was based on average strength rather than characteristic strength used in γ_m. Secondly, dry curing was used for cube specimens instead of standard curing specified in γ_m. Finally, the possible higher variation in concrete supply used on site due to batching error and variation in mixing, since efforts have been made to deliberately minimized these effects in this investigation as described earlier in Section 8.3.

8.6 SIGNIFICANCE OF TEST RESULTS

Regarding the insitu tests applied on different types of concrete beam, the rebound hammer tests were shown to over-estimate the concrete strengths. Other insitu tests used on the beams were satisfactory as in most cases similar predictions in concrete strengths resulted. The pull out tests and core tests taken from exterior zone with 50mm length on fully Lytag concrete resulted in a strength prediction which was about 20% lower than those suggested by pulse velocity and core tests taken from interior body of concrete. This is believed to be linked with shrinkage cracks as detected on the surface of concrete.

There was found to be a tendency for the compressive strength of concrete in beams to increase with increasing distance from the top. This tendency however was found to be dependent on the concrete type. Among concretes under investigation, fully Lytag concrete was found to be the most homogenous with least variation in strength across the depth (approximately 10% reduction in strength at the top with respect to the bottom). The next most homogenous concrete was found to be Pellite concrete with about 20% strength reduction at the top compared to the bottom. In semi Lytag, Leca and normal weight concretes strength reduction at the top came out to be around 35% as compared to concrete strength at the bottom. From the results on lightweight concrete, it may be concluded that in some cases the planning and interpretation of insitu investigations may not necessarily be the same as used for normal weight concrete.

The coefficients of variation of insitu test results based on pulse velocity and Lok-Test at each level of beams were found to be similar as those obtained on the cube specimens or even lower in the case of pulse velocity test results. Overall coefficients of variation of these test results were shown to be significantly higher than those obtained at each level, but lowest for fully Lytag concrete.

Comparison of the ratio of population mean concrete strength in cubes divided by the corresponding strength in beams (based on the Lok-Test results) showed that this ratio is mostly similar in lightweight and normal weight concretes, and this confirms the suitability of $\gamma_m = 1.5$ given in BS 8110 : Part 1 (1985) for lightweight concretes as well as normal weight concrete.

Table 8.1:Control cube strength results (cured the same as beams)

Batch No.	28 days 100 mm Cube Compressive Strength (N/mm^2)				
	Fully Lytag Concrete	Semi Lytag Concrete	Leca Concrete	Pellite Concrete	Normal Weight Concrete
1	43.1 43.8 44.1	43.8 42.8 42.9	29.7 24.0 26.0	47.6 49.5 48.4	40.7 38.7 40.0
2	44.0 43.5 43.9	41.8 41.8 38.7	26.7 29.2 25.7	47.0 48.1 49.5	38.5 41.2 39.7
3	46.3 45.3 46.6	41.4 42.9 44.5	27.8 29.2 30.7	48.1 47.9 47.8	42.2 38.5 39.4
4	43.4 44.7 43.5	41.5 41.9 40.8	27.6 27.5 27.7	48.6 45.8 48.8	42.3 40.7 43.0
5	43.2 41.8 45.9	41.3 40.0 37.3	29.2 32.7 29.4	47.1 45.0 43.9	38.0 40.5 39.0
Average	44.2	41.6	28.2	47.5	40.2
c.v(%)	3.0	4.5	7.7	3.3	3.8

Table 8.2:Estimated cube strengths based on insitu test methods at the age of 28 days

Concrete Type	Level	Rebound Hammer R	Rebound Hammer f_c N/mm^2	Pulse Velocity V Km/sec	Pulse Velocity f_c N/mm^2	Lok-Test L KN	Lok-Test f_c N/mm^2	% of Strength Reduction based on Rebound Hammer	% of Strength Reduction based on Pulse Velocity	% of Strength Reduction based on Lok-Test
Fully	Top	39.8	46.9	3.61	40.9	17.9	32.5	12.1*	7.7	12.4
Lytag	Mid	37.5	41.3	3.62	43.2	18.2	33.3	1.1	2.5	10.2
Concrete	Bottom	37.7	41.9	3.63	44.3	19.9	37.1	0	0	0
Average			43.4		42.0		34.3			
Semi	Top	30.8	32.2	3.76	26.3	15.2	27.5	25.6	36.9	33.7
Lytag	Mid	33.2	37.6	3.82	31.4	18.3	33.5	13.2	24.7	19.3
Concrete	Bottom	35.5	43.3	3.91	41.7	22.4	41.5	0	0	0
Average			37.7		32.3		34.2			
Leca	Top	25.5	28.6	3.23	19.6	11.4	20.0	4.3	29.7	34.4
Concrete	Mid	26.3	29.4	3.38	23.2	15.1	26.6	1.7	16.8	12.8
	Bottom	26.9	29.9	3.56	27.9	17.3	30.5	0	0	0
Average			29.3		23.4		25.7			
Pellite	Top	32.3	46.1	4.19	35.4	18.7	32.5	12.7	17.9	24.2
Concrete	Mid	32.7	47.5	4.21	36.7	20.4	35.8	10.0	14.8	16.6
	Bottom	34.1	52.8	4.28	43.1	24.0	42.9	0	0	0
Average			48.9		38.5		37.1			
Normal	Top	30.8	33.7	4.51	26.4	20.5	24.2	27.5	32.3	38.1
Weight	Mid	30.5	33.1	4.53	27.2	23.7	29.1	28.8	25.4	25.6
Concrete	Bottom	36.0	46.5	4.68	39.0	30.1	39.1	0	0	0
Average			37.8		30.2		30.8			

*:Increased in strength

Table 8.3:Estimated cube strength of fully Lytag concrete based on insitu test methods at the age of six months

Level	Pulse Velocity V Km/sec	Pulse Velocity f_c N/mm^2	Capo-Test C KN	Capo-Test f_c N/mm^2	Cores f_{cc} N/mm^2	Cores f_c N/mm^2	% of Strength Reduction based on Pulse Velocity	Capo-Test	Cores
Top	3.71	55.8	24.6	45.2	51.0	56.0	8.5	14.1	9.5
Mid	3.73	59.2	26.1	49.5	53.6	59.0	4.2	5.9	4.7
Bottom	3.75	61.9	27.2	52.6	56.0	61.9	0	0	0
Average		58.5		48.6		59.0			

Table 8.4:Coefficients of variation of insitu test methods on different types of concrete beam

Concrete Type	Level	Coefficient of Variation(%) Pulse Velocity	Lok-Test
Fully Lytag Concrete	Top Mid Bottom	0.4 0.7 0.7	8.9 8.0 6.9
Overal c.v		0.6	9.1
Semi Lytag Concrete	Top Mid Bottom	0.4 0.8 0.7	14.0 7.2 5.8
Overal c.v		1.5	18.2
Leca Concrete	Top Mid Bottom	1.0 1.6 1.2	16.3 9.7 8.8
Overal c.v		3.6	19.9
Pellite Concrete	Top Mid Bottom	0.6 0.8 0.3	9.5 6.2 8.8
Overal c.v		1.0	13.4
Normal Weight Concrete	Top Mid Bottom	1.4 1.1 0.9	8.4 11.6 8.8
Overal c.v		2.1	18.9

Table 8.5:Predictions of population concrete strengths

Concrete Type	Lower Limit Population Concrete Strength(N/mm^2) Concrete Cube A	Concrete Beam B	A/B
Fully Lytag Concrete	40.6	32.5	1.25
Semi Lytag Concrete	43.5	33.4	1.30
Leca Concrete	27.0	23.7	1.14
Pellite Concrete	46.6	35.4	1.32
Normal Weight Concrete	39.4	28.7	1.37

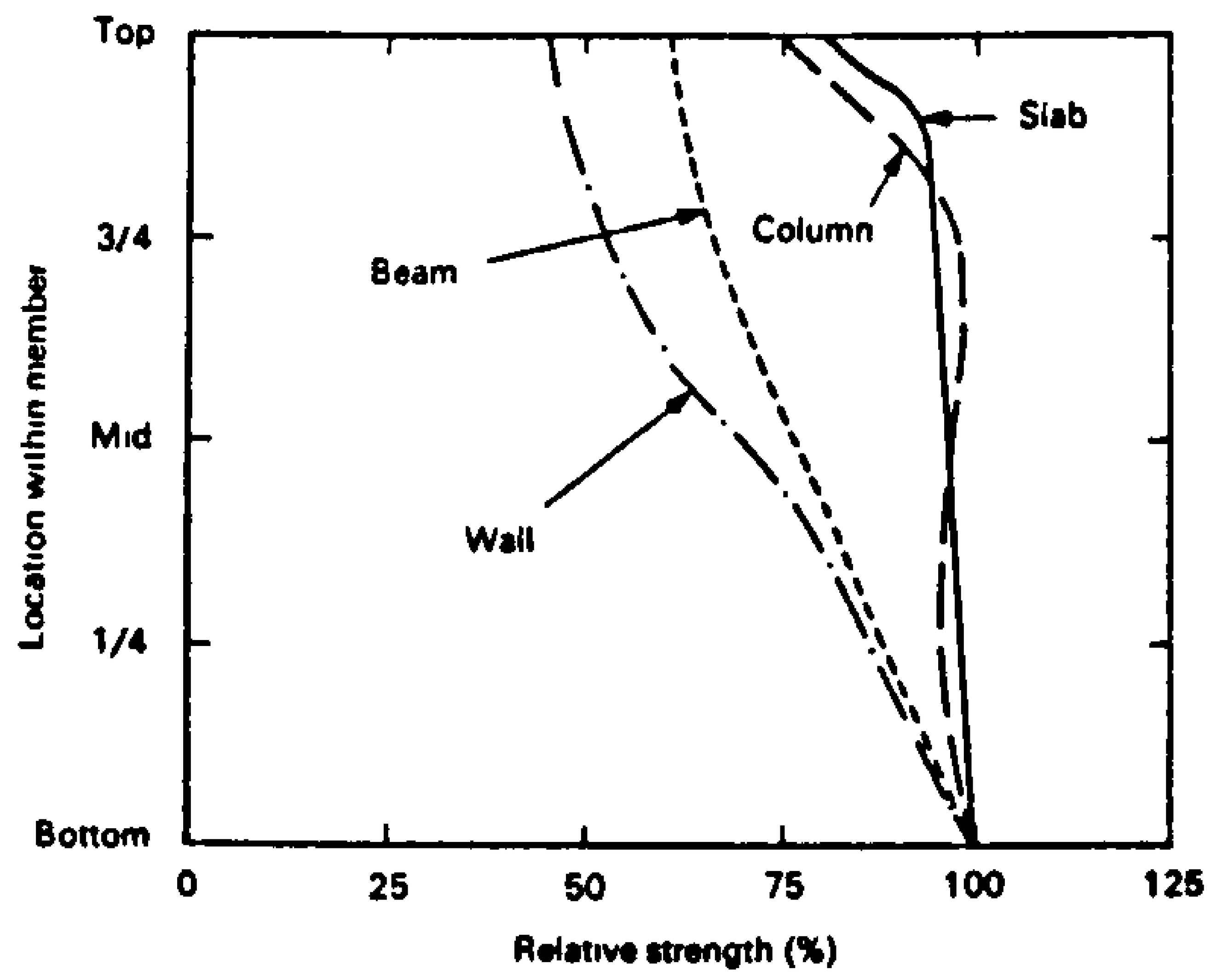


Figure8.1:Within member variations (Bungey,1989)

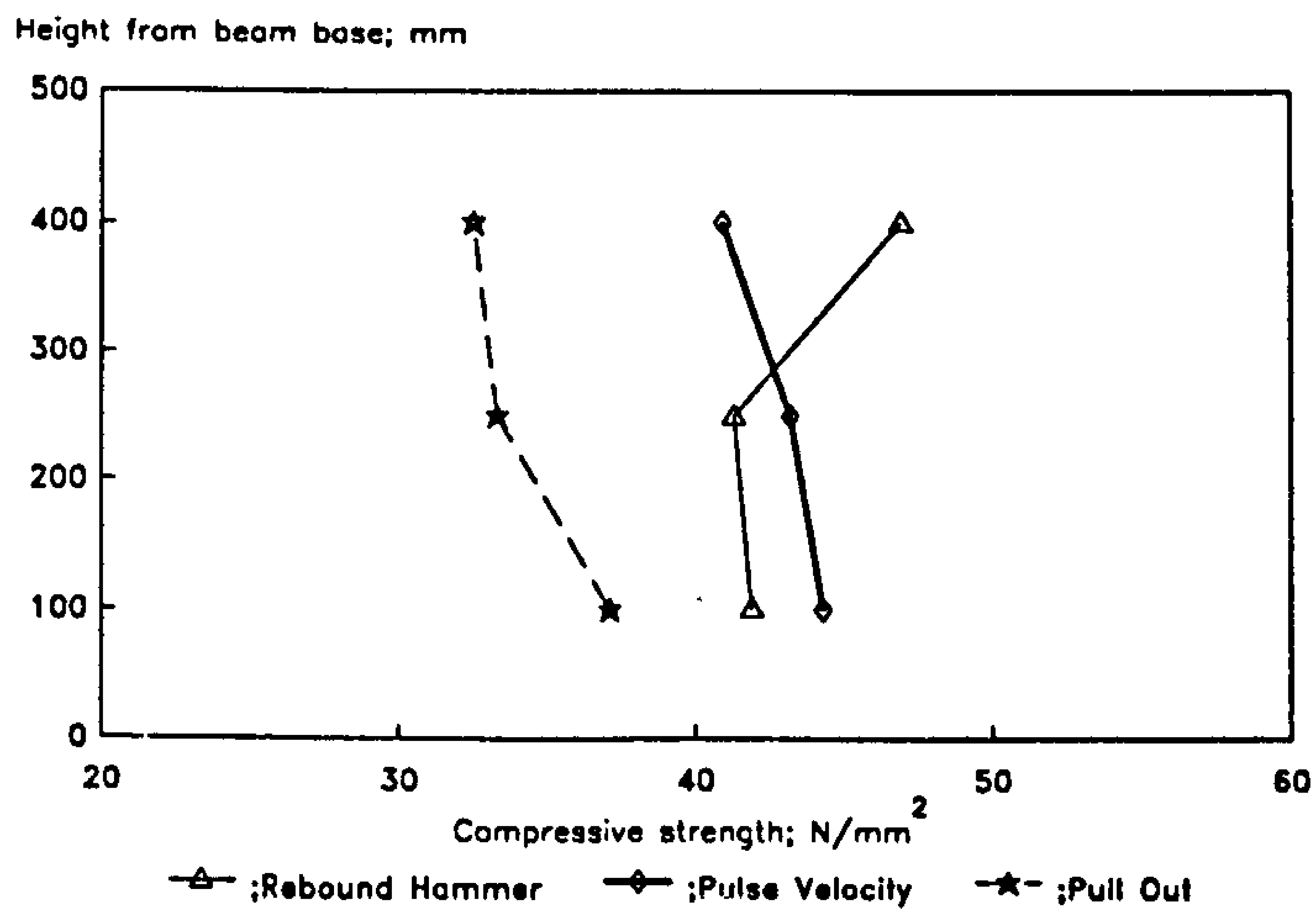


Figure 8.2:Variation in strength through the height of beam made from fully Lytag concrete

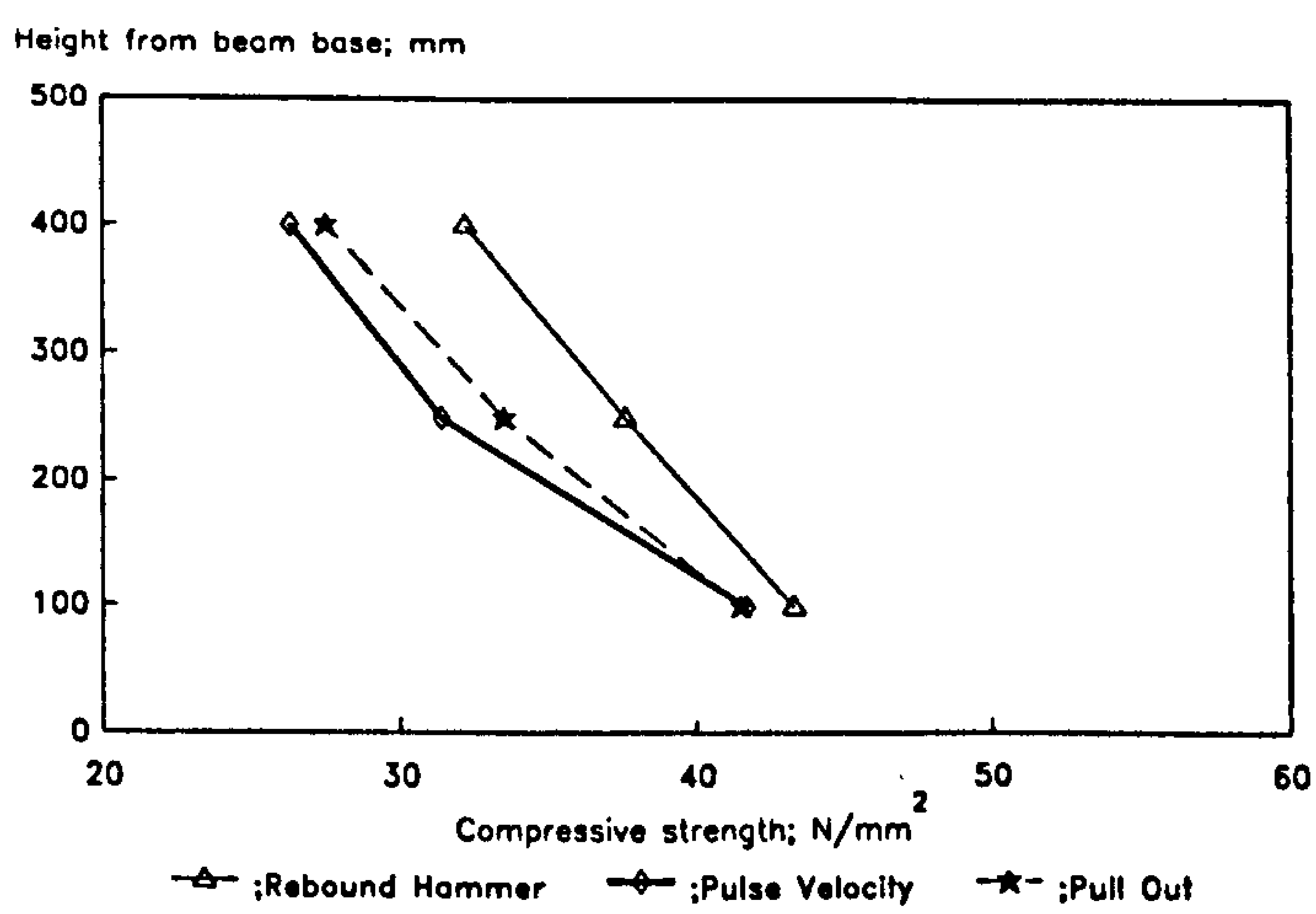


Figure 8.3:Variation in strength through the height of beam made from semi Lytag concrete

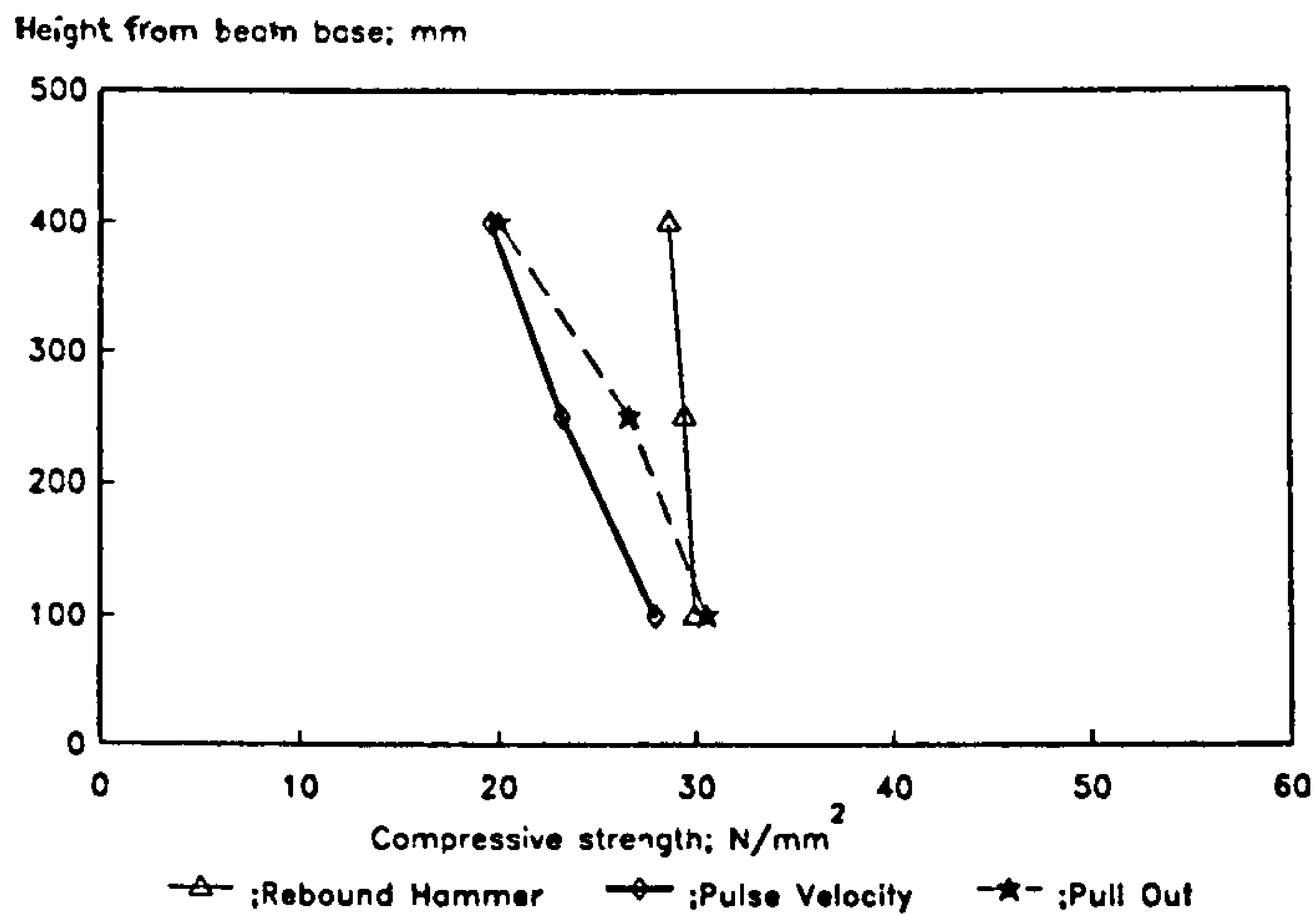


Figure 8.4:Variation in strength through the height of beam made from Lecc concrete

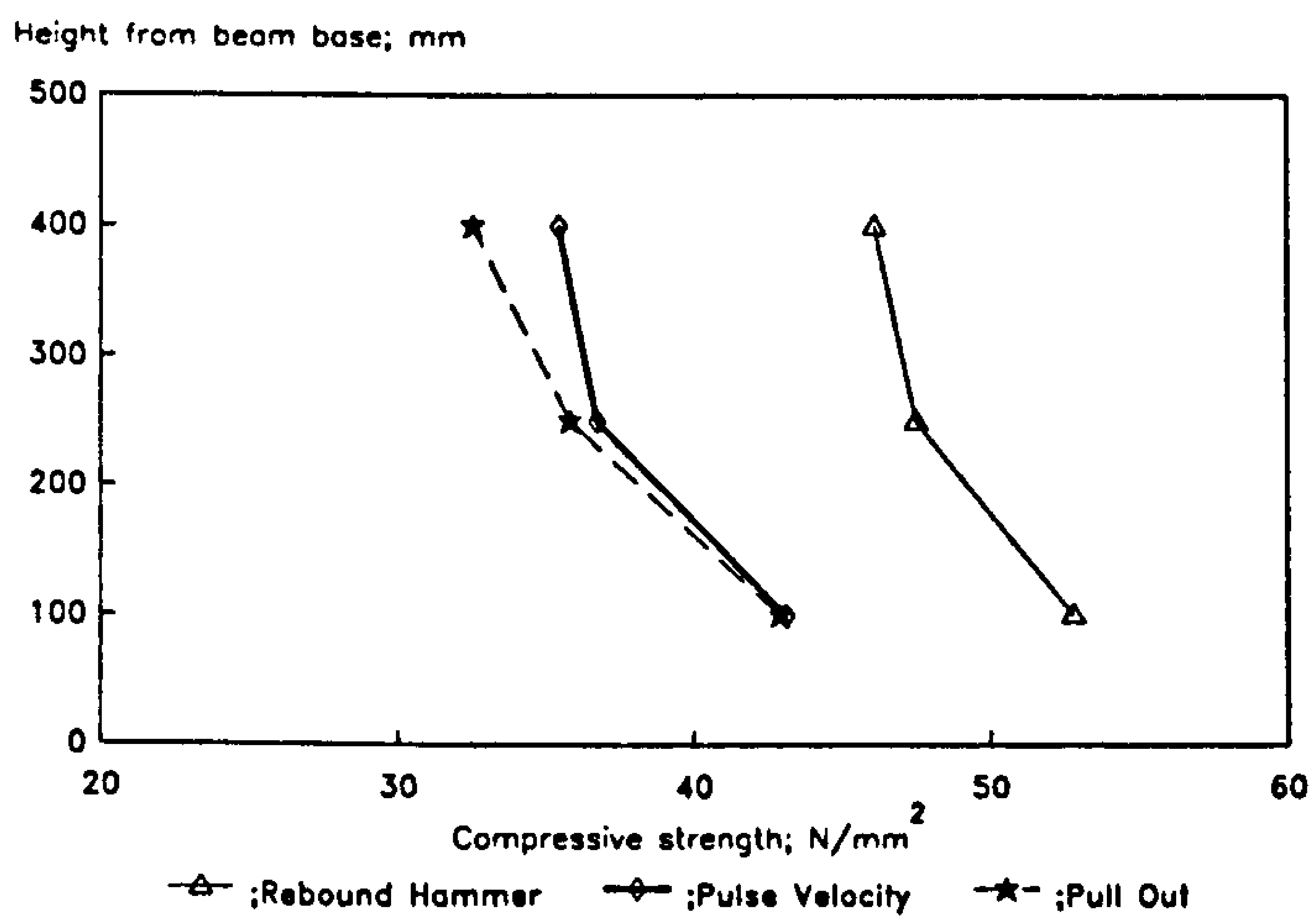


Figure 8.5:Variation in strength through the height of beam made from Pellite concrete

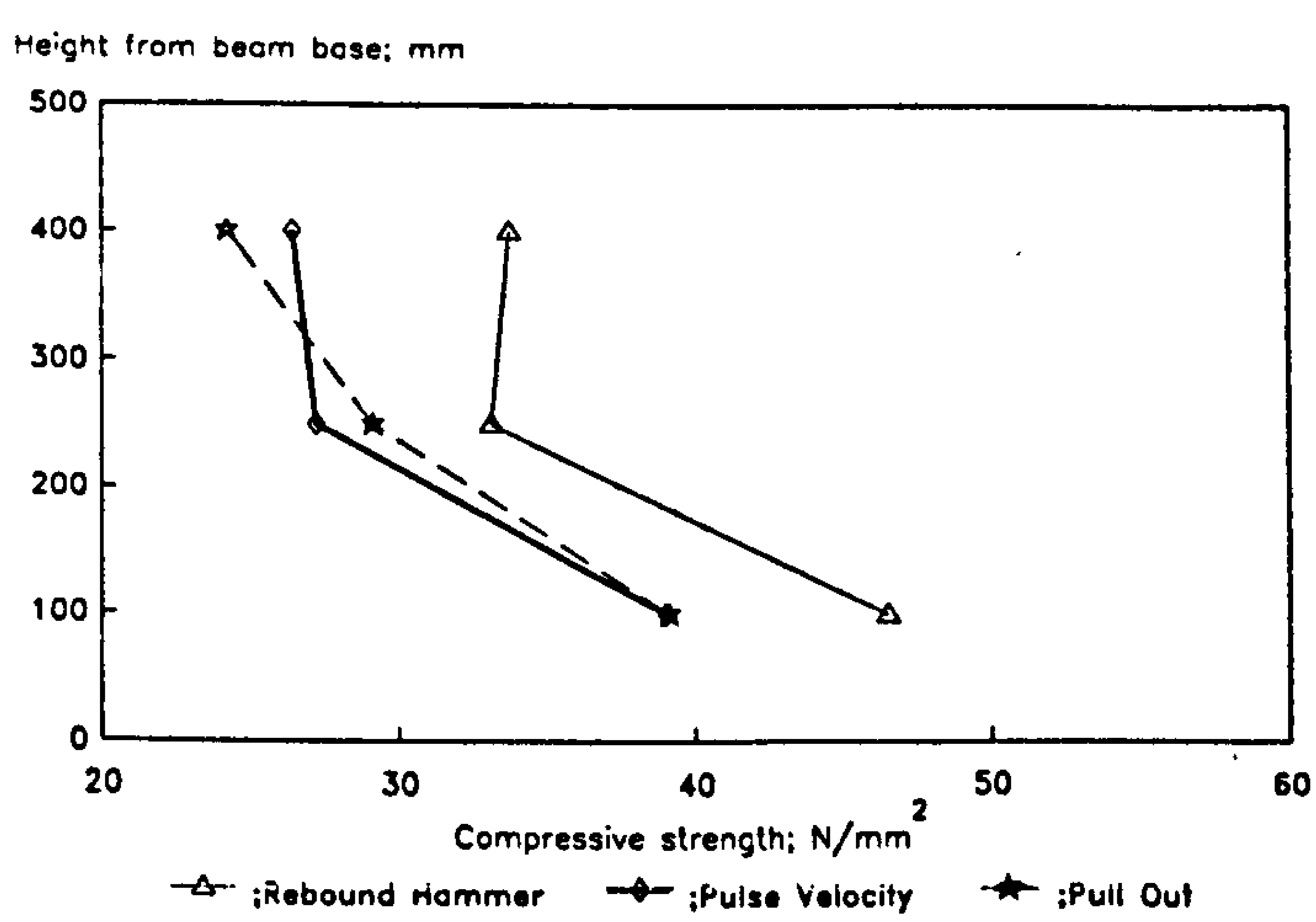
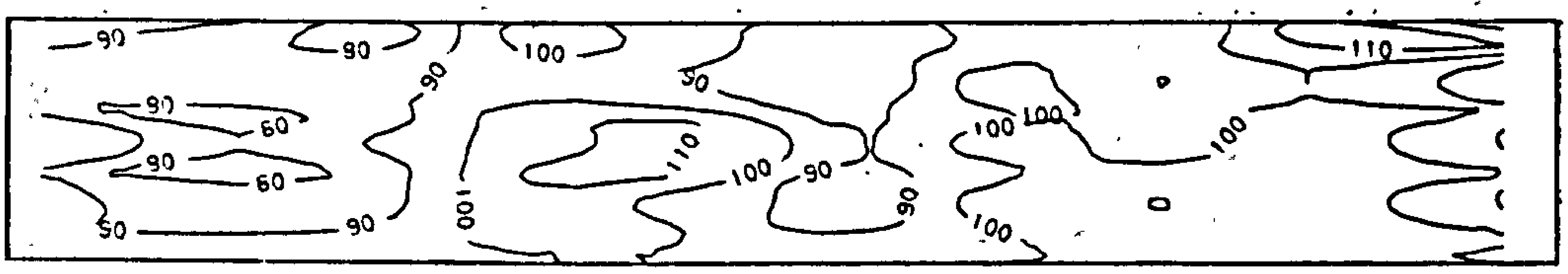
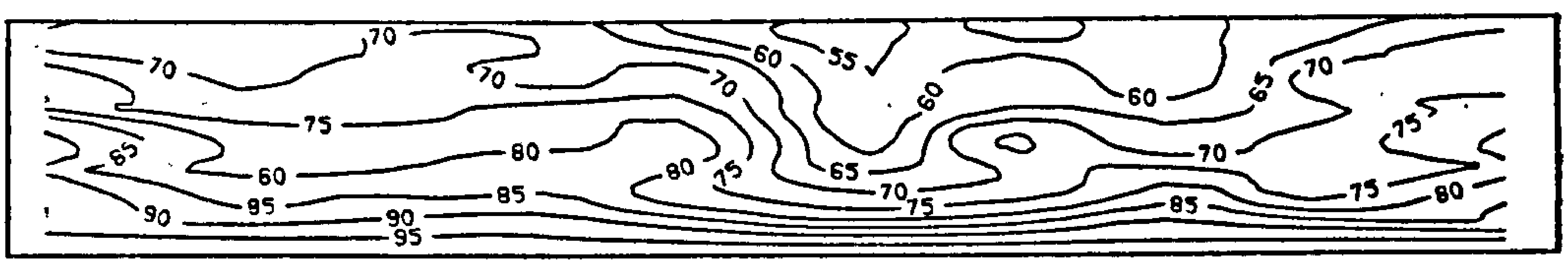


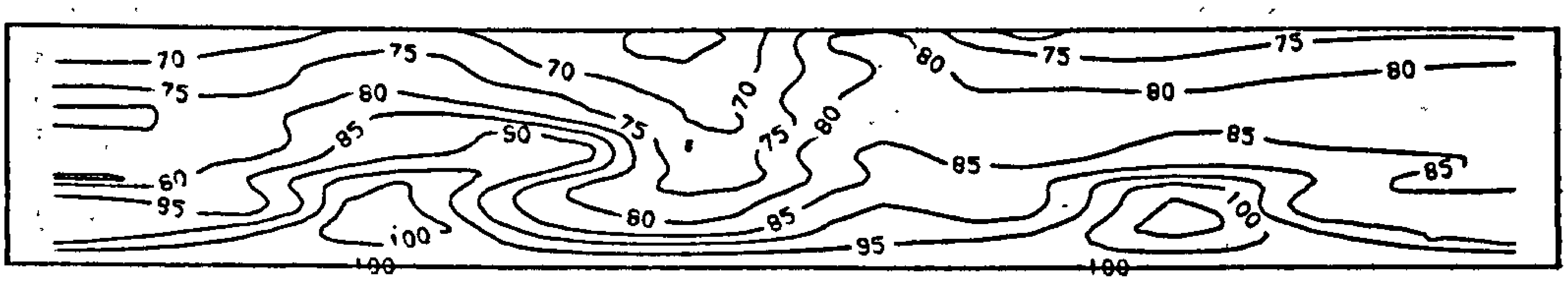
Figure 8 6:Variation in strength through the height of beam made from normal weight concrete



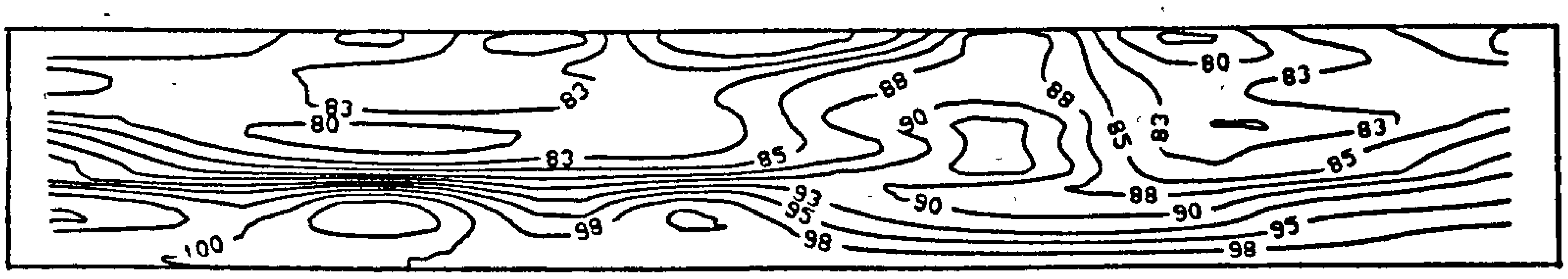
(a);Fully Lytag concrete



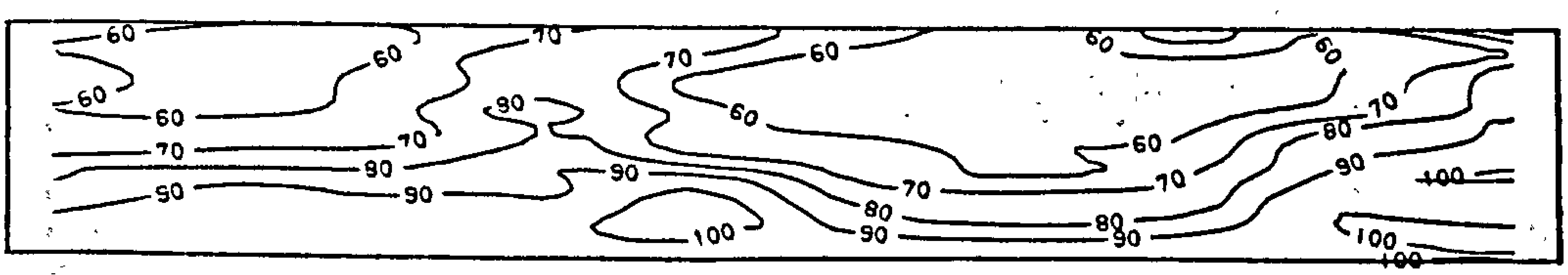
(b);Semi Lytag concrete



(c);Leca concrete

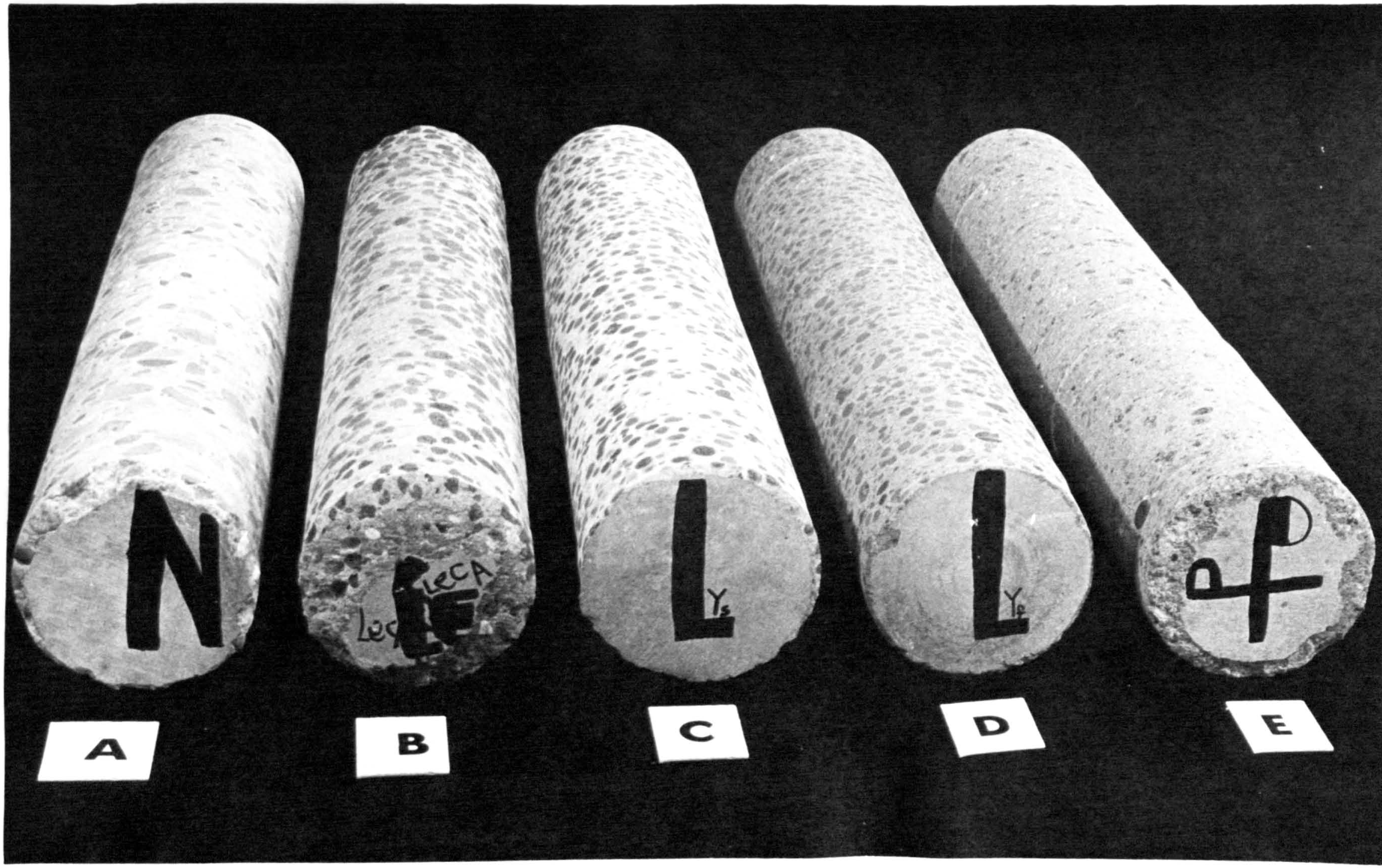


(d);Pellite concrete



(e);Normal weight concrete

Figure 8.7: Relative percentage strength contours for different types of concrete beam



A : Normal   B : Leca   C : Semi Lytag   D : Fully Lytag   E : Pellite

Figure 8.8: Distribution of aggregate in cores drilled from large scale concrete beams

CHAPTER 9. CONCLUSIONS AND RECOMMENDATIONS

9.1 CONCLUSIONS

This thesis has examined the insitu strength assessment of lightweight concrete. Aspects considered include measurement of the principal properties of lightweight concretes, validity of established insitu tests applied to lightweight concretes, mechanisms of failure of a range of partially destructive tests, and strength variability within large scale beams. For clarity, conclusions have been divided into these four main areas:

A. Principal properties of lightweight concretes

1. It has been demonstrated experimentally that lightweight concretes made with differing lightweight aggregates required different cement contents to produce similar compressive strengths for a given workability. The lightweight aggregates used can be arranged in the following order of increasing cement content; Lytag, Pellite and Leca.

2. Among available lightweight concretes, the compressive strength development in Lytag concretes is highly dependent on curing conditions whilst in Pellite concretes this effect is negligible.

3. At 28 days, the dry cured compressive strengths of lightweight concretes (tested dry) made of Lytag and Leca are higher than the wet cured compressive strengths as tested wet. This is more pronounced in fully Lytag concrete.

4. Beyond the age of 28 days, the compressive strength development in some types of concrete is significant, especially for wet cured conditions. Based on available data for fully Lytag, Leca and normal weight concretes, the fully Lytag concrete showed the largest increase in compressive strength beyond 28 days. At one year, this increase for fully Lytag concrete varied from 41-116% for wet curing. The corresponding increase for dry curing ranged from only 11-23%.

5. The tensile strengths of concretes are affected by curing conditions and this is most significant at high strength levels. Among available lightweight concretes, the tensile strength of fully Lytag and Leca concretes are most sensitive to curing conditions.

6. The influence of sand replacement for fine lightweight aggregates on tensile strength was observed on Lytag concretes. This was found to be more significant under dry curing conditions.

7. The estimation of static modulus of elasticity from measured dynamic modulus of elasticity using the equation $E_s=1.04E_d-4.11$ proposed for lightweight concrete (Swamy and Bandyopadhyay, 1975) was examined on fully Lytag, Leca and Pellite concretes along with published data on semi Lytag concrete (Lambert, 1982). The results confirmed the suitability of this latter equation which fitted the results

reasonably well.

8. The modulus of elasticity is affected by the type of concrete. The available concretes can be ranged in the order of decreasing dynamic modulus of elasticity; normal weight, Pellite, semi Lytag, fully Lytag and Leca concretes. At 25N/mm² strength level, Pellite, semi Lytag, fully Lytag and Leca concretes were found to be about 70%, 56%, 46% and 41% of the normal weight concrete respectively.

9. An equation of the form $E_s = 1.7\rho^2 f_c^{0.33} \times 10^{-6}$ reported by Neville (1988) was also found to be satisfactory to estimate static modulus of elasticity of all lightweight concretes available in this investigation.

B. Insitu tests on lightweight concrete

The results of all insitu tests showed dependency upon the type of concrete under investigation. In most cases, high correlation coefficients were found to exist between compressive strength and the parameter determined by the insitu tests considered. All also demonstrated testing variability dependent upon the type of concrete. The lowest variability was obtained with fully Lytag concrete, possibly as a result of observed improved homogeneity, whereas the highest variability was found with Leca concrete which seems to be related to non-uniform distribution of aggregates. On Leca concrete, tremendous differences in the test results of rebound hammer and partially destructive tests were detected between side and bottom faces of the laboratory cube specimens. Hence, the bottom faces of structural elements would not be recommended for assessing insitu strength of this type of lightweight concrete.

B1. Non-destructive tests

1. For rebound hammer tests, the general calibration provided by the manufacturer was found to predict the cube strength of fully Lytag concrete with reasonable agreement, and with good agreement up to a strength level of 25N/mm^2. In other types of concrete, the manufacturer's calibration under-estimated the actual cube strength, which is most pronounced for Leca and Pellite concretes. As a result of the influence of concrete type on correlation, a general calibration as provided by the manufacturer is unlikely to be of any practical value.

2. In various types of lightweight concrete, as a result of differences in concrete stiffness, a wide range of pulse velocities occur and in all cases these are significantly lower than expected with normal weight concrete of comparable strengths.

3. The type of curing over the first week was found to have a significant effect on pulse velocity measurements at later ages.

4. On dry concrete cubes, a drop in pulse velocity measurements was generally detected on available concretes after three to four weeks which was more significant at low strength concrete possibly owing to higher permeability.

5. The increase of pulse velocity with increasing strength was generally found to be less on dry cured concrete than on wet cured. However, due to high water absorption of lightweight aggregate, this effect seems to be less significant than in normal weight concrete.

6. In Lytag concretes, sand replacement for fine lightweight aggregate caused an increase of 7% on pulse velocity measurements.

7. At the age of six months, pulse velocity readings on dry cured cube specimens and large scale beams were compared. Pulse velocities through cube specimens were lower than at 28 days, but in the large scale beams were increased due to available moisture content in the interior body of concrete. This demonstrates the importance of pulse velocity/compressive strength relationships determined on specimens from the structure when assessing long term strength development.

B2. Partially destructive tests

The effect on correlations of curing conditions ranging from wet to dry was shown to be generally small and of no practical significance. It must be noted that dry conditions are required for the pull off test in the form used. Based on accuracy of strength estimation, the pull out and pull off methods were found to be the most satisfactory tests for all the types of lightweight concrete examined.

1. In Windsor Probe tests, using low power, the calibration supplied by the manufacturer is shown to be unreliable for strength prediction of fully Lytag concrete (available for this test) except for strength between about 14-22N/mm^2.

2. Splitting of the concrete and consequent difficulty in measuring the depth of penetration was experienced when Windsor Probes were fired on fully Lytag concrete under standard power.

3. At low power, the Windsor Probes appeared unable to evaluate dry cured strength of fully Lytag concrete above 35N/mm^2.

4. Lightweight concretes showed lower resistance against the probe penetration as compared to that on normal weight concrete, as a result of easier crushing of the lightweight aggregates and possibly also due to lower rigidity of lightweight concrete.

5. Deeper probe penetration was obtained on fully Lytag concrete when compared with published results on semi Lytag concrete. The lower probe penetration on semi Lytag concrete seems to be related to the sand replacement for fine lightweight aggregate which may increase the skin friction between probe and concrete as well as increasing the rigidity of the concrete.

6. Pull out resistances of Lok-Test on available concretes were significantly lower than those for normal weight concrete at

comparable strengths. Sand replacement for fine lightweight aggregate produced slightly higher pulling resistances on semi lightweight concretes than that obtained on the fully Lytag concrete, with the highest being for Pellite concrete possibly due to its particular aggregate characteristics.

7. Considerable difficulties were encountered with the (direct pull) internal fracture test applied to concrete made with Leca, and this method cannot be recommended for this type of lightweight concrete.

8. Failure loads for both B.R.E. and direct pull internal fracture tests applied on lightweight concrete were reduced as compared to those applied on normal weight concrete. The degree of reduction was found to be dependent on concrete type and loading method.

9. The ratios of internal fracture pull out force to torque were found to be 1.75 for fully Lytag concrete, and 1.37 and 1.44 for semi lightweight concretes made with Lytag and Pellite respectively. The values for semi lightweight concrete were thus close to the value of 1.4 reported for normal weight concrete.

10. In direct pull internal fracture tests, lightweight concretes (except Leca) showed similar characteristics as those obtained for the correlation between tensile and compressive strengths.

11. Either partial or full debonding occurred when pull off tests were applied to fully Lytag concrete at the age of 7 days. This occurred because of dampness of the surface of the concrete resulting from high water absorption of fine and coarse Lytag. It may thus not be desirable to use this test at an early age on fully Lytag or similar types of concrete.

12. For surface pull off tests, higher forces were achieved on lightweight concretes than obtained on normal weight concrete of comparable strength. Pull off resistance varied among lightweight concretes with Leca concrete producing the highest values.

13. A reduction in pull off strength was found to be caused by partial coring of specimens. The amount of reduction was found to be dependent on concrete type as well as core length, although the influence of core length was shown to be negligible beyond 20mm. In normal weight concrete, the pull off strength reduction was partially due to drilling action, as demonstrated by comparison between results obtained from 20mm drilled cores and 20mm formed cores. However, for fully Lytag concrete no such influence was observed.

B3. Destructive tests

1. The ℓ/d correction factor for 50mm cores is generally shown to depend on concrete type. However sand replacement for fine lightweight aggregate showed no significant change in correction factors.

2. Dry cured core specimens generally give higher ℓ/d correction factors than specimens wet cured and tested wet.

3. The effect of ℓ/d ratio on low strength concretes was generally higher than those on high strength concretes. This effect however is not clearly defined for fully Lytag concrete under dry curing. To minimise errors in interpretation of the core strength data, whenever possible, cores of $\ell/d=2.0$ should be tested.

4. The effect of ℓ/d ratio for lightweight concrete was considerably less than for the normal weight concrete.

5. For all lightweight concretes, the effect of varying ℓ/d ratios was close to that given by A.S.T.M. specification (which includes lightweight concretes), except for wet cured Pellite concrete which was more similar to that indicated by the British Standard. It can be noted that Pellite concrete is the densest lightweight concrete used in this investigation.

6. Vertically drilled cores give strengths which are not significantly different from horizontally drilled cores for lightweight concrete.

7. Core strengths ($\ell/d=2.0$) were lower than cube strengths, and the percentage reduction was dependent on concrete type. At a cube strength of $25N/mm^2$, the strength reduction in cores varied from 5% for fully Lytag concrete to 44% for Leca

concrete. This seems to be related to the effect of difference in shape of specimens, possible weakening of the aggregate by drilling action, and the non-uniform distribution of aggregate within the matrix as occurred in Leca concrete.

C. **Failure mechanisms of partially destructive tests**

1. The Windsor Probe penetrates into the concrete until its initial kinetic energy is absorbed by crushing and fracturing of the concrete. Some energy is also absorbed by friction between the probe and the concrete. A fracture zone is created at some distance above the probe tip in the form of triangular fracture lines.

2. Linear elastic stress analysis of the pull out test, before concrete cracking, indicates similar behaviour for different types of concrete (i.e. lightweight and normal weight concretes). Therefore the significant reduction found experimentally in pull out resistance of lightweight concrete is likely to be related to the stage after completion of crack formation. This would appear to be related to the absence, or limited degree, of effective aggregate interlock in lightweight concrete reported by Taylor (1970), Hamadi (1976) and Arasteh (1988) and supports the hypothesis given by Stone and Carino (1983), which states that the ultimate pull out resistance is controlled by aggregate interlock.

3. A theoretical relationship between the pull out force and the strength of concrete was obtained for the direct pull

internal fracture test, using basic engineering mechanics, assuming a simple tensile separation for the pull out cone of failure. This theoretical relationship indicates a good agreement with the experimental results on the lightweight concretes to which this test can be applied satisfactorily.

4. Based upon linear elastic stress analysis, concrete stiffness was shown to have a significant effect on pull off resistance. This implies that in surface pull off tests, concrete with high stiffness results in a lower pull off strength due to high localized stress created in the concrete beneath the axis of loading. This is in reasonable agreement with the experimental findings. Stress analysis also suggests that disk rigidity and thickness/diameter ratio have a great influence on pull off resistance of concrete. Lower disk stiffness and lower thickness/diameter ratio create lower pull off resistance, whereas higher disk stiffness and greater thickness/diameter ratio give rise to higher pull off resistance.

5. The lower pull off strength in partially cored pull off tests, as compared to surface tests, has been shown by means of stress analysis to be related to the removal of edge elements (this is some of the concrete outside the periphery of the disk, which carries a substantial part of the applied load) and stress concentrations created at the base of the partial core. The removal of edge elements was found to be more significant on pull off resistance of concrete with lower stiffness, which is in good agreement with experimental

findings. Observed dependency of pull off strength upon core length within 20mm depth from the concrete surface was verified by stress analysis which shows that edge elements within this distance are contributing substantially to carrying the applied load for surface tests.

D. **Concrete strength and variability in beams**

1. Among the insitu test methods used on the various concrete beams, the rebound hammer is found to over-estimate the concrete strength. This is believed to be related to the mould type, since the calibration of rebound hammer was established using cube specimens cast in steel moulds as opposed to the wooden moulds used for beam.

2. Shinkage cracks in fully Lytag concrete created lower pull out resistance, and hence an estimate of strength in the concrete beam nearly 20% lower than those obtained from pulse velocity or core tests from the interior body of concrete.

3. The strength of concrete in beams, as expected, was generally found to decrease towards the top. This phenomenon was shown to be related to the concrete type. The average strength variation between top and bottom of fully Lytag and Pellite concretes were approximately 10% and 20% respectively. The corresponding variation in semi Lytag, Leca and normal weight concretes were found to be around 35%.

4. The coefficients of variation of pulse velocity and Lok-Test results at each level of beams appeared to be similar, or

even lower in the case of pulse velocity, than those obtained on the cube specimens. However, the overall coefficients of variation of these test methods on concrete beams were significantly higher, with the least variability on fully Lytag concrete.

5. The ratios of population mean concrete strength in cube specimens to those obtained in beams (based on Lok-Test results) were shown in most cases to be similar in lightweight and normal weight concretes. This confirms the suitability of the value of the partial material safety factor for concrete strength, recommended in BS 8110 : Part 1 (1985) for lightweight concretes as well as normal weight concrete.

9.2 GENERAL

Performance of the insitu test methods for strength evaluation of lightweight concrete has been shown to be satisfactory in most instances. It is confirmed however that correlations between measured insitu test values and strength for lightweight concretes are different from those for normal weight concrete. Within lightweight concretes, these correlations are shown to be dependent upon the physical characterstics of the lightweight aggregates and their influence upon the failure mechanism associated with each test.

Most test methods may thus be used with confidence for lightweight concrete provided that fully relevant strength correlations are developed. There is however a need for pull-off disk proportions and material to be standardized, as well as depths of partial coring.

Test variability is in most cases less than, or comparable to, that expected for normal weight concrete. Insitu variability of lightweight concretes may however differ according to concrete type, and may not necessarily be the same as that expected for normal weight concrete. This leads to the conclusion that planning and interpretation of insitu strength assessment of lightweight concrete should not necessarily be assumed to be similar to that appropriate to normal weight concrete.

9.3 RECOMMENDATIONS FOR FUTURE WORK

Several topics needing further examination came to light which are given below:

1. In the present work the Windsor Probe test gave considerable difficulty when standard power was used on the fully Lytag concrete beam (1000x150x250mm). Further research would be required to develop an intermediate power level by placing the probe/driving head assembly at a different distance into the driver barrel and examining the reliability of the test on different types of lightweight concrete.

2. In the pull out test, further theoretical analysis is required at the stage of postcracking to fully confirm the influence of aggregate interlock on ultimate pull out resistance.

3. Finite element results on pull off tests highlight the significant effect of disk rigidity and its thickness/diameter ratio on pull off resistance of concrete. A more thorough experimental investigation of this effect is required.

4. Improvements can be made to the finite element analysis of pull off tests, by considering the non-linear behaviour of the concrete in tension and taking into account the presence of particles of various sizes, shapes and stiffness. These would permit more accurate analyses and enable more detailed studies at the ultimate stage.

5. In the present study the concrete strength variation within the large scale beam of semi Lytag concrete was found to be significantly higher than in the fully Lytag concrete beam. This was related to high bleeding which occurred during concreting. Further investigation would be useful to try to reduce the bleeding using partial replacement of cement with P.F.A. and to study its influence on strength variations of semi Lytag concrete beams.

6. The ratios of population mean concrete strength in cubes divided by the concrete strength in beams has been shown to be similar in lightweight and normal weight concretes. This similarity needs to be further assessed on other types of structural elements. It is recommended that this should be carried out under site conditions. The strength ratio obtained may then be compared quantitatively with γ_m=1.5 given in BS 8110 : Part 1 (1985).

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APPENDIX A
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SUPPLEMENTARY TABLES FOR CHAPTER 5.

A-1 ; Compressive strength development in short term(dry cured as for beam)

A-2 ; Long term compressive strength development with age

A-3 ; Effect of curing conditions on tensile splitting strength

A-4 ; Development of modulus of elasticity with age

Table A-1:Compressive strength development in short term(dry cured as for beam)

Type of Concrete	Mix No.	Age Days	Cube Compressive Strength (N/mm^2)	Percentage of 28 Day Strength (%)
Fully Lytag Concrete	L-C	1	11.1	25.1
		2	17.0	38.5
		3	20.7	46.8
		5	24.0	54.3
		7	27.4	62.0
		14	34.0	76.9
		21	42.1	95.2
		28	44.2	100.0
Semi Lytag Concrete	L-3	1	13.1	31.5
		2	16.6	39.9
		3	20.4	49.0
		5	25.0	64.9
		7	31.5	75.7
		14	39.3	94.5
		21	41.3	99.3
		28	41.6	100.0
Leca Concrete	Le-4	1	15.9	50.6
		2	20.8	66.2
		3	21.3	67.8
		5	23.5	74.8
		7	28.0	89.2
		14	29.7	94.6
		21	29.2	93.0
		28	31.4	100.0
Pellite Concrete	P-5	1	12.4	26.1
		2	24.0	50.5
		3	27.3	57.5
		5	33.2	69.9
		7	35.9	75.6
		14	42.3	89.1
		21	44.5	93.7
		28	47.5	100.0
Normal Weight Concrete	N-4	1	12.9	32.1
		2	18.2	45.3
		3	21.3	53.0
		5	27.9	69.4
		7	30.6	76.1
		14	38.0	94.5
		21	39.1	97.3
		28	40.2	100.0

Table A-2:Long term compressive strength development with age

Type of Concrete	Mix No.	Age Days	Cube Compressive Strength: Wet Cured f_c N/mm²	Wet Cured c.v. %	Dry Cured f_c N/mm²	Dry Cured c.v. %
Fully Lytag Concrete	L-A	7	15.0	4.1	16.9	3.8
		28	23.0	0.8	28.7	2.6
		180	42.6	2.8	32.2	8.2
		360	49.6	4.2	32.0	3.0
	L-B	7	19.3	2.8	20.8	3.8
		28	31.6	1.3	34.3	3.5
		180	46.9	2.2	41.2	3.0
		360	53.6	3.5	39.0	4.1
	L-C	7	22.4	1.6	27.7	3.4
		28	35.1	1.1	39.3	1.2
		180	49.9	4.2	46.5	4.1
		360	56.1	2.3	48.3	6.3
	L-D	7	26.8	2.9	30.9	2.3
		28	41.3	3.4	46.5	3.7
		180	57.9	1.8	48.9	1.7
		360	58.4	2.1	51.4	8.5
Leca Concrete	Le-1	7	9.2	5.4	-	-
		14	-	-	13.1	5.0
		28	13.7	6.4	13.5	3.8
		360	-	-	14.0	4.8
	Le-2	7	17.7	6.4	-	-
		14	-	-	18.1	7.2
		28	19.9	2.9	19.1	10.2
		360	24.7	5.3	21.5	4.0
	Le-3	7	17.9	4.3	-	-
		14	-	-	19.7	8.5
		28	20.8	5.8	19.8	4.5
		360	26.2	6.1	22.2	12.0
	Le-4	7	20.7	5.1	-	-
		14	-	-	23.1	3.9
		28	24.8	7.5	24.9	5.7
		360	27.1	4.7	25.2	1.5
Normal Weight Concrete	N-1	7	-	-	13.5	5.5
		28	18.5	10.9	19.3	5.0
		180	22.5	2.1	22.1	1.9
		360	22.1	6.3	20.6	4.2
	N-2	7	-	-	16.9	4.2
		28	23.0	1.8	22.9	1.8
		180	24.1	4.2	26.4	4.2
		360	25.4	6.0	24.4	5.3
	N-3	7	-	-	20.7	1.0
		28	31.6	0.9	28.7	2.7
		180	37.0	3.7	32.8	2.1
		360	37.0	2.9	32.5	8.7
	N-4	7	-	-	29.1	2.8
		28	44.8	2.8	38.3	2.8
		180	53.3	1.4	43.2	4.6
		360	55.4	2.8	43.7	2.9

Table A-3:Effect of curing conditions on tensile splitting strength

Type of Concrete	Mix No.	Age Days	Cube compressive Strength N/mm^2				Tensile splitting Strength N/mm^2			
			Curing Condition							
			Wet		Dry		Wet		Dry	
			f_c	c.v %	f_c	c.v %	f_t	c.v %	f_t	c.v %
Fully Lytag Concrete	L-A	28	22.1	4.8	29.6	0.9	2.08	3.1	2.26	2.4
		360	49.6	4.2	27.8	3.0	2.96	8.9	2.28	3.1
	L-B	28	27.5	4.5	31.9	6.0	2.34	7.9	2.38	4.7
		360	50.6	2.5	39.0	4.1	3.02	5.8	2.49	10.0
	L-C	28	35.3	1.0	39.1	5.0	2.90	4.6	2.39	5.5
		360	56.1	2.3	51.0	0.9	3.13	7.3	2.62	3.8
	L-D	3	24.2	1.7	23.3	5.0	2.17	5.1	2.00	1.8
		7	30.6	6.0	33.6	1.1	2.51	17.0	2.31	6.7
		28	42.4	5.7	46.7	1.7	3.06	6.6	2.32	3.9
		360	57.1	3.0	51.4	8.5	3.08	6.8	2.64	1.4
Semi Lytag Concrete	L-1	28	15.8	4.5	21.3	2.2	1.68	3.8	2.34	6.4
	L-2	28	30.4	5.3	34.5	5.0	2.57	5.0	2.50	4.5
	L-3	28	38.7	1.6	41.5	1.3	3.34	5.0	2.84	8.7
	L-4	28	47.8	1.5	48.2	5.2	3.53	3.3	3.08	12.0
Leca Concrete	Le-1	28	14.2	10.0	14.2	1.1	1.45	7.6	1.42	7.2
	Le-2	28	19.4	11.0	17.6	9.1	1.95	5.1	1.50	20.0
	Le-3	28	23.8	6.5	23.2	12.0	2.07	6.3	1.70	8.4
	Le-4	28	25.2	6.6	25.2	0.69	2.28	8.7	1.71	3.7
Pellite Concrete	P-1	28	19.7	3.1	19.8	4.9	1.88	12.0	1.67	13.0
	P-2	28	34.1	5.3	31.9	0.79	2.73	3.4	2.25	7.2
	P-5	28	44.1	1.0	43.4	2.7	3.48	2.0	2.66	4.5
	P-6	28	51.8	4.9	51.7	5.4	3.54	2.5	2.84	2.8
Normal Weight Concrete	N-1	28	-	-	20.3	3.6	-	-	2.13	3.3
		360	-	-	21.4	4.0	-	-	1.99	7.3
	N-2	28	-	-	24.7	2.9	-	-	2.35	3.2
		360	-	-	25.4	5.3	-	-	2.35	5.4
	N-3	28	-	-	29.5	4.0	-	-	2.59	5.8
		360	-	-	33.8	3.1	-	-	2.47	18.0
	N-4	28	-	-	41.6	2.8	-	-	2.90	10.0
		360	-	-	45.4	2.8	-	-	3.17	12.0

Table A-4:Development of modulus of elasticity with age

Type of Concrete	Mix No.	Age Days	Cube Compressive Strength N/mm^2		Dynamic Modulus of Elasticity KN/mm^2		Static Modulus of Elasticity KN/mm^2
			Curing Condition				
			Wet	Dry	Wet	Dry	Dry
Fully Lytag Concrete	L-A	7	-	16.8	-	16.1	-
		28		29.6	-	17.4	15.2
		360	-	32.0	-	16.2	-
	L-B	28	-	32.9	-	17.7	15.5
		360	-	39.0	-	16.4	-
	L-C	28	-	42.7	-	18.6	16.3
		360	-	48.3	-	17.1	-
	L-D	3	22.8	23.9	17.8	17.4	-
		5	-	-	18.5	18.2	-
		7	30.3	32.6	18.9	18.6	-
		10	-	-	19.2	19.0	-
		14	33.2	41.7	19.5	19.0	-
		21	-	-	19.8	19.1	-
		28	39.9	46.7	20.1	18.9	17.6
		60	-	-	20.9	18.2	-
		90	-	-	21.4	17.6	-
		180	57.9	48.9	22.3	16.9	-
		270	-	-	22.4	17.0	-
		360	58.4	51.4	22.5	17.0	-
Semi Lytag Concrete	L-1	1	-	-	8.7	-	-
		3	6.5	-	13.9	-	-
		7	9.5	-	16.6	-	-
		14	11.8	-	18.7	-	-
		28	15.8	-	20.4	-	-
	L-2	7	21.6	-	21.8	-	-
		28	30.7	-	23.8	-	-
	L-3	7	27.9	-	22.4	-	-
		28	37.5	-	24.5	-	-
	L-4	1	-	-	19.0	-	-
		3	29.5	-	23.2	-	-
		7	35.1	-	24.3	-	-
		14	41.7	-	25.5	-	-
		28	47.8	-	26.6	-	-
Leca Concrete	Le-1	2	5.9	6.3	8.6	8.5	-
		4	7.1	8.0	10.0	9.9	-
		9	9.3	10.7	11.3	10.8	-
		21	-	-	12.4	11.3	-
		28	12.7	13.1	12.6	11.3	
		60	-	-	13.3	11.2	-
		90	-	-	13.4	11.0	-
		180	-	-	13.6	10.4	-

Continued....................

Table A-4 (Contd.)

Type of Concrete	Mix No.	Age Days	Cube Compressive Strength N/mm²		Dynamic Modulus of Elasticity KN/mm²		Static Modulus of Elasticity KN/mm²
			Curing Condition				
			Wet	Dry	Wet	Dry	Dry
Leca Concrete	Le-4	2	18.3	19.3	13.3	13.9	-
		4	22.0	21.4	16.1	15.4	-
		9	23.0	24.5	16.8	15.9	-
		21	-	-	17.7	15.8	-
		28	29.6	25.8	17.7	16.0	-
		60	-	-	18.4	15.7	-
		90	-	-	18.1	15.1	-
		270	-	-	18.7	15.3	-
		360	-	-	19.3	14.6	-
Pellite Concrete	P-1	1	-	-	16.3	16.1	-
		3	11.1	11.1	21.6	21.1	-
		5	-	-	23.4	21.8	-
		7	14.1	14.3	24.5	23.1	-
		14	17.2	17.1	25.8	23.8	-
		21	-	-	27.2	24.3	-
		28	19.7	19.8	27.7	24.0	
		90	-	-	30.1	23.7	-
		180	-	-	30.7	23.7	-
		360	-	-	31.0	22.4	-
	P-4	1	-	-	25.0	24.9	-
		3	27.5	28.6	28.5	28.1	-
		5	-	-	30.2	29.3	-
		7	35.3	35.9	31.1	29.6	-
		14	40.6	41.0	32.6	29.6	-
		21	-	-	33.2	29.9	-
		28	44.1	43.4	33.7	29.8	
		90	-	-	34.4	29.6	-
		180	-	-	34.8	29.7	-
		360	-	-	35.2	28.7	-
Normal Weight Concrete	N-1	28	18.5	-	37.6	-	-
		180	22.5	-	39.9	-	-
	N-2	28	23.0	-	40.5	-	-
		180	24.1	-	42.6	-	-
	N-3	28	31.6	-	41.4	-	-
		180	37.0	-	43.3	-	-
	N-4	28	44.8	-	44.0	-	-
		180	53.3	-	46.1	-	-

APPENDIX B

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SUPPLEMENTARY TABLES FOR CHAPTER 6.

Table B-1:Rebound hammer test results for different types of concrete

Type of Concrete	Mix No.	Age Days	Rebound Number Side Face R	Side Face c.v (%)	Bottom Face R	Bottom Face c.v (%)	Compressive Strength N/mm^2 f_c	c.v (%)
Fully Lytag Concrete	L-C	1	20.1	5.2	21.3	4.9	11.1	4.1
		2	25.4	5.0	26.3	5.7	17.0	2.7
		3	27.2	7.3	29.9	4.4	20.7	2.7
		5	29.5	5.1	30.9	3.2	24.0	3.5
		7	31.0	6.1	33.0	3.4	27.4	2.6
		14	35.8	5.8	37.0	4.8	34.0	2.7
		21	38.1	5.5	39.4	3.0	42.1	2.4
		28	37.6	5.2	40.2	3.0	46.1	1.5
Semi Lytag Concrete	L-3	1	19.4	8.3	23.7	4.5	13.1	1.2
		2	23.0	6.0	26.0	3.4	16.6	6.1
		3	24.9	8.8	29.6	7.1	20.4	2.7
		5	29.4	6.5	32.4	7.1	27.0	8.5
		7	30.5	7.1	33.5	5.2	31.5	3.1
		14	33.4	5.1	36.7	4.9	39.3	2.5
		21	34.2	6.0	37.1	4.3	41.3	2.7
		28	34.7	5.9	37.6	5.1	42.9	3.6
Leca Concrete	Le-4	2	16.3	15.8	36.4	4.1	20.8	4.9
		3	17.8	16.2	39.1	4.4	21.3	13.3
		5	22.4	16.8	38.6	3.2	23.5	5.3
		7	25.2	17.4	40.2	4.4	28.0	8.0
		14	24.2	19.9	40.9	3.3	29.7	17.9
		21	26.2	18.0	40.9	4.3	29.2	5.0
		28	26.2	15.6	40.4	4.7	31.4	13.4
Pellite Concrete	P-5	1	19.1	6.6	21.1	3.2	12.4	2.0
		2	25.5	7.5	26.8	7.7	24.0	1.1
		3	27.7	6.9	28.7	4.2	27.3	0.2
		5	28.8	6.8	31.3	6.3	33.2	1.7
		7	29.1	5.7	31.0	4.7	35.9	2.4
		14	30.4	8.4	32.3	5.1	42.3	1.3
		21	32.4	6.6	32.8	5.6	44.5	0.8
		28	32.7	7.0	32.8	8.3	47.9	0.3
Normal Weight Concrete	N-4	1	19.1	8.9	23.6	18.6	12.9	3.4
		2	23.1	9.4	29.1	11.7	18.2	0.6
		3	24.9	11.6	26.6	9.4	21.3	3.3
		5	28.4	10.1	27.9	6.5	27.9	2.5
		7	29.8	9.4	35.3	13.2	30.6	0.9
		14	33.0	7.6	38.0	7.4	38.0	5.3
		21	31.8	8.5	37.1	8.2	39.1	1.2
		28	34.1	8.6	35.3	7.5	40.0	4.8

Table B-2:Measured pulse velocity versus dynamic modulus of elasticity

Type of Concrete	Mix No.	Age Days	Pulse Velocity Km/sec				Dynamic Modulus of Elasticity KN/mm^2			
			Wet Cured		Dry Cured		Wet Cured		Dry Cured	
			V	c.v %	V	c.v %	E_d	c.v %	E_d	c.v %
Fully Lytag Concrete	L-A	28	-	-	3.53	0.53	-	-	17.4	0.90
		360	-	-	3.40	0.80	-	-	16.2	0.90
	L-B	28	-	-	3.54	0.82	-	-	17.7	1.30
		360	-	-	3.42	0.20	-	-	16.4	0.40
	L-C	28	-	-	3.65	0.56	-	-	18.6	2.0
		360	-	-	3.48	0.54	-	-	17.1	1.2
	L-D	3	3.53	0.80	3.54	0.40	17.8	0.40	17.4	0.81
		5	3.59	0.39	3.63	0.78	18.5	0.38	18.2	1.17
		7	3.64	0.58	3.66	0.19	18.9	0.51	18.6	0.76
		10	3.66	0.19	3.69	0.74	19.2	0	19.0	0.74
		14	3.66	0	3.69	0	19.5	0.87	19.0	1.49
		21	3.70	0.19	3.70	0.75	19.8	0.62	19.1	0.74
		28	3.69	0.57	3.63	0.57	20.0	0.41	18.9	1.13
		60	3.77	0.56	3.61	1.14	20.9	1.02	18.2	1.17
		90	3.79	0	3.57	1.34	21.4	0.33	17.6	1.21
		180	3.80	0.55	3.43	0.62	22.3	0.32	16.9	1.67
		270	3.82	0.18	3.45	0.60	22.4	0.95	17.0	0.83
		360	3.84	0.37	3.32	1.24	22.5	0.31	17.0	5.66
Semi Lytag Concrete	L-1	1	2.75	3.90	-	-	8.7	5.50	-	-
		3	3.21	0.80	-	-	13.9	6.50	-	-
		7	3.37	0.58	-	-	16.6	4.60	-	-
		14	3.45	0.80	-	-	18.7	4.10	-	-
		28	3.58	1.10	-	-	20.4	3.80	-	-
	L-2	7	3.74	0.27	-	-	21.8	1.90	-	-
		28	3.91	0	-	-	23.8	1.80	-	-
	L-3	7	3.81	0.80	-	-	22.4	1.90	-	-
		28	3.98	0.84	-	-	24.5	2.00	-	-
	L-4	1	3.58	0.74	-	-	19.0	0.60	-	-
		3	3.82	0	-	-	23.2	0.40	-	-
		7	3.93	0.22	-	-	24.3	0.40	-	-
		14	4.00	0.80	-	-	25.5	0.60	-	-
		28	4.06	0.23	-	-	26.6	0	-	-
Leca Concrete	Le-1	2	2.73	1.90	2.68	0.68	8.6	1.80	8.5	1.80
		4	2.88	2.50	2.82	1.30	10.0	2.60	9.9	2.00
		9	3.05	2.40	2.90	1.60	11.3	1.80	10.8	1.40
		21	3.17	3.40	3.00	1.20	12.4	2.00	11.3	2.20
		28	3.19	3.20	2.99	1.30	12.6	1.20	11.3	2.20
		60	3.28	3.80	3.02	2.90	13.3	2.70	11.2	2.90
		90	3.30	3.50	2.98	2.60	13.4	1.60	11.0	3.60
		180	3.32	2.70	2.85	3.70	13.6	1.50	10.4	4.30
	Le-4	2	3.17	2.30	3.19	0.74	13.3	12.10	13.9	9.40
		4	3.36	4.70	3.23	0.82	16.1	1.40	15.4	3.00
		9	3.55	0.99	3.45	2.50	16.8	4.50	15.9	3.30
		21	3.66	0.62	3.36	0.60	17.7	3.00	15.8	3.10
		28	3.66	5.30	3.34	2.70	17.7	3.50	16.0	3.10
		60	3.69	0.90	3.32	1.60	18.4	5.70	15.7	2.30
		90	3.74	0.50	3.29	1.00	18.1	4.10	15.1	2.70
		270	3.76	0.90	3.26	0.90	18.7	3.50	15.3	2.70
		360	3.76	0.90	3.26	0.90	19.3	0.50	14.6	2.10

Continued..............

Table B-2 (contd.)

Type of Concrete	Mix No.	Age Days	Pulse Velocity Km/sec				Dynamic Modulus of Elasticity KN/mm^2			
			Wet Cured		Dry Cured		Wet Cured		Dry Cured	
			V	c.v %	V	c.v %	E_d	c.v %	E_d	c.v %
Pellite Concrete	P-1	1	3.47	1.02	3.23	2.40	16.3	3.47	16.1	8.78
		3	3.83	1.11	3.58	0.59	21.6	2.30	21.1	4.02
		5	3.86	0.55	3.58	1.19	23.4	1.51	21.8	0.33
		7	3.94	0.18	3.70	1.34	24.5	2.02	23.1	2.76
		14	4.03	0.53	3.77	0.94	25.8	1.37	23.8	2.40
		21	4.12	0.86	3.82	0.92	27.2	0.78	24.3	3.50
		28	4.13	0.51	3.79	0.56	27.7	0.51	24.0	1.18
		90	4.25	0.83	3.77	0.94	30.1	0.47	23.7	0.60
		180	4.26	1.16	3.77	1.13	30.7	0.46	23.7	1.50
		360	4.26	1.16	3.62	0.78	31.0	0.69	22.4	1.26
	P-5	1	3.97	1.07	3.88	0	25.0	0.28	24.9	1.70
		3	4.19	0.34	4.07	0.52	28.5	1.70	28.1	2.00
		5	4.28	0.33	4.13	0.51	30.2	0.23	29.3	0.48
		7	4.34	0.33	4.17	0.51	31.1	0.45	29.6	1.90
		14	4.36	0.32	4.16	0.34	32.6	0.87	29.6	2.20
		21	4.38	0.32	4.15	0.49	33.2	0.64	29.9	1.90
		28	4.41	0.16	4.20	0	33.7	0.63	29.8	1.20
		90	4.41	0.16	4.18	0.17	34.4	0.82	29.6	1.90
		180	4.44	0	4.12	0.17	34.8	0.81	29.7	1.40
		360	4.44	0.41	4.10	0	35.2	0.20	28.7	1.20
Normal Weight Concrete	N-1	28	4.61	0	-	-	37.6	3.60	-	-
		180	4.61	1.70	-	-	39.9	3.50	-	-
	N-2	28	4.65	0	-	-	40.5	0.70	-	-
		180	4.67	1.20	-	-	42.6	1.50	-	-
	N-3	28	4.68	0.33	-	-	41.4	0.51	-	-
		180	4.82	0.60	-	-	43.3	1.60	-	-
	N-4	28	4.80	0.34	-	-	44.0	3.90	-	-
		180	4.82	0.70	-	-	46.1	2.90	-	-

Table B-3:Measured pulse velocity versus cube compressive strength

Type of Concrete	Mix No.	Age Days	Pulse Velocity Km/sec				Cube Compressive Strength N/mm^2			
			Wet		Dry		Wet		Dry	
			V	c.v %	V	c.v %	f_c	c.v %	f_c	c.v %
Fully Lytag Concrete	L-A	3	3.20	0.36	3.28	0.83	10.5	4.0	12.5	7.8
		7	3.32	0.51	3.32	0.77	14.7	3.9	18.8	1.7
		14	3.41	0	3.37	0.70	16.8	2.4	21.8	2.4
		28	3.44	0.99	3.31	1.10	21.2	7.7	26.3	3.4
		180	3.63	0.76	3.13	1.30	40.1	2.4	27.9	8.5
		360	3.67	0.77	3.17	0.63	44.7	3.7	27.8	5.9
	L-D	3	3.50	0.36	3.55	0.41	22.8	2.5	23.9	4.1
		7	3.63	0.21	3.65	0.96	30.3	1.4	32.6	10.4
		14	3.66	0.36	3.68	0.42	33.2	3.8	41.7	3.2
		28	3.69	0.38	3.63	0.21	39.9	1.0	46.7	2.9
		180	3.82	0.38	3.42	0.52	57.9	1.8	48.9	1.7
		360	3.84	0.23	3.42	1.80	58.4	2.1	51.4	8.5
Semi Lytag Concrete	L-1	3	3.21	0.80	3.16	2.30	6.5	1.5	8.4	4.8
		7	3.37	0.58	3.26	0.66	9.5	6.7	13.5	7.1
		14	3.45	0.80	3.29	2.30	11.8	4.4	15.8	3.9
		28	3.58	1.10	3.31	0.51	15.8	4.5	17.6	5.4
	L-4	3	3.82	0.40	3.80	0.82	29.5	4.4	33.2	3.8
		7	3.94	0.23	3.93	0.22	35.1	3.1	40.2	3.7
		14	4.00	0.80	3.97	1.5	41.7	2.0	48.1	2.4
		28	4.06	0.23	3.99	0.46	47.8	1.5	51.3	5.6
Leca Concrete	Le-1	2	2.74	1.60	2.68	0.29	5.9	5.1	6.3	4.0
		4	2.88	1.90	2.87	0.19	7.1	5.0	8.0	4.4
		9	3.06	4.10	2.95	2.50	9.3	2.2	10.7	3.3
		28	3.19	1.10	2.99	1.30	12.7	1.2	13.1	6.5
	Le-4	2	3.17	2.30	3.19	0.74	18.3	6.9	19.3	4.1
		4	3.36	4.70	3.23	0.82	22.0	3.3	21.4	18.3
		9	3.55	0.99	3.45	2.50	23.0	3.1	24.5	9.4
		28	3.66	5.30	3.33	2.70	29.6	13.0	25.8	5.7
Pellite Concrete	P-1	3	3.83	1.11	3.65	0.96	11.1	2.3	11.1	7.0
		7	3.95	0.69	3.67	2.40	14.1	3.9	14.3	6.7
		14	4.04	0.57	3.75	0.68	17.2	2.3	17.1	3.2
		21	4.05	1.20	3.77	1.30	19.2	2.1	18.7	3.5
		28	4.08	0.61	3.72	0.77	19.7	3.1	19.8	4.9
	P-5	3	4.16	0.25	4.06	0.23	27.5	3.6	28.6	2.8
		7	4.30	0.24	4.16	0.25	35.3	2.2	35.9	4.4
		14	4.37	0	4.19	0.48	40.6	2.1	41.0	2.4
		28	4.44	0.25	4.23	0.64	44.1	1.0	43.4	2.7

Table B-4:Windsor Probe test results for fully Lytag concrete(low power)

Mix No.	Age Days	Exposed Length (mm) Wet Cured W	c.v (%)	Dry Cured W	C.v (%)	Compressive Strength (N/mm^2) Wet Cured f_c	c.v (%)	Dry Cured f_c	c.v (%)
L-A	7	45.2	3.00	45.5	3.00	15.0	4.1	16.9	3.8
	28	49.2	0.86	53.7	2.70	23.0	0.8	28.7	2.6
L-B	7	48.7	3.80	49.2	6.10	19.3	2.8	20.8	3.8
	28	52.7	2.95	60.7	2.60	31.6	7.4	34.3	3.5
L-C	7	49.4	5.80	49.5	1.50	22.4	1.6	27.7	3.4
	28	53.9	3.90	60.3	2.80	35.1	1.1	39.3	1.2
L-D	7	52.0	0.29	54.7	1.80	26.8	2.9	30.9	2.3
	28	-	-	60.2	3.30	-	-	46.5	3.7

Table B-5:Measured compressive strength versus estimated strength from Windsor Probe manual

Exposed Probe Length mm	Cube Compressive Strength N/mm^2 Manual	Actual
45.2	17.8	15.0
45.5	18.1	16.9
48.7	20.4	19.3
49.2	21.0	20.8
49.4	21.1	22.4
49.5	21.9	27.7
51.0	23.0	26.8
54.7	25.0	30.9

Table B-6:Pull out test results for different types of concrete

Type of Concrete	Mix No.	Age Days	Lok force KN				Cube Compressive Strength N/mm^2			
			Wet		Dry		Wet		Dry	
			L	c.v %	L	c.v %	f_c	c.v %	f_c	c.v %
Fully Lytag Concrete	L-A	7	9.2	5.2	9.5	6.0	16.0	3.3	17.6	2.0
		28	14.9	10.8	17.4	8.5	24.8	4.2	30.0	3.1
		180	22.1	6.6	18.8	11.3	42.6	2.8	27.9	8.2
		360	24.6	6.7	18.4	19.5	49.6	4.2	27.8	3.0
	L-B	7	10.5	10.7	10.6	4.6	20.0	6.7	22.5	8.9
		28	15.4	5.3	17.7	4.7	32.3	2.3	34.1	6.2
		180	25.4	5.5	22.7	11.7	46.9	2.2	41.2	3.0
		360	28.1	5.7	21.7	10.8	53.6	3.5	39.0	4.1
	L-C	7	13.5	4.1	14.1	7.0	23.7	5.8	27.3	3.6
		28	19.4	5.6	22.5	9.1	34.9	2.1	40.0	4.8
		180	26.8	11.0	25.4	8.8	51.4	1.9	48.9	6.9
		360	28.6	9.4	25.0	11.4	55.6	1.8	51.0	0.9
	L-D	7	16.4	5.2	16.9	5.9	31.2	5.1	35.6	5.5
		28	22.9	5.7	23.5	7.0	41.6	5.1	47.2	7.5
		180	30.1	10.2	26.5	16.7	57.9	1.8	48.9	5.7
		360	32.0	4.7	25.2	14.4	58.4	2.1	51.4	8.5
Semi Lytag Concrete	L-1	7	-	-	10.4	6.2	-	-	17.8	5.5
		28	-	-	13.6	12.9	-	-	23.7	2.2
	L-2	7	-	-	13.0	8.8	-	-	22.4	4.7
		28	-	-	17.1	4.9	-	-	34.5	5.0
	L-3	7	-	-	16.6	4.9	-	-	30.7	8.8
		28	-	-	22.1	9.9	-	-	40.6	4.8
	L-4	7	-	-	21.6	9.6	-	-	39.4	3.9
		28	-	-	25.9	11.2	-	-	47.9	5.4
Leca+ Concrete	Le-1	7	6.6 (9.5)	10.7	-	-	10.4	6.4	-	-
		28	9.2 (16.3)	16.3	-	-	15.1	5.1	-	-
	Le-2	7	9.0 (19.5)	6.7	-	-	18.4	6.4	-	-
		28	11.9 (21.0)	7.0	-	-	20.7	2.9	-	-
	Le-3	7	11.6 (16.5)	8.7	-	-	18.6	4.3	-	-
		28	12.7 (22.5)	14.9	-	-	21.6	5.8	-	-
	Le-4	7	13.0 (19.8)	16.0	-	-	21.5	5.1	-	-
		28	13.5 (24.5)	14.2	-	-	25.8	7.5	-	-
Pellite Concrete	P-1	7	-	-	9.8	9.4	-	-	16.0	3.5
		28	-	-	14.1	8.2	-	-	20.8	4.8
	P-2	7	-	-	15.5	8.6	-	-	27.2	4.0
		28	-	-	21.0	9.9	-	-	35.2	0.2
	P-3	7	-	-	18.3	11.2	-	-	30.8	3.7
		28	-	-	22.9	7.4	-	-	40.1	4.3
	P-5	7	-	-	19.3	10.0	-	-	36.9	3.3
		28	-	-	25.2	11.5	-	-	46.2	0.5
Normal Weight Concrete	N-1	360	16.8	18.3	-	-	22.1	6.3	-	-
	N-2	360	21.0	11.8	-	-	25.4	6.0	-	-
	N-3	360	25.6	12.4	-	-	37.0	2.9	-	-

+;The values given in bracket represent the measured pull out resistance at the bottom face of cube specimens

Table B-7:B.R.E. internal fracture test results for different types of lightweight concrete

Type of Concrete	Mix No.	Age Days	Torque Meter N-m				Cube Compressive Strength N/mm^2			
			Wet		Dry		Wet		Dry	
			T	c.v %	T	c.v %	f_c	c.v %	f_c	c.v %
Fully Lytag Concrete	L-A	7	2.05	14.0	2.10	13.5	13.5	5.1	16.0	5.0
		28	2.33	6.0	2.56	10.8	24.3	2.9	29.2	2.9
		180	3.82	7.1	3.11	9.3	41.9	0.9	33.5	2.4
		360	4.06	10.8	3.31	9.0	45.3	1.5	33.6	6.7
	L-B	7	2.33	12.0	2.45	9.6	17.3	2.5	19.6	3.1
		28	2.73	5.0	3.00	8.7	27.1	1.9	33.7	1.5
		180	3.94	3.0	3.33	11.3	42.0	3.1	39.4	2.3
		360	4.15	9.1	3.32	8.8	49.4	2.3	44.2	3.1
	L-C	7	2.55	12.8	2.80	7.5	20.6	3.6	24.6	4.6
		28	3.40	6.0	3.70	13.3	31.1	2.7	37.5	1.6
		180	4.73	9.0	3.63	15.4	49.2	3.3	45.2	6.2
		360	4.55	8.1	3.44	9.6	50.5	1.8	49.9	4.6
	L-D	7	2.88	8.0	3.03	7.4	28.4	3.4	31.4	6.1
		28	3.48	7.3	3.75	6.3	40.2	2.5	45.8	2.4
		180	4.76	11.6	4.19	10.3	53.6	2.6	47.4	5.7
		360	4.83	11.8	4.08	12.1	58.1	3.0	50.4	6.7
Semi Lytag Concrete	L-1	7	2.12	4.6	2.38	6.1	10.1	2.9	10.7	2.6
		28	2.99	11.5	2.62	15.6	16.2	3.4	17.3	2.4
	L-2	7	3.68	12.3	-	-	20.6	1.7	-	-
		28	3.70	15.1	-	-	29.3	1.9	-	-
	L-3	7	4.02	6.8	-	-	26.6	2.2	-	-
		28	4.20	13.2	-	-	35.8	4.2	-	-
	L-4	7	4.63	15.3	4.47	7.2	29.6	3.1	31.1	2.3
		28	4.93	7.1	5.47	18.4	41.9	0.6	43.6	2.7
Pellite Concrete	P-1	7	2.02	11.1	-	-	12.1	1.3	-	-
		28	2.91	9.6	-	-	17.9	6.3	-	-
	P-2	14	3.53	15.4	-	-	26.6	2.1	-	-
		28	3.45	7.0	-	-	30.5	5.5	-	-
	P-3	7	3.31	10.3	-	-	23.7	5.4	-	-
		28	3.71	3.9	-	-	32.6	0.7	-	-
	P-5	14	4.10	13.5	-	-	38.1	2.3	-	-
		28	4.37	16.9	-	-	42.6	2.1	-	-

Table B-8:Direct pull internal fracture test results for different types of lightweight concrete

Type of Concrete	Mix No.	Age Days	Pull Out Force KN				Cube Compressive Strength N/mm^2			
			Wet		Dry		Wet		Dry	
			F	c.v %	F	c.v %	f_c	c.v %	f_c	c.v %
Fully Lytag Concrete	L-A	7	3.22	9.8	3.70	6.6	13.5	5.1	16.0	5.0
		28	4.40	9.5	5.07	9.5	24.3	2.9	29.2	2.9
		180	6.22	11.4	5.25	13.8	41.9	0.9	33.5	2.4
		360	6.89	7.7	5.89	14.7	45.3	1.5	33.6	6.7
	L-B	7	3.90	8.1	4.05	5.3	17.3	2.5	19.6	3.1
		28	5.18	10.8	5.31	8.8	27.1	1.9	33.7	1.5
		180	7.21	9.8	6.23	9.2	42.0	3.1	39.4	2.3
		360	7.62	7.2	6.47	12.1	49.4	2.3	44.2	3.1
	L-C	7	4.52	8.9	4.64	6.6	20.6	3.6	24.6	4.6
		28	5.70	3.8	6.11	5.9	31.1	2.7	37.5	1.6
		180	7.39	8.3	7.33	9.2	49.2	3.3	45.2	6.2
		360	7.05	9.3	6.32	11.3	50.5	1.8	49.9	4.6
	L-D	7	5.26	10.6	5.33	3.0	28.4	3.4	31.4	6.1
		28	6.64	3.8	6.73	3.7	40.2	2.5	45.8	2.4
		180	7.22	7.1	6.25	10.7	53.6	2.6	47.4	5.7
		360	7.99	3.9	7.58	16.8	58.1	3.0	50.4	6.7
Semi Lytag Concrete	L-1	7	3.32	19.9	3.39	7.1	10.1	2.9	10.7	2.6
		28	3.72	11.0	3.63	9.5	16.2	3.4	17.3	2.4
	L-2	7	4.47	7.2	-	-	20.6	1.7	-	-
		28	5.43	5.8	-	-	29.3	1.9	-	-
	L-3	7	5.29	14.8	-	-	26.6	2.2	-	-
		28	6.40	6.2	-	-	35.8	4.2	-	-
	L-4	7	6.14	11.4	5.87	8.9	29.6	3.1	31.1	2.3
		28	7.11	8.9	6.82	6.4	41.9	0.6	43.6	2.7
Leca+ Concrete	Le-1	7	1.38 (1.97)	30.0	-	-	9.5	5.4	-	-
		28	1.57 (3.29)	24.1	-	-	14.2	6.4	-	-
	Le-2	7	1.64 (4.73)	9.8	-	-	16.3	8.7	-	-
	Le-3	7	3.15 (4.85)	41.1	-	-	16.9	6.7	-	-
		28	2.27 (5.14)	48.0	-	-	20.3	6.3	-	-
	Le-4	7	3.31 (4.90)	12.2	-	-	21.1	5.0	-	-
		28	2.91 (5.40)	42.0	-	-	23.1	4.8	-	-
Pellite Concrete	P-1	7	2.94	10.7	2.99	13.5	12.1	1.3	13.0	2.8
		28	3.80	10.7	3.56	10.2	17.9	6.3	17.8	6.6
	P-2	14	4.81	12.0	5.03	8.3	26.6	2.1	28.4	1.5
		28	4.98	11.7	5.24	5.6	30.5	5.5	31.1	5.6
	P-3	7	5.10	10.8	4.73	9.4	23.7	5.4	24.9	4.4
		28	5.94	6.0	5.46	9.2	32.6	0.7	31.2	4.7
	P-5	14	6.05	8.0	6.12	8.9	38.1	2.3	40.8	0.5
		28	6.68	7.7	6.54	6.4	42.6	2.1	43.7	2.0

+;The values given in bracket represent the measured pull out resistance at the bottom face of cube specimens

Table B-9:Surface pull off test results for different types of concrete

Type of Concrete	Mix No.	Age Days	Pull Off Strength (N/mm^2) p	c.v %	150 mm Cube Compressive Strength N/mm^2 f_c	c.v %
Fully	L-A	7	2.77	6.1	20.1	3.1
Lytag		28	3.11	9.6	29.2	3.9
Concrete		180	3.14	7.5	32.4	1.2
		360	3.03	13.1	33.7	1.4
	L-B	7	2.86	8.6	21.1	2.6
		28	3.38	9.8	33.7	1.5
		180	4.02	6.4	43.3	4.5
		360	3.95	10.6	42.9	0.6
	L-C	7	3.51	6.3	29.6	6.1
		28	3.88	9.9	37.5	3.1
		180	4.50	12.6	50.3	3.8
		360	4.16	12.0	49.6	5.5
	L-D	7	3.54	6.4	34.4	3.4
		28	4.15	6.9	45.8	2.4
		180	4.66	5.7	57.1	5.6
		360	4.74	9.0	56.1	1.5
Semi	L-1	7	1.55	9.1	11.0	1.8
Lytag		28	2.06	10.4	19.0	0.9
Concrete	L-2	7^+	2.90	10.4	25.8	1.8
		28	3.62	6.2	33.6	1.5
	L-3	7	3.42	4.7	33.1	0.3
		28	4.17	12.0	40.9	3.9
	L-4	7	3.70	5.4	36.6	3.0
		28	4.39	6.9	45.6	2.6

Continued.........

Table B-9 (Contd.)

Type of Concrete	Mix No.	Age Days	Pull Off Strength (N/mm^2) p	c.v %	150 mm Cube Compressive Strength (N/mm^2) f_c	c.v %
Leca Concrete	Le-1	14	1.65	15.9	13.1	5.0
		28	1.77	8.1	13.5	3.8
		360	2.00	13.6	14.0	4.8
	Le-2	14	2.25	11.9	18.1	7.2
		28	2.35 (4.30)*	23.8	19.1	10.2
		360	2.98	26.2	21.5	8.4
	Le-3	14	2.84	10.3	19.7	8.5
		28	2.66 (4.66)*	9.5	19.8	4.5
		360	3.29	21.6	22.2	12.0
	Le-4	14	3.16	25.2	23.1	7.9
		28	3.14 (5.03)*	14.0	24.9	5.7
		360$^+$	3.29	20.3	25.2	9.5
Pellite Concrete	P-1	7	1.78	6.3	12.8	7.0
		28	1.88	7.2	18.4	2.4
	P-2	14	2.81	9.7	27.9	2.6
		28	3.15	12.9	31.6	2.1
	P-3	7	3.15	15.1	28.7	3.5
		28	3.40	7.3	36.6	4.5
	P-4	7	3.38	15.2	31.5	3.6
		28	3.68	9.0	40.0	4.3
	P-5	14	3.81	7.6	39.2	3.3
		28	4.20	9.7	44.1	8.3

Continued.........

Table B-9 (Contd.)

Type of Concrete	Mix No.	Age Days	Pull Off Strength (N/mm^2) p	c.v %	150 mm Cube Compressive Strength (N/mm^2) f_c	c.v %
Normal	N-1	7	1.95	4.3	13.5	5.5
Weight		28	2.25	12.5	19.3	5.0
Concrete		180	2.31	12.7	22.1	1.9
		360	2.35	12.8	20.6	4.2
	N-2	7	2.11	7.2	16.9	4.2
		28	2.39	5.8	22.9	1.8
		180	2.83	11.0	26.4	4.2
		360	2.51	14.4	24.4	5.3
	N-3	7	2.14	11.0	20.7	1.0
		28	2.64	11.0	28.7	2.7
		180	2.87	7.8	32.8	2.1
		360	2.94	16.9	32.5	8.7
	N-4	7	2.82	11.1	29.1	2.8
		28	3.06	17.0	38.3	2.8
		180	3.31	9.2	43.2	4.6
		360	3.35	12.2	43.7	2.9

+;Fully or partially debonding occured on one of the tests and the average of five readings was considered

*;Measured pull off resistance at the bottom face of cube specimens

Table B-10:Partial cored pull off test results for different types of lightweight concrete

Type of Concrete	Mix No.	Pull Off Strength (N/mm^2) p	c.v %	150 mm Cube Compressive Strength (N/mm^2) f_c	c.v %
Fully Lytag Concrete	L-A	2.38	5.7	30.0	4.7
	L-B	2.58	6.2	34.4	4.4
	L-C	2.64	6.4	41.7	2.2
	L-D	2.69	10.2	45.5	4.1
Semi Lytag Concrete	L-1	2.18	6.5	24.5	3.3
	L-2	2.86	9.4	35.2	3.3
	L-3	2.94	7.7	41.7	2.3
	L-4	3.21	9.8	45.3	4.6
Leca Concrete	Le-1	1.04	15.3	13.5	6.4
	Le-2	1.22	17.1	19.1	10.2
	Le-3	1.34	20.6	19.8	4.5
	Le-4	1.59	16.7	24.9	5.7
Pellite Concrete	P-1	1.65	9.0	18.4	2.4
	P-2	2.59	6.7	31.6	2.1
	P-3	2.55	6.0	36.6	4.5
	P-5	3.14	8.3	44.1	4.3

Table B-11:50 mm Core test results in vertical direction

Type of Concrete	Mix No.	Age Days	ℓ/d	Core Strength N/mm² Wet f_{cc}	Wet c.v %	Dry f_{cc}	Dry c.v %	Cube Compressive Strength N/mm² Wet f_c	Wet c.v %	Dry f_c	Dry c.v %
Fully Lytag Concrete	L-A	28	1.0	30.1	2.8	30.4	4.4	24.8	4.2	30.0	3.1
			1.4	27.2	1.6	30.1	2.8				
			1.6	26.3	1.6	30.2	7.5				
			2.0	25.0	1.1	28.0	3.6				
		180	2.0	38.5	5.8	33.1	9.1	42.6	2.8	27.9	8.2
		360	2.0	43.6	3.1	36.8	8.9	49.6	4.2	27.8	3.0
	L-B	28	1.0	30.6	2.9	34.9	7.4	29.1	4.4	31.9	4.7
			1.4	28.1	2.9	31.0	3.9				
			1.6	27.5	2.9	31.3	4.3				
			2.0	26.1	4.2	30.3	6.0				
	L-C	28	1.0	33.1	4.7	42.0	3.0	33.3	1.7	40.5	4.8
			1.4	30.6	2.3	38.7	8.3				
			1.6	29.1	3.1	36.5	7.4				
			2.0	28.8	5.2	37.2	2.3				
	L-D	28	1.0	43.9	5.0	46.9	5.1	41.6	5.1	47.2	7.5
			1.4	40.7	1.7	44.3	2.4				
			1.6	39.5	4.7	43.1	2.9				
			2.0	39.1	1.4	42.7	7.4				
		180	2.0	51.4	2.9	47.0	6.1	57.9	1.8	48.9	5.7
		360	2.0	53.5	1.4	47.9	9.3	58.4	2.1	51.4	8.5
Semi Lytag Concrete	L-1	28	1.0	23.2	4.1	-	-	20.7	3.6	-	-
			2.0	19.2	6.8	-	-				
	L-2	28	2.0	23.4	9.6	-	-	27.9	2.4	-	-
	L-3	28	2.0	34.5	1.3	-	-	36.8	1.6	-	-
	L-4	28	1.0	47.4	3.3	-	-	47.8	1.7	-	-
			2.0	41.9	1.2	-	-				
Leca Concrete	Le-1	28	1.0	11.2	17.6	-	-	15.1	5.1	-	-
			2.0	9.3	11.9						
	Le-2	28	2.0	11.6	17.3	-	-	20.7	2.9	-	-
	Le-3	28	2.0	13.3	11.4	-	-	21.6	5.8	-	-
	Le-4	28	1.0	15.2	7.9	-	-	25.8	7.5	-	-
			2.0	13.8	6.6						
Pellite Concrete	P-1	28	1.0	20.6	6.7	17.9	7.3	21.8	2.3	20.8	4.8
			2.0	16.3	1.5	14.9	5.5				
	P-2	28	2.0	31.9	5.2	28.8	6.9	36.5	1.5	35.2	0.2
	P-3	28	2.0	32.0	4.9	29.9	8.8	40.5	0.5	38.9	2.6
	P-5	28	1.0	45.0	1.2	42.4	4.3	46.4	3.1	45.4	2.9
			2.0	38.4	3.2	36.7	8.4				

Table B-12:50 mm Core test results in horizontal direction

Type of Concrete	Mix No.	Age (Days)	Curing Regime	ℓ/d	Core Strength (N/mm²)
Fully Lytag Concrete	L-A	360	Wet	2.0	52.6(4.1+)
			Dry	2.0	48.4(3.5+)
Semi Lytag Concrete	L-1	28	Wet	1.0	24.6(6.5+)
			Wet	2.0	22.3(7.0+)
	L-4	28	Wet	1.0	47.1(5.9+)
			Wet	2.0	39.4(2.2+)
Leca Concrete	Le-1	28	Wet	2.0	10.4(7.6+)
	Le-4	28	Wet	1.0	16.1(14.5+)
Pellite Concrete	P-1	28	Dry	1.0	19.2(6.3+)
				2.0	16.0(2.6+)
	P-5	28	Dry	1.0	42.1(3.4+)
				2.0	36.7(1.9+)

+;Coefficient of variation as percentage

APPENDIX C

****

SUPPLEMENTARY TABLES FOR CHAPTER 7.

C-1 ; Maximum principal stress for surface pull off test on normal weight concrete

C-2 ; Maximum principal stress for surface pull off test on Pellite concrete

C-3 ; Maximum principal stress for surface pull off test on semi Lytag concrete

C-4 ; Maximum principal stress for surface pull off test on fully Lytag concrete

C-5 ; Maximum principal stress for surface pull off test on Leca concrete

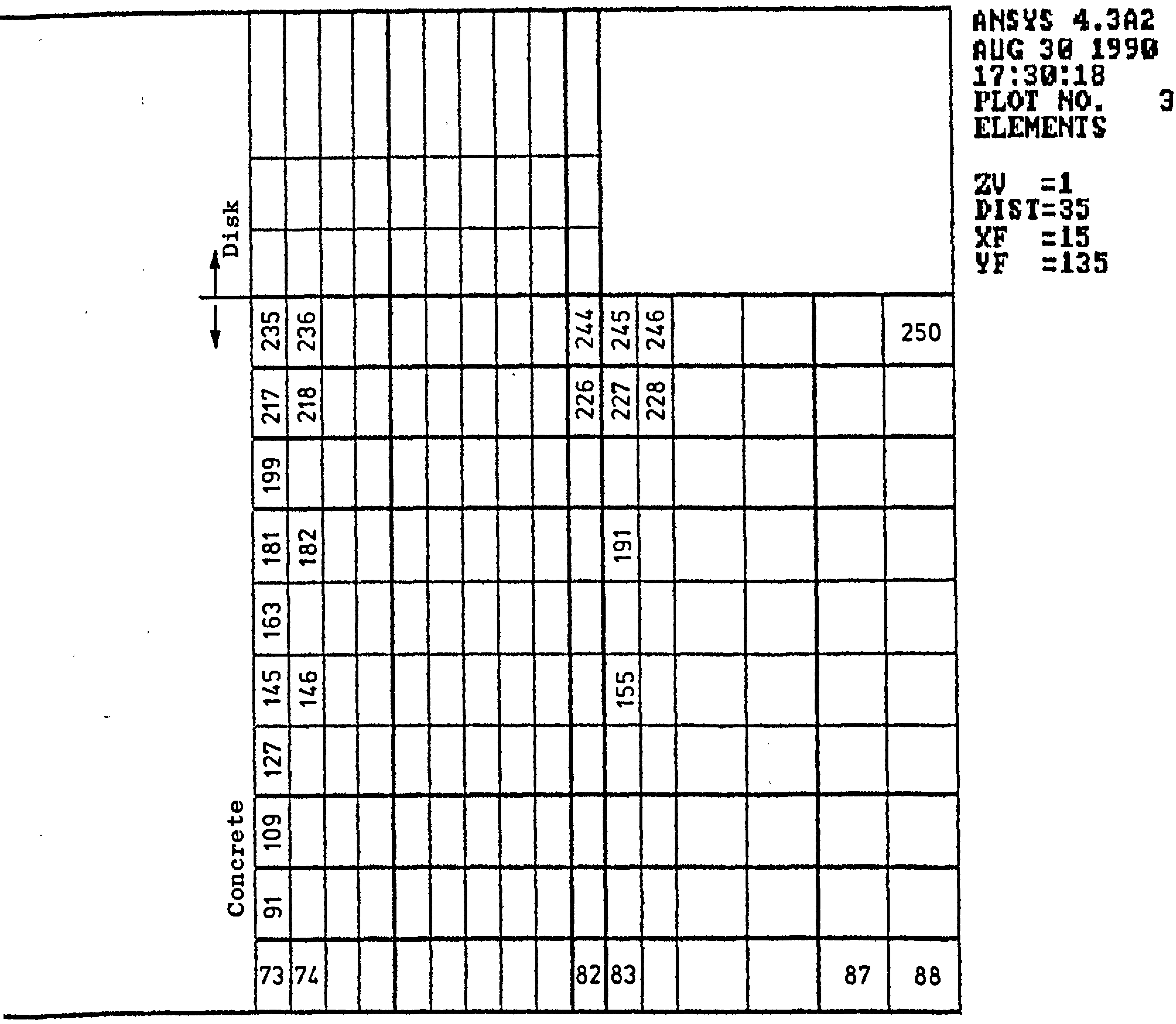

Figure C: Part of element mesh for surface pull off test

Table C-1;Maximum principal stress for surface pull off test on normal weight concrete

ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)
73	0.231	121	0.185	169	0.583	217	1.187
74	0.229	122	0.145	170	0.556	218	1.162
75	0.225	123	0.108	171	0.523	219	1.140
76	0.221	124	0.077	172	0.486	220	1.105
77	0.215	125	0.053	173	0.446	221	1.057
78	0.208	126	0.016	174	0.402	222	1.011
79	0.199	127	0.445	175	0.336	223	0.970
80	0.189	128	0.440	176	0.247	224	0.943
81	0.177	129	0.434	177	0.171	225	0.942
82	0.167	130	0.425	178	0.112	226	0.883
83	0.156	131	0.414	179	0.071	227	0.779
84	0.143	132	0.399	180	0.029	228	0.723
85	0.125	133	0.382	181	0.801	229	0.423
86	0.100	134	0.363	182	0.793	230	0.225
87	0.077	135	0.341	183	0.784	231	0.141
88	0.056	136	0.317	184	0.771	232	0.033
89	0.039	137	0.292	185	0.754	233	0.046
90	0.010	138	0.266	186	0.735	234	-0.014
91	0.288	139	0.226	187	0.711	235	1.568
92	0.285	140	0.174	188	0.679	236	1.528
93	0.281	141	0.128	189	0.641	237	1.467
94	0.275	142	0.090	190	0.597	238	1.399
95	0.268	143	0.061	191	0.545	239	1.312
96	0.258	144	0.020	192	0.493	240	1.215
97	0.247	145	0.547	193	0.399	241	1.130
98	0.235	146	0.542	194	0.285	242	1.073
99	0.221	147	0.535	195	0.186	243	1.073
100	0.206	148	0.525	196	0.109	244	1.209
101	0.191	149	0.512	197	0.066	245	1.158
102	0.175	150	0.494	198	0.029	246	0.457
103	0.152	151	0.474	199	0.963	247	0.169
104	0.120	152	0.450	200	0.951	248	0.055
105	0.091	153	0.423	201	0.937	249	-0.091
106	0.066	154	0.393	202	0.917	250	-0.135
107	0.045	155	0.361	203	0.892	251	-0.035
108	0.012	156	0.328	204	0.869	252	0.004
109	0.359	157	0.276	205	0.846		
110	0.355	158	0.209	206	0.822		
111	0.350	159	0.150	207	0.778		
112	0.342	160	0.103	208	0.724		
113	0.333	161	0.069	209	0.661		
114	0.321	162	0.025	210	0.568		
115	0.307	163	0.665	211	0.465		
116	0.292	164	0.660	212	0.304		
117	0.274	165	0.652	213	0.172		
118	0.256	166	0.641	214	0.098		
119	0.236	167	0.626	215	0.036		
120	0.216	168	0.607	216	0.017		

+;Element No. ,See Figure C

*;MPS=Maximum Principal Stress, in N/mm^2 with tension positive

Table C-2;Maximum principal stress for surface pull off test on Pellite concrete

ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)
73	0.227	121	0.183	169	0.570	217	1.098
74	0.224	122	0.144	170	0.545	218	1.078
75	0.221	123	0.108	171	0.515	219	1.060
76	0.217	124	0.077	172	0.480	220	1.033
77	0.211	125	0.052	173	0.443	221	0.996
78	0.204	126	0.015	174	0.399	222	0.963
79	0.196	127	0.433	175	0.335	223	0.936
80	0.186	128	0.429	176	0.247	224	0.926
81	0.176	129	0.423	177	0.171	225	0.943
82	0.165	130	0.415	178	0.113	226	0.894
83	0.154	131	0.404	179	0.072	227	0.794
84	0.142	132	0.391	180	0.029	228	0.747
85	0.124	133	0.375	181	0.760	229	0.436
86	0.099	134	0.356	182	0.754	230	0.230
87	0.076	135	0.335	183	0.747	231	0.144
88	0.056	136	0.313	184	0.738	232	0.034
89	0.039	137	0.288	185	0.725	233	0.046
90	0.010	138	0.263	186	0.711	234	-0.014
91	0.282	139	0.224	187	0.693	235	1.429
92	0.279	140	0.173	188	0.665	236	1.401
93	0.275	141	0.128	189	0.632	237	1.352
94	0.270	142	0.090	190	0.592	238	1.297
95	0.263	143	0.061	191	0.542	239	1.229
96	0.253	144	0.020	192	0.494	240	1.152
97	0.243	145	0.529	193	0.401	241	1.089
98	0.231	146	0.525	194	0.287	242	1.055
99	0.218	147	0.519	195	0.187	243	1.079
100	0.204	148	0.510	196	0.110	244	1.256
101	0.189	149	0.498	197	0.066	245	1.229
102	0.173	150	0.482	198	0.029	246	0.480
103	0.150	151	0.464	199	0.903	247	0.178
104	0.119	152	0.442	200	0.893	248	0.057
105	0.091	153	0.416	201	0.882	249	-0.089
106	0.066	154	0.388	202	0.868	250	-0.135
107	0.045	155	0.357	203	0.850	251	-0.035
108	0.012	156	0.325	204	0.835	252	0.004
109	0.351	157	0.274	205	0.822		
110	0.347	158	0.208	206	0.807		
111	0.342	159	0.150	207	0.771		
112	0.335	160	0.103	208	0.722		
113	0.326	161	0.069	209	0.665		
114	0.315	162	0.025	210	0.572		
115	0.302	163	0.638	211	0.472		
116	0.287	164	0.634	212	0.308		
117	0.270	165	0.628	213	0.175		
118	0.252	166	0.619	214	0.100		
119	0.233	167	0.606	215	0.037		
120	0.213	168	0.590	216	0.017		

+;Element No. ,See Figure C

*;MPS=Maximum Principal Stress, in N/mm^2 with tension positive

Table C-3;Maximum principal stress for surface pull off test on semi Lytag concrete

ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)
73	0.224	121	0.182	169	0.560	217	1.037
74	0.222	122	0.143	170	0.537	218	1.019
75	0.219	123	0.107	171	0.510	219	1.005
76	0.214	124	0.077	172	0.476	220	0.984
77	0.209	125	0.052	173	0.440	221	0.954
78	0.202	126	0.015	174	0.398	222	0.930
79	0.194	127	0.425	175	0.334	223	0.912
80	0.185	128	0.421	176	0.248	224	0.913
81	0.174	129	0.415	177	0.171	225	0.944
82	0.164	130	0.408	178	0.113	226	0.901
83	0.152	131	0.398	179	0.072	227	0.804
84	0.140	132	0.385	180	0.029	228	0.764
85	0.123	133	0.370	181	0.732	229	0.444
86	0.099	134	0.352	182	0.728	230	0.234
87	0.076	135	0.332	183	0.722	231	0.145
88	0.056	136	0.310	184	0.715	232	0.035
89	0.039	137	0.286	185	0.706	233	0.046
90	0.010	138	0.261	186	0.695	234	-0.014
91	0.278	139	0.223	187	0.680	235	1.332
92	0.275	140	0.173	188	0.656	236	1.310
93	0.271	141	0.127	189	0.626	237	1.269
94	0.266	142	0.089	190	0.589	238	1.225
95	0.259	143	0.060	191	0.541	239	1.168
96	0.250	144	0.019	192	0.494	240	1.106
97	0.240	145	0.517	193	0.401	241	1.058
98	0.228	146	0.513	194	0.288	242	1.040
99	0.215	147	0.508	195	0.188	243	1.081
100	0.202	148	0.499	196	0.111	244	1.289
101	0.187	149	0.489	197	0.066	245	1.280
102	0.172	150	0.474	198	0.029	246	0.496
103	0.149	151	0.457	199	0.861	247	0.185
104	0.119	152	0.436	200	0.853	248	0.058
105	0.090	153	0.412	201	0.845	249	-0.089
106	0.065	154	0.385	202	0.835	250	-0.135
107	0.045	155	0.354	203	0.822	251	-0.035
108	0.012	156	0.323	204	0.812	252	0.004
109	0.345	157	0.273	205	0.804		
110	0.341	158	0.208	206	0.797		
111	0.337	159	0.150	207	0.765		
112	0.330	160	0.102	208	0.721		
113	0.322	161	0.069	209	0.667		
114	0.311	162	0.025	210	0.575		
115	0.298	163	0.620	211	0.477		
116	0.283	164	0.616	212	0.311		
117	0.267	165	0.611	213	0.176		
118	0.250	166	0.603	214	0.100		
119	0.231	167	0.593	215	0.037		
120	0.211	168	0.579	216	0.017		

+;Element No. ,See Figure C

*;MPS=Maximum Principal Stress, in N/mm^2 with tension positive

Table C-4;Maximum principal stress for surface pull off test on fully Lytag concrete

ELEM. (+)	MPS (*)	ELEM. (+)	MPS. (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)
73	0.222	121	0.181	169	0.555	217	1.000
74	0.220	122	0.143	170	0.533	218	0.984
75	0.217	123	0.107	171	0.506	219	0.972
76	0.213	124	0.076	172	0.474	220	0.954
77	0.207	125	0.052	173	0.438	221	0.929
78	0.200	126	0.015	174	0.397	222	0.910
79	0.192	127	0.420	175	0.334	223	0.898
80	0.183	128	0.416	176	0.248	224	0.905
81	0.173	129	0.411	177	0.171	225	0.944
82	0.163	130	0.404	178	0.113	226	0.905
83	0.151	131	0.394	179	0.072	227	0.809
84	0.140	132	0.382	180	0.029	228	0.774
85	0.122	133	0.367	181	0.716	229	0.449
86	0.098	134	0.349	182	0.712	230	0.236
87	0.076	135	0.330	183	0.708	231	0.146
88	0.055	136	0.308	184	0.702	232	0.035
89	0.038	137	0.284	185	0.694	233	0.046
90	0.010	138	0.260	186	0.686	234	-0.014
91	0.276	139	0.222	187	0.673	235	1.274
92	0.273	140	0.172	188	0.650	236	1.256
93	0.269	141	0.127	189	0.622	237	1.220
94	0.264	142	0.089	190	0.586	238	1.181
95	0.257	143	0.060	191	0.540	239	1.131
96	0.249	144	0.019	192	0.494	240	1.077
97	0.238	145	0.510	193	0.402	241	1.038
98	0.227	146	0.506	194	0.289	242	1.030
99	0.214	147	0.501	195	0.188	243	1.082
100	0.200	148	0.493	196	0.111	244	1.309
101	0.186	149	0.483	197	0.066	245	1.312
102	0.171	150	0.470	198	0.029	246	0.505
103	0.148	151	0.453	199	0.837	247	0.189
104	0.118	152	0.433	200	0.830	248	0.058
105	0.090	153	0.409	201	0.823	249	-0.088
106	0.065	154	0.382	202	0.815	250	-0.135
107	0.045	155	0.353	203	0.805	251	-0.035
108	0.012	156	0.322	204	0.798	252	0.004
109	0.342	157	0.273	205	0.794		
110	0.338	158	0.208	206	0.791		
111	0.334	159	0.149	207	0.762		
112	0.327	160	0.102	208	0.720		
113	0.319	161	0.068	209	0.668		
114	0.308	162	0.025	210	0.577		
115	0.296	163	0.609	211	0.480		
116	0.281	164	0.606	212	0.313		
117	0.265	165	0.601	213	0.177		
118	0.248	166	0.594	214	0.101		
119	0.230	167	0.585	215	0.037		
120	0.210	168	0.572	216	0.017		

+;Element No. ,See Figure C

*;MPS=Maximum Principal Stress, in N/mm^2 with tension positive

Table C-5;Maximum principal stress for surface pull off test on Leca concrete

ELEM. (+)	MPS (*)	ELEM. (+)	MPS. (*)	ELEM. (+)	MPS (*)	ELEM. (+)	MPS (*)
73	0.206	121	0.169	169	0.511	217	0.882
74	0.204	122	0.133	170	0.493	218	0.869
75	0.201	123	0.100	171	0.469	219	0.861
76	0.197	124	0.071	172	0.440	220	0.849
77	0.192	125	0.049	173	0.408	221	0.832
78	0.186	126	0.014	174	0.370	222	0.821
79	0.178	127	0.386	175	0.312	223	0.820
80	0.170	128	0.383	176	0.232	224	0.836
81	0.161	129	0.378	177	0.161	225	0.884
82	0.151	130	0.372	178	0.106	226	0.853
83	0.141	131	0.363	179	0.067	227	0.766
84	0.130	132	0.352	180	0.027	228	0.741
85	0.114	133	0.339	181	0.646	229	0.428
86	0.092	134	0.323	182	0.644	230	0.225
87	0.071	135	0.305	183	0.641	231	0.139
88	0.052	136	0.286	184	0.638	232	0.033
89	0.036	137	0.264	185	0.633	233	0.043
90	0.009	138	0.242	186	0.628	234	-0.013
91	0.255	139	0.207	187	0.619	235	1.105
92	0.252	140	0.161	188	0.601	236	1.093
93	0.249	141	0.119	189	0.577	237	1.066
94	0.244	142	0.084	190	0.546	238	1.037
95	0.238	143	0.056	191	0.504	239	1.002
96	0.230	144	0.018	192	0.462	240	0.964
97	0.221	145	0.467	193	0.377	241	0.941
98	0.210	146	0.464	194	0.272	242	0.949
99	0.199	147	0.459	195	0.177	243	1.013
100	0.186	148	0.453	196	0.104	244	1.257
101	0.173	149	0.444	197	0.062	245	1.279
102	0.159	150	0.433	198	0.027	246	0.488
103	0.138	151	0.418	199	0.747	247	0.183
104	0.110	152	0.400	200	0.742	248	0.056
105	0.084	153	0.379	201	0.738	249	-0.082
106	0.061	154	0.355	202	0.733	250	-0.126
107	0.042	155	0.328	203	0.728	251	-0.033
108	0.011	156	0.299	204	0.727	252	0.004
109	0.315	157	0.254	205	0.728		
110	0.312	158	0.194	206	0.731		
111	0.308	159	0.140	207	0.708		
112	0.302	160	0.096	208	0.673		
113	0.295	161	0.064	209	0.628		
114	0.285	162	0.023	210	0.542		
115	0.274	163	0.554	211	0.454		
116	0.261	164	0.552	212	0.296		
117	0.246	165	0.549	213	0.167		
118	0.230	166	0.543	214	0.095		
119	0.213	167	0.536	215	0.035		
120	0.196	168	0.526	216	0.016		

+;Element No. ,See Figure C

*;MPS=Maximum Principal Stress, in N/mm^2 with tension positive

APPENDIX D
****

SUPPLEMENTARY TABLES FOR CHAPTER 8.

Table D-1:Equivalent cube strength for fully Lytag concrete based on non-destructive test results(at the age of 28 days)

Test Location	Rebound Hammer R	Pulse Velocity V(Km/sec)	Equivalent Cube Strength(N/mm^2) based on Rebound Hammer	Equivalent Cube Strength(N/mm^2) based on Pulse Velocity
A-1	42.9	3.63	55.1	43.8
A-2	39.8	3.62	46.7	42.2
A-3	40.3	3.58	48.2	36.5
A-4	34.0	3.61	33.4	40.7
A-5	39.7	3.62	46.5	42.2
A-6	40.9	3.62	49.6	42.2
A-7	42.7	3.59	54.5	37.9
A-8	38.3	3.62	43.1	42.2
A-9	40.2	3.61	47.9	40.7
A-10	38.6	3.61	43.9	40.7
B-1		3.60		39.3
B-2		3.60		39.3
B-3		3.61		40.7
B-4		3.59		37.9
B-5		3.61		40.7
B-6		3.61		40.7
B-7		3.62		42.2
B-8		3.61		40.7
B-9		3.58		36.5
B-10		3.60		39.3
C-1	39.7	3.65	46.5	47.1
C-2	36.7	3.59	39.4	37.9
C-3	33.6	3.62	32.4	42.2
C-4	38.4	3.62	43.4	42.2
C-5	39.9	3.68	47.0	52.6
C-6	38.0	3.63	42.3	43.8
C-7	39.3	3.62	45.6	42.2
C-8	33.4	3.63	32.2	43.8
C-9	35.4	3.60	36.3	39.3
C-10	40.5	3.61	48.5	40.7
D-1		3.63		43.8
D-2		3.60		39.3
D-3		3.61		40.7
D-4		3.62		42.2
D-5		3.63		43.8
D-6		3.61		40.7
D-7		3.62		42.2
D-8		3.64		45.4
D-9		3.60		39.3
D-10		3.61		40.7
E-1	36.7	3.65	39.4	47.1
E-2	39.3	3.65	45.6	47.1
E-3	37.0	3.65	39.9	47.1
E-4	38.3	3.60	43.1	39.3
E-5	41.0	3.64	49.9	45.4
E-6	37.2	3.66	40.4	48.9
E-7	33.8	3.61	32.9	40.7
E-8	36.5	3.64	38.8	45.4
E-9	37.4	3.58	40.9	36.5
E-10	40.2	3.64	47.9	45.4

Table D-2:Equivalent cube strength for semi Lytag concrete based on non-destructive test results(at the age of 28 days)

Test Location	Rebound Hammer R	Pulse Velocity V(Km/sec)	Equivalent Cube Strength(N/mm^2) based on Rebound Hammer	Equivalent Cube Strength(N/mm^2) based on Pulse Velocity
A-1	31.2	3.78	33.0	27.8
A-2	31.3	3.76	33.2	26.1
A-3	30.0	3.76	30.4	26.1
A-4	33.0	3.78	37.1	27.8
A-5	32.6	3.76	36.1	26.1
A-6	30.9	3.76	32.3	26.1
A-7	31.4	3.76	33.4	26.1
A-8	30.6	3.76	31.6	26.1
A-9	29.4	3.73	29.1	23.8
A-10	27.7	3.77	25.7	27.0
B-1		3.81		30.5
B-2		3.77		27.0
B-3		3.78		27.8
B-4		3.79		28.7
B-5		3.83		32.5
B-6		3.77		27.0
B-7		3.78		27.8
B-8		3.75		25.3
B-9		3.77		27.0
B-10		3.79		28.7
C-1	33.1	3.86	37.3	35.7
C-2	33.9	3.81	39.2	30.5
C-3	31.0	3.80	32.5	29.6
C-4	32.6	3.83	36.1	32.5
C-5	31.6	3.87	33.8	36.8
C-6	33.7	3.78	38.8	27.8
C-7	33.2	3.83	37.6	32.5
C-8	34.2	3.80	40.0	29.6
C-9	34.7	3.77	41.2	27.0
C-10	34.2	3.83	40.0	32.5
D-1		3.87		36.8
D-2		3.83		32.5
D-3		3.83		32.5
D-4		3.86		35.7
D-5		3.85		34.6
D-6		3.85		34.6
D-7		3.82		31.5
D-8		3.87		36.8
D-9		3.78		27.8
D-10		3.84		33.5
E-1	35.4	3.89	43.0	39.2
E-2	34.5	3.89	40.7	39.2
E-3	35.5	3.88	43.2	38.0
E-4	32.5	3.91	35.9	41.7
E-5	34.7	3.93	41.2	44.4
E-6	36.5	3.96	45.8	48.8
E-7	36.4	3.92	45.6	43.0
E-8	36.5	3.93	45.8	44.4
E-9	37.3	3.88	48.0	38.0
E-10	35.7	3.90	43.7	40.4

Table D-3:Equivalent cube strength for Leca concrete based on non-destructive test results(at the age of 28 days)

Test Location	Rebound Hammer R	Pulse Velocity V(Km/sec)	Equivalent Cube Strength(N/mm^2) based on Rebound Hammer	Equivalent Cube Strength(N/mm^2) based on Pulse Velocity
A-1	25.9	3.21	29.0	19.2
A-2	22.8	3.22	26.1	19.4
A-3	27.3	3.19	30.3	18.8
A-4	27.5	3.21	30.4	19.2
A-5	23.1	3.21	26.4	19.2
A-6	25.2	3.30	28.3	21.2
A-7	21.6	3.22	25.0	19.4
A-8	28.8	3.22	31.6	19.4
A-9	26.4	3.23	29.4	19.7
A-10	26.4	3.26	29.4	20.3
B-1		3.31		21.4
B-2		3.33		21.9
B-3		3.30		21.2
B-4		3.30		21.2
B-5		3.27		20.5
B-6		3.32		21.6
B-7		3.35		22.3
B-8		3.31		21.4
B-9		3.33		21.9
B-10		3.38		23.1
C-1	27.5	3.35	30.4	22.3
C-2	28.9	3.38	31.7	23.1
C-3	25.0	3.39	28.2	23.3
C-4	24.7	3.51	27.9	26.5
C-5	26.0	3.30	29.1	21.2
C-6	25.8	3.37	28.9	22.8
C-7	23.8	3.37	27.0	22.8
C-8	26.3	3.39	29.4	23.3
C-9	29.4	3.37	32.2	22.8
C-10	26.1	3.39	29.2	23.3
D-1		3.45		24.9
D-2		3.44		24.6
D-3		3.54		27.4
D-4		3.38		23.1
D-5		3.39		23.3
D-6		3.42		24.1
D-7		3.43		24.3
D-8		3.58		28.6
D-9		3.39		23.3
D-10		3.44		24.6
E-1	27.3	3.58	30.3	28.6
E-2	28.1	3.61	31.0	29.5
E-3	26.7	3.51	29.7	26.5
E-4	26.8	3.58	29.8	28.6
E-5	25.7	3.63	28.8	30.1
E-6	28.4	3.51	31.3	26.5
E-7	29.3	3.56	32.1	28.0
E-8	24.8	3.52	28.0	26.8
E-9	25.3	3.52	28.4	26.8
E-10	27.1	3.55	30.1	27.7

Table D-4:Equivalent cube strength for Pellite concrete based on non-destructive test results(at the age of 28 days)

Test Location	Rebound Hammer R	Pulse Velocity V(Km/sec)	Equivalent Cube Strength(N/mm^2) based on Rebound Hammer	Equivalent Cube Strength(N/mm^2) based on Pulse Velocity
A-1	31.9	4.23	44.6	38.3
A-2	32.9	4.20	48.3	35.8
A-3	31.6	4.18	43.6	34.2
A-4	31.7	4.19	43.9	35.0
A-5	31.6	4.16	43.6	32.7
A-6	32.5	4.17	46.8	33.4
A-7	33.0	4.21	48.6	36.6
A-8	33.6	4.18	50.9	34.2
A-9	32.2	4.20	45.7	35.8
A-10	32.1	4.23	45.3	38.3
B-1		4.21		36.6
B-2		4.19		35.0
B-3		4.20		35.8
B-4		4.18		34.2
B-5		4.19		35.0
B-6		4.22		37.4
B-7		4.22		37.4
B-8		4.20		35.8
B-9		4.20		35.8
B-10		4.21		36.6
C-1	33.8	4.27	51.7	41.9
C-2	33.2	4.19	49.4	35.0
C-3	31.2	4.18	42.2	34.2
C-4	31.2	4.17	42.2	33.4
C-5	32.8	4.19	47.9	35.0
C-6	32.1	4.22	45.3	37.4
C-7	32.0	4.24	45.0	39.2
C-8	34.0	4.20	52.5	35.8
C-9	32.8	4.20	47.9	35.8
C-10	33.5	4.24	50.5	39.2
D-1		4.32		46.9
D-2		4.27		41.9
D-3		4.30		44.9
D-4		4.23		38.3
D-5		4.28		42.9
D-6		4.24		39.2
D-7		4.22		37.4
D-8		4.24		39.2
D-9		4.25		40.1
D-10		4.27		41.9
E-1	33.3	4.31	49.8	45.9
E-2	34.1	4.28	52.8	42.9
E-3	33.2	4.28	49.4	42.9
E-4	34.0	4.26	52.5	41.0
E-5	35.6	4.28	58.9	42.9
E-6	34.6	4.28	54.8	42.9
E-7	34.6	4.27	54.8	41.9
E-8	34.2	4.29	53.2	43.9
E-9	34.2	4.28	53.2	42.9
E-10	33.7	4.29	51.3	43.9

Table D-5:Equivalent cube strength for normal weight concrete based on non-destructive test results(at the age of 28 days)

Test Location	Rebound Hammer R	Pulse Velocity V(Km/sec)	Equivalent Cube Strength(N/mm^2) based on Rebound Hammer	Equivalent Cube Strength(N/mm^2) based on Pulse Velocity
A-1	32.3	4.60	37.2	32.0
A-2	30.9	4.53	34.0	26.9
A-3	30.3	4.46	32.6	32.2
A-4	29.3	4.48	30.5	24.4
A-5	30.1	4.54	32.2	27.6
A-6	29.8	4.54	31.5	27.6
A-7	32.1	4.41	36.7	20.5
A-8	29.6	4.55	31.1	28.3
A-9	30.7	4.45	33.5	22.6
A-10	32.6	4.59	37.9	31.2
B-1		4.57		29.7
B-2		4.46		23.2
B-3		4.48		24.4
B-4		4.54		27.6
B-5		4.44		22.1
B-6		4.42		21.0
B-7		4.44		22.1
B-8		4.44		22.1
B-9		4.41		20.5
B-10		4.55		28.3
C-1	30.1	4.56	32.2	29.0
C-2	32.0	4.58	36.5	30.4
C-3	29.0	4.52	29.8	26.2
C-4	30.8	4.56	33.7	29.0
C-5	29.8	4.49	31.5	25.0
C-6	32.4	4.51	37.4	25.6
C-7	29.9	4.44	31.8	22.1
C-8	30.7	4.47	33.5	23.8
C-9	29.9	4.58	31.8	30.4
C-10	30.4	4.58	32.8	30.4
D-1		4.67		38.0
D-2		4.73		42.9
D-3		4.64		35.3
D-4		4.60		32.0
D-5		4.70		39.9
D-6		4.60		32.0
D-7		4.56		29.0
D-8		4.56		29.0
D-9		4.64		35.3
D-10		4.58		30.4
E-1	34.2	4.75	41.8	45.1
E-2	37.3	4.75	49.9	45.1
E-3	37.0	4.69	49.0	38.9
E-4	38.3	4.64	52.6	35.3
E-5	36.1	4.67	46.6	38.0
E-6	36.8	4.72	48.5	41.9
E-7	34.6	4.69	42.8	38.9
E-8	36.4	4.67	47.4	38.0
E-9	35.9	4.65	46.1	36.1
E-10	33.8	4.61	40.8	32.7

Table D-6:Equivalent cube strength for fully Lytag concrete based on Lok-Test results(at age of 28 days)

Test	Lok Force(KN)		Equivalent Cube Strength(N/mm^2)	
Location	Front Side	Back Side	Front Side	Back Side
I-1	16.7	18.9	29.9	34.8
I-2	16.7	19.4	29.9	35.9
I-3	15.5	19.8	27.2	36.8
I-4	20.3	17.0	37.9	30.6
I-5	17.5	17.0	31.7	30.6
J-1	15.7	19.3	27.7	35.7
J-2	16.5	18.2	29.4	33.2
J-3	18.5	18.7	33.9	34.3
J-4	19.0	20.9	35.0	39.2
J-5	18.2	17.5	33.2	31.7
K-1	19.7	20.8	36.5	39.0
K-2	21.7	20.0	41.0	37.2
K-3	19.0	20.9	35.0	39.2
K-4	18.3	21.6	33.4	40.8
K-5	17.5	19.8	31.7	36.8

Table D-7:Equivalent cube strength for semi Lytag concrete based on Lok-Test results(at the age of 28 days)

Test	Lok Force(KN)		Equivalent Cube Strength(N/mm^2)	
Location	Front Side	Back Side	Front Side	Back Side
I-1	18.0	15.7	32.9	28.5
I-2	16.5	17.5	30.0	32.0
I-3	16.0	15.3	29.0	27.7
I-4	12.8	13.9	22.8	25.0
I-5	11.0	15.4	19.3	27.9
J-1	19.8	17.6	36.4	32.2
J-2	17.8	18.0	32.5	32.9
J-3	19.9	20.1	36.6	37.0
J-4	18.2	17.2	33.3	31.4
J-5	18.2	16.0	33.3	29.1
K-1	22.0	23.0	40.7	42.6
K-2	21.7	22.7	40.1	42.1
K-3	23.5	24.7	43.6	45.9
K-4	22.4	19.9	41.5	36.6
K-5	22.8	21.3	42.2	39.3

Table D-8:Equivalent cube strength for Leca concrete based on LOk-Test results(at age of 28 days)

Test Location	Lok Force(KN) Front Side	Lok Force(KN) Back Side	Equivalent Cube Strength(N/mm^2) Front Side	Equivalent Cube Strength(N/mm^2) Back Side
I-1	10.0	12.8	17.4	22.4
I-2	10.5	10.7	18.3	18.6
I-3	12.0	9.8	21.0	17.0
I-4	10.0	16.0	17.4	28.2
I-5	11.7	11.0	20.5	19.2
J-1	15.0	15.0	26.4	26.4
J-2	15.5	13.0	27.3	22.8
J-3	16.0	12.7	28.2	22.3
J-4	15.2	17.5	26.8	30.9
J-5	16.5	14.5	29.1	25.5
K-1	17.0	17.0	30.0	30.0
K-2	18.7	17.0	33.1	30.0
K-3	18.0	15.0	31.8	26.4
K-4	17.8	15.0	31.5	26.4
K-5	17.3	20.0	30.6	35.5

Table D-9:Equivalent cube strength for Pellite concrete based on Lok-Test results(at the age of 28 days)

Test Location	Lok Force(KN) Front Side	Lok Force(KN) Back Side	Equivalent Cube Strength(N/mm^2) Front Side	Equivalent Cube Strength(N/mm^2) Back Side
I-1	15.5	19.9	26.1	34.8
I-2	20.0	20.5	35.0	36.0
I-3	19.2	16.0	33.4	27.1
I-4	18.7	20.7	32.4	36.4
I-5	18.0	18.8	31.0	32.6
J-1	21.0	19.9	36.9	34.8
J-2	21.8	22.5	38.5	39.9
J-3	20.6	21.2	36.2	37.3
J-4	19.5	18.0	34.0	31.0
J-5	19.9	20.0	34.8	35.0
K-1	24.0	26.7	42.9	48.2
K-2	22.5	20.8	39.9	36.6
K-3	25.2	25.2	45.2	45.2
K-4	27.0	24.0	48.8	42.9
K-5	24.0	21.0	42.9	37.0

Table D-10: Equivalent cube strength for normal weight concrete based on Lok-Test results (at age of 28 days)

Test Location	Lok Force (KN) Front Side	Lok Force (KN) Back Side	Equivalent Cube Strength (N/mm^2) Front Side	Equivalent Cube Strength (N/mm^2) Back Side
I-1	19.2	20.1	22.2	23.6
I-2	21.7	19.1	26.1	22.0
I-3	22.8	19.3	27.8	22.3
I-4	23.6	20.4	29.0	24.0
I-5	20.7	18.2	24.5	20.6
J-1	23.8	22.1	29.3	26.7
J-2	26.6	24.7	33.7	30.7
J-3	20.3	22.6	23.9	27.5
J-4	22.6	29.7	27.5	38.5
J-5	21.4	22.8	25.6	27.8
K-1	34.4	32.1	45.9	42.3
K-2	28.9	29.9	37.3	38.8
K-3	27.0	27.7	34.3	35.4
K-4	31.8	29.7	41.8	38.5
K-5	32.8	26.5	43.4	33.5

Table D-11: Equivalent cube strength for fully Lytag concrete based on insitu test results (at the age of 360 days)

Test Location	Pulse Velocity Km/sec	Capo Force KN Front Side	Capo Force KN Back Side	Core^{+} Strength ($\ell/d=2.0$) N/mm^2	Equivalent Cube Strength (N/mm^2) based on Pulse Velocity	Capo-Test Front Side	Capo-Test Back Side	Cores^{+}
A-1	3.68				52.7			
A-2	3.72			51.9	57.6			57.0
A-3		25.8	24.8			48.7	45.8	
A-4		23.7	25.7			42.7	48.4	
A-5	3.71				56.3			
A-6		23.0	25.0			40.7	46.4	
A-7	3.73			51.3	58.9			56.3
A-8		25.5	23.0			47.8	40.7	
A-9	3.68				52.7			
A-10	3.71			49.9 (43.0)^{x}	56.3			54.6
C-1	3.73			53.1	58.9			58.5
C-2								
C-3	3.74	26.0	26.3		60.2	49.2	50.1	
C-4								
C-5								
C-6	3.73			54.0	58.9			59.5
C-7								
C-8		27.0	25.0			52.1	46.4	
C-9								
C-10	3.73				58.9			
E-1	3.75			56.0	61.5			61.9
E-2								
E-3		29.0	26.0			57.8	49.2	
E-4	3.73			55.2	58.9			61.0
E-5								
E-6								
E-7		27.0	26.3			52.1	50.1	
E-8	3.76	27.7	27.0		62.9	54.1	52.1	
E-9								
E-10	3.77			56.7 (52.5)^{x}	64.3			62.8

+; Average of 2 tests
x; $\ell/d=1.0$

Table D-12:Within-test variation for Lok-test results on semi Lytag concrete

Lok Force; KN
19.8
19.5
21.7
20.8
23.3
20.5
20.8
21.9
22.8
20.7
24.6
22.5

Average=21.6 KN
Coefficient of Variation= 7.0 %

Table D-13:Within-test variation for cube strength results on semi Lytag concrete

Cube Strength;N/mm^2
40.5
38.5
38.7
40.6
39.7
38.4
41.6
37.3
39.1
40.3
37.0
39.9

Average=39.3
Coefficient of Variation=3.5 %

APPENDIX E

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PUBLICATIONS

(1) ; Insitu Strength Assessment of Lightweight Concrete
IABSE Symposium, Lisbon, Sept. 1989, pp. 847-852.

(2) ; Partially Destructive Testing of Lightweight Concrete
Proc. Int. Conference on Structural Faults and Repair
Vol.1, London, June 1989, pp. 25-31.

In-situ Strength Assessment of Lightweight Concrete

Evaluation in situ de la résistance du béton léger

Festigkeitsbestimmung an Bauwerken aus Leichtbeton

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SUMMARY

A range of non-destructive and partially-destructive test methods have been examined in terms of their reliability when used for in-situ strength assessment of lightweight concrete. These may be used with confidence provided that specially developed correlation curves are available. Testing variability has been found to be generally lower than for concrete with normal weight aggregates, possibly due to differences in failure mechanisms associated with the use of relatively weak aggregate particles.

RÉSUMÉ

Plusieurs méthodes d'essai non-destructives et partiellement destructives ont été examinées selon leur fiabilité lorsqu'elles sont utilisées pour l'évaluation in-situ de la résistance du béton léger. Celles-ci peuvent être utilisées avec confiance pourvu que des courbes de corrélation spécialement développées soient disponibles. Les variations obtenues lors des essais sont généralement de moindre importance que celles obtenues avec du béton normal, cela peut être lié aux différences dans les mécanismes de rupture associés à l'utilisation de particules d'agrégats relativement faibles.

ZUSAMMENFASSUNG

Untersucht wurden verschiedene nicht-zerstörende und teilweise-zerstörende Testmethoden im Hinblick auf ihre Verlässlichkeit bei der Bestimmung der Festigkeit von Ortbeton und Leichtbeton. Die Methoden können ohne weiteres angewendet werden, sofern für diesen Zweck entwickelte Korrelationskurven zur Verfügung stehen. Es wurde festgestellt, dass die Testabweichungen insgesamt niedriger sind als für Beton mit normalen Gewichts-Zuschlagstoffen, möglicherweise aufgrund der Unterschiede bei Fehlermechanismen, die mit der Verwendung relativ schwacher Zuschlagkörner einhergehen.

1. INTRODUCTION

It is now recognized that insitu strength evaluation of concrete by means of non-destructive and partially destructive methods has an important role to play in the building and civil engineering industries. These techniques have a wide range of applications when evaluating structural deficiencies and details of their use are given elsewhere [1].

Assessment of the strength of the concrete in structures has received considerable attention relating to natural dense aggregates, whilst concrete made of lightweight aggregates has received only limited attention. Lightweight concrete has proved itself to be a useful structural material, and applications are becoming more numerous as Engineers gain confidence. Most lightweight aggregates are artificially manufactured, and in the UK the most widely available material suitable for structural concrete is Lytag. This is produced from pulverised fuel ash (Pfa), by a sintering process [2].

A comprehensive experimental programme is being undertaken to examine the reliability and mechanisms of different methods applied to a range of lightweight concretes. In this paper the most important results obtained by six different test methods applied to fully lightweight concrete are presented.

2. EXPERIMENTAL PROGRAMME

An Ordinary Portland cement together with coarse and fine Lytag satisfying the relevant British Standards were used for all the mixes. The 24 hour water absorptions (based on oven-dried condition) for coarse and fine Lytag were 12% and 15% respectively. Four different mixes were designed with 28-day cube strengths between about 23 - 47 N/mm^2. For each mix, the following specimens were cast in four batches; 650 x 225 x 120 mm beams for 50 mm cores, 225 mm cubes for pull-out, 150 mm cubes for internal fracture and pull-off, and 100 mm cubes for pulse velocity testing.

All specimens were compacted on a vibrating table and left in the laboratory. Two curing regimes were adopted, wet and dry. Tests were carried out at ages of 7 and 28 days, except for the core tests which were performed at 28 days only.

Pull-out tests were performed on 25 mm diameter cast-in inserts using commercially available Lok test apparatus with procedures following the manufacturer's recommendations, whilst through transmission pulse velocity measurements were taken with widely used 'Pundit' equipment. The internal fracture tests using 6 mm diameter expanding wedge anchor bolts were carried out by using torquemeter apparatus (B.R.E.) as well as a modified form based on a direct pull. Pull-off tests were performed by gluing a 50 mm diameter aluminium disk to the surface of concrete followed by loading with commercially available Limpet apparatus. The 50 mm nominal diameter cores were cut vertically from the specified beams at the age of 28 days followed by trimming and capping to give overall length/diameter (L/D) ratios of 1.0, 1.4, 1.6 and 2.0. Detailed test procedures for all these methods are given elsewhere [1].

3. TEST RESULTS AND DISCUSSION

3.1 General

Table 1 summarises the average test results based on three readings for cores, pulse velocities and cube crushing strengths, and on six readings for the remaining methods. The cube compressive strengths have also been plotted against test results in figures 1 to 4. In all cases the relationship was found to be dependent on the age and curing conditions. With the exception of pulse velocities this dependency is small, and a single relationship could be adopted for practical purposes.

Mix	Age	100mm Cube Strength N/mm²		Core Strength L/D=2.0 N/mm²		Pull-Out Force kN		Internal Fracture B.R.E. N-m		Internal Fracture Direct Pull kN		Pull-Off Stress N/mm²	Pulse Vel. km/sec	
		Wet	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Dry	Wet	Dry
1	7	15.5	17.3	-	-	9.2	9.5	2.05	2.10	3.22	3.70	2.77	3.39	3.39
	28	23.9	29.4	25.0	28.0	14.9	17.4	2.33	2.56	4.40	5.07	3.11	3.53	3.47
2	7	19.7	21.7	-	-	10.5	10.6	2.33	2.45	3.90	4.05	2.86	3.53	3.51
	28	32.0	34.2	26.1	30.3	15.4	17.7	2.73	3.00	5.18	5.31	3.38	3.64	3.59
3	7	23.1	27.5	-	-	13.5	14.1	2.55	2.80	4.52	4.64	3.51	3.57	3.57
	28	35.0	39.7	28.8	37.2	19.4	22.5	3.40	3.70	5.70	6.11	3.88	3.68	3.60
4	7	29.0	33.3	-	-	16.4	16.9	2.88	3.03	5.26	5.33	3.54	3.56	3.60
	28	41.5	46.9	39.1	42.7	22.9	23.5	3.48	3.75	6.64	6.73	4.15	3.68	3.61

Table 1 Summary of test results on fully lightweight concrete

Test Method	Coefficient of Variation % Test Result	Coefficient of Variation % Normal Concrete	Correlation Coefficient	95% Confidence Limit on Estimated Strength
Core	4.3	8.8	0.985	±12%
Pull-Out	5.6	7.0	0.968	±17%
Internal Fracture				
B.R.E.	9.0	15.9	0.978	±34%
Direct Pull	9.8	15.6	0.987	±16%
Pull-Off	5.7	8.0	0.986	±24%

Table 2 Statistical evaluation for partially destructive tests

Statistical analyses based on the coefficient of variation have been summarized in table 2. It can be seen that these values are significantly less than those anticipated for normal weight concrete [1], however there are indications that within member material variability may be higher due to compaction differentials. Correlation coefficients given in table 2 based on single practical curves show that in general each test method applied to lightweight concrete gives a better correlation to cube strength than expected for normal weight concrete [1]. The accuracies of strength estimations based on 95% confidence limit for strength level of 30 N/mm² are also given in table 2. It is clearly seen that of the six insitu testing methods, the core test along with pull-out and direct pull internal fracture tests demonstrate the best ability to assess the insitu equivalent cube strength.

3.2 Core tests

As expected, core strengths were generally found to increase with decreasing length/diameter (L/D) ratio, although for dry cores the effect was not as large and not always as consistent as anticipated. This may be due to lack of uniformity in moisture content resulting from air drying, and emphasizes the importance of use of standardised specimens soaked for at least 48 hours. Correction factors to obtain the equivalent strength of a core with L/D = 2.0 are given in table 3. Comparison with the data for small cores of normal weight concrete reported by Bungey [1] suggests that considerably less correction is required for fully lightweight concrete. A similar finding has been obtained by Swamy [3] for semi-lightweight concrete. Recommended correction factors according to A.S.T.M. [4] and British Standards [5] are also included in table 3 and it can be seen that widely accepted British Standard values overestimate those required, even for wet specimens of lightweight concrete. Analysis of correction factors related to strength level also suggests that some dependency is present, as for normal weight concrete [1]. From the limited number of results at present available this relationship is however not clearly defined and it would be prudent to keep the L/D ratio as close to 2.0 as possible. The correlation between crushing strength of lightweight cores of this ratio and cube compressive strength agrees closely with that anticipated for comparable normal weight concrete cores.

L/D Ratio	Core L/D Correction Factor				
	Lightweight Test Results		Bungey [1]	ASTM C42-82 [4]	BS 1881 pt 120 [5]
	Wet	Dry			
2.0	1.00	1.00	1.00	1.00	1.00
1.6	0.97	0.98	0.91	0.97	0.94
1.4	0.94	0.96	0.86	0.95	0.90
1.0	0.86	0.90	0.77	0.87	0.80

Table 3 Comparison of core correction factors

3.3 Ultrasonic Pulse Velocities

It is well known that correlation between pulse velocity and compressive strength will be influenced considerably by factors such as mix proportions, aggregate type and curing regime. A relationship may however be developed for a particular concrete of specific proportions under defined conditions of age, moisture and curing. It can be noted from Table 1 that pulse velocities are significantly lower than expected with normal weight concrete of comparable strengths. Table 1 also shows that the influence of curing is less significant at early ages, possibly due to the large reservoir of water absorbed in the aggregate. It is thus considered inappropriate to use a strength/pulse velocity relationship developed during early stages for longer term strength assessment since the drying out effects may be misleading. Nevertheless, insitu pulse velocity measurements may provide valuable information concerning concrete uniformity within structural members.

3.4 Pull-out, Internal fracture and Pull-off tests

Fig. 1 shows that, although of the same general form, the relationship between pullout strength and compressive strength for lightweight concrete is significantly different to that for normal weight concrete. To permit inspection of the failure mechanism some truncated cones of concrete were completely extracted following testing, and visual examination of the failure surface showed that this mostly passed through the relatively weak aggregate particles. Behaviour of the overall system is thus more homogeneous than normal weight concrete with strong aggregates and may explain the lower variability of testing. The reduced pull-out force achieved at a given strength level may also be explained by the differences in failure mechanism, with no aggregate interlock occurring [6]. It is clear from Fig. 2 and Fig. 3 that the failure force for both internal fracture loading methods applied to lightweight concrete is also reduced. This feature, coupled with the much

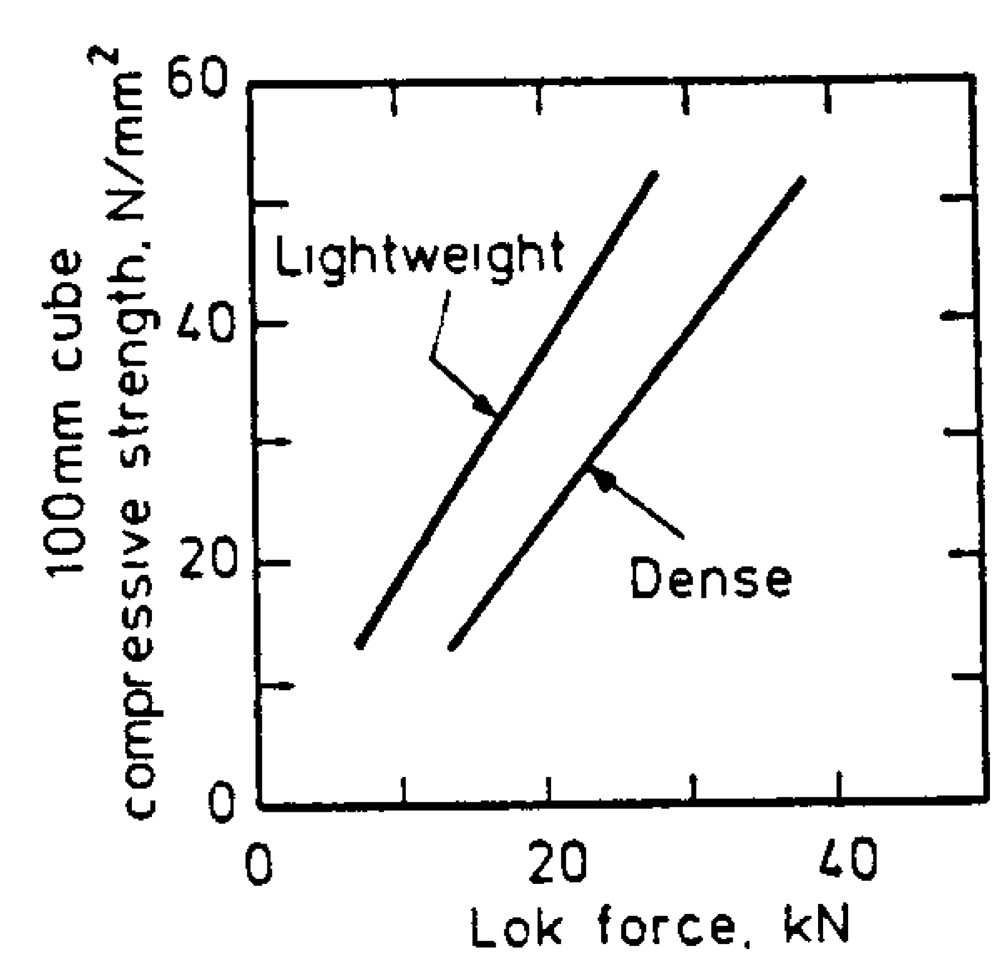

Fig. 1 Correlations between compressive strength and pull-out force

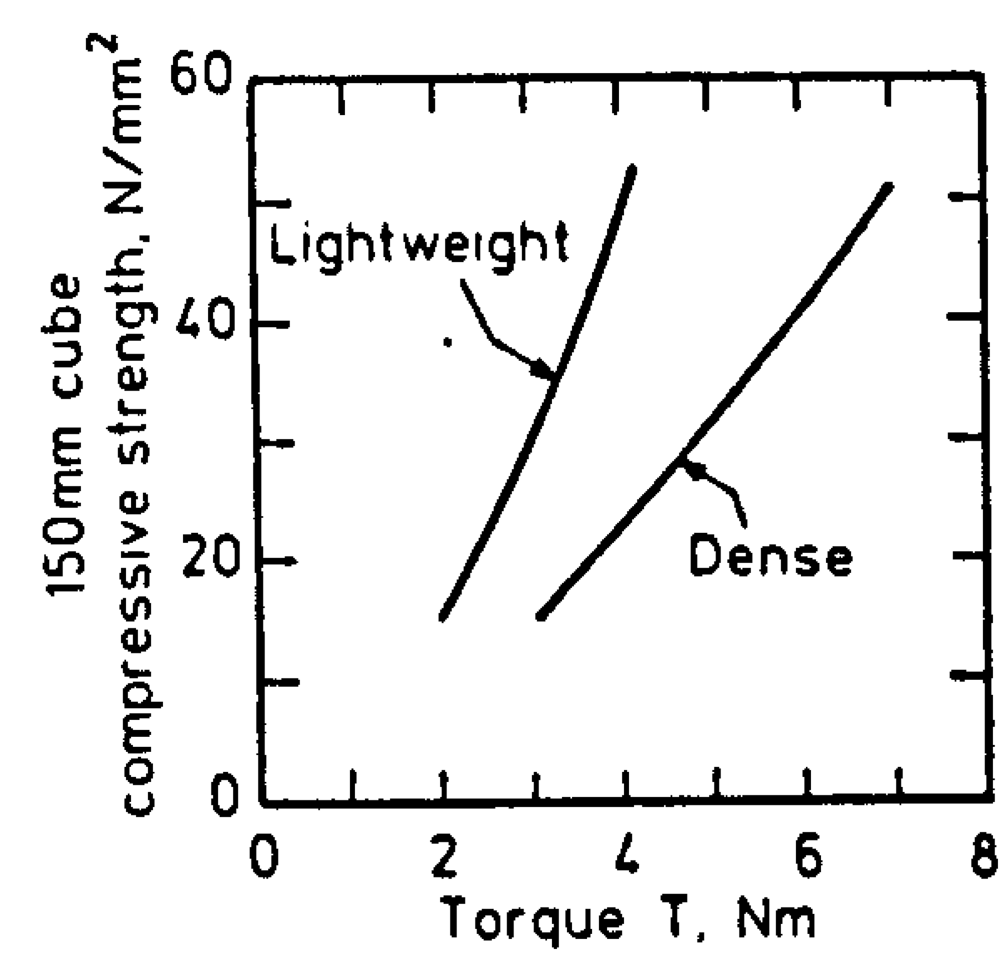

Fig. 2 Correlations between compressive strength and B.R.E. Internal fracture torque

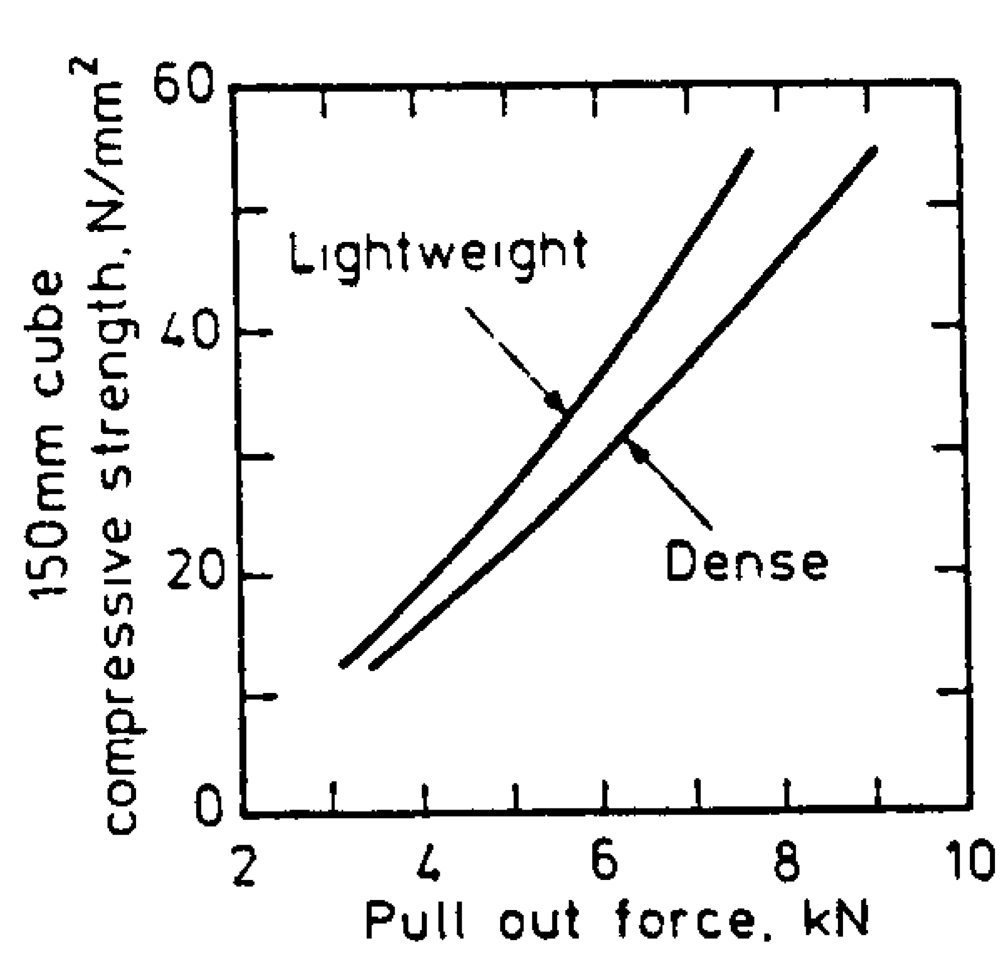

Fig. 3 Correlations between compressive strength and direct-pull internal fracture force

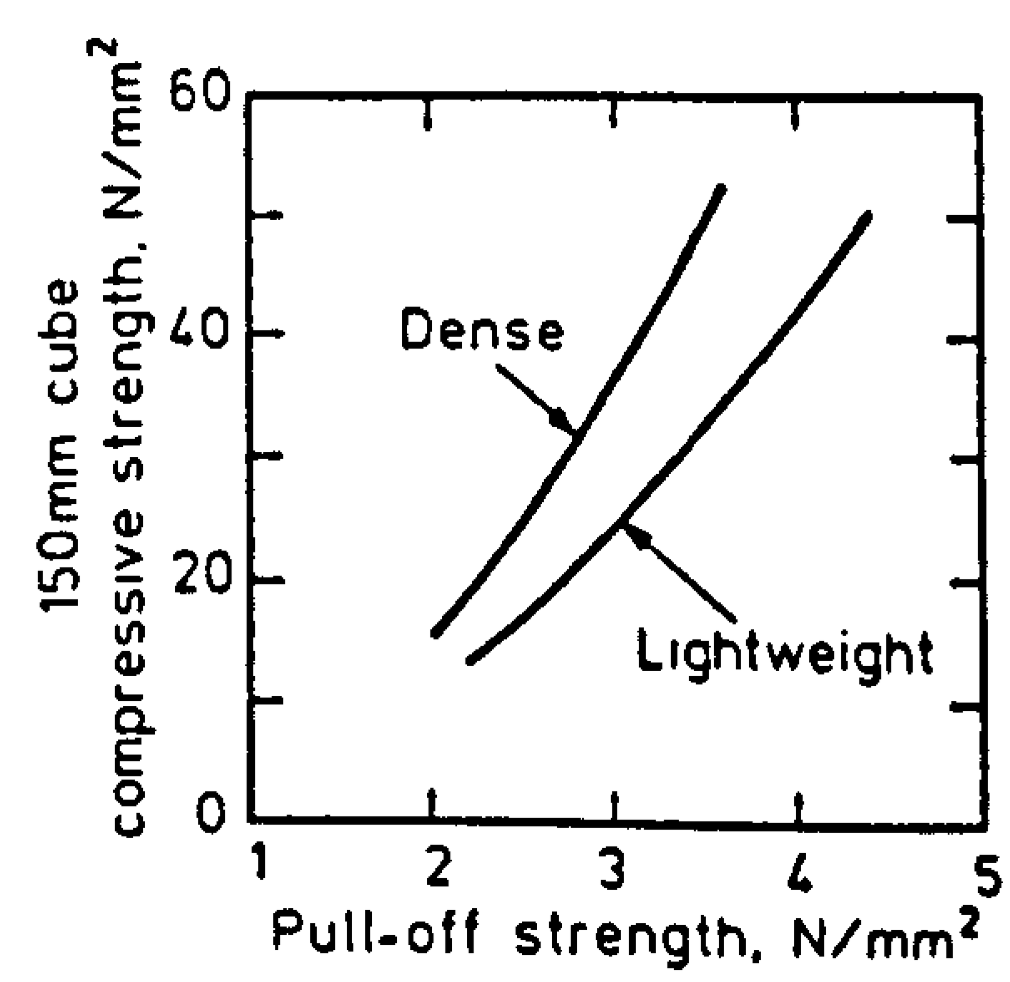

Fig. 4 Correlations between compressive strength and pull-off strength

reduced test variability, is likely to be for similar reasons. It can be noted from table 3 that the accuracy of strength estimation is improved significantly by use of the direct pull method, as also found with normal weight concrete [1].

For the pull-off tests a higher force was achieved at a given compressive strength level (Fig. 4). The reason for this is unclear at present but it is suspected that greater surface porosity may permit deeper adhesive penetration below the concrete surface, and hence increased pull-off strength. Possible differences in relationships between tensile and compressive strength may also be a contributory factor.

4. CONCLUSIONS

From the data presented in this paper it can be seen that all insitu tests, with the exception of cores, showed dependency upon the type of concrete under investigation. All also demonstrated lower testing variability for fully lightweight concrete than for that made with natural dense aggregates, possibly as a result of improved homogeneity due to the absence of strong aggregate particles. Correction factors for core length/diameter ratio were also found to be considerably reduced.

Good correlation was found to exist between compressive strength and results of each test, and accuracies of strength estimation by core, pull-out and direct-pull internal fracture methods were marginally better than assumed for normal weight concrete. Practical usage will however depend upon the aesthetic acceptability of surface damage and consequent repairs, as well as the availability of relevant correlations for the materials used.

It is recommended that ultrasonic pulse velocity measurements be confined to comparative situations, whilst any of the partially-destructive tests may be used as an alternative to cores although providing strength estimates of lower accuracy.

5. ACKNOWLEDGEMENTS

The authors express their thanks to Boral Lytag Ltd. for their generous supply of lightweight aggregates used in this research.

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PARTIALLY DESTRUCTIVE TESTING OF LIGHTWEIGHT CONCRETE

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This paper presents results of a study to assess the reliability of pull out, direct pull internal fracture, and pull off methods when evaluating the insitu strength of lightweight concrete. Three different types of lightweight concrete have been examined incorporating a range of aggregate types. The influence of the physical characteristics of lightweight aggregates upon test performance and strength measurement have been considered.

It is confirmed that strength correlations differ from those for concrete with natural dense aggregates, but it is demonstrated that the pull out and pull off methods may be used successfully on a range of lightweight concrete types with accuracies of strength estimation comparable to, or better than, those possible with dense aggregates.

INTRODUCTION

Interest in determination of insitu concrete quality and strength has increased steadily in many countries for over 30 years. In recognition of the need for insitu testing, a great deal of research has been carried out to assess the reliability of a range of techniques including partially destructive methods on concrete made from natural dense aggregate.

As a result of the large consumption of aggregates by the construction industry, the extraction of raw materials from the ground has become a matter of major planning concern. In view of anticipated increasing shortages of supply of suitable materials in the not too distant future increased emphasis is likely to be placed on manufactured lightweight aggregates. Consumption of lightweight concrete has already grown in recent years, and it is believed that the use of partially destructive test methods will become increasingly important when assessing the insitu strength and quality for the purposes of maintenance, inspection and repair.

In the general field of insitu testing very little attention has so far been paid to lightweight concrete. This paper presents some results of a laboratory experimental program to study the reliability of three different partially-destructive tests (pull out, direct pull internal fracture and pull off methods) applied to lightweight concretes based on two types of aggregate. These are Lytag (ref. 1), made from sintered pulverised fuel ash, and Leca (ref. 2) made from expanded clay.

SCOPE OF THE PRESENT INVESTIGATION

Materials

Two categories of concrete identified as fully-lightweight (F.L.) and semi-lightweight (S.L.) were used for the entire study. Fine and coarse (12 mm) particle sizes of Lytag were used for the fully-lightweight concrete, whereas, either coarse Lytag or Leca together with North-Notts crushed sand were used to provide two alternative semi-lightweight materials. Fully-lightweight concrete made with Leca was not used due to its very poor strength qualities, as reported by the manufacturer, which limit its value for structural concrete. The physical properties of aggregates are given in Table 1, and 'Castle' Ordinary Portland Cement was used throughout the tests.

Type of Aggregate	Apparent Specific Gravity	24 Hour Water Absorption, (%)
Coarse Lytag	2.01	13
Coarse Leca	0.80	32
Fine Lytag	2.35	15
North Notts. Sand	2.67	0.78

Table 1. Physical Properties of Aggregates

Four different mixes were designed for each type of concrete and the mix details are given in Table 2. A pan type batch mixer was used for the mixing and the consistency of the fresh concrete measured by the compacting factor test.

Testing

Laboratory test specimens were made and subjected to a number of partially destructive tests. Results for pull out, internal fracture and pull off tests are presented in this paper. The tests were performed following wet and dry curing regimes at different ages up to 28 days. The methods used are described briefly below, whilst the number of tests in each case refers to those being performed at any one age. Compressive strengths related to pull out tests were obtained from groups of three companion cubes cured under identical conditions and tested at the same ages. For the other methods the actual tested cubes were

Type of Concrete	28 Day Wet Cube Strength (N/mm²)	28 Day Wet Density (kg/m³)	Material per cubic metre: Cement (kg)	Water (kg)	Fine Aggregate (kg)	Coarse Aggregate (kg or m³)	Compacting Factor
Fully-lightweight concrete (Lytag)	23	1835	200	300	750	554 kg	0.93
	32	1848	226	287	633	628 kg	0.91
	35	1864	259	287	601	642 kg	0.92
	42	1876	329	289	526	657 kg	0.93
Semi-lightweight concrete (Lytag)	19	2007	201	258	790	717 kg	0.95
	30	2018	240	257	753	715 kg	0.95
	39	2023	301	258	695	717 kg	0.94
	48	2037	345	260	658	722 kg	0.96
Semi-lightweight concrete (Leca)	14	1568	230	210	840	0.75 m³	0.88
	19	1627	320	204	770	0.75 m³	0.94
	23	1595	380	204	720	0.75 m³	0.95
	25	1669	508	230	662	0.74 m³	0.95

Table 2. Details of concrete mixes for different types of lightweight concrete

crushed, and predetermined corrections applied to obtain compressive strength values.

1. Pull Out Test (ref. 3) Six pull out inserts, of a type commercially available (Lok-Test), with 25 mm head diameter were installed at a depth of 25 mm in the faces of a 225 mm cube before concreting. These inserts were loaded using commercially available Lok test model L12.3 equipment (Fig. 1) to obtain the failure force.

2. Direct Pull - Internal Fracture Test (ref. 4) Six tests were performed on a pair of 150 mm cubes with three tests applied to each cube such that two were located on side faces and one on the bottom face. The test involved drilling a 6 mm diameter hole followed by fixing an expanding wedge anchor bolt into the hole and measuring the load required to pull this out. A modified form of the established B.R.E loading technique was used based on a direct pull force (Fig. 2). This has the advantage of avoiding the twisting action which may be present with the BRE loading method and will reduce the scatter of results obtained.

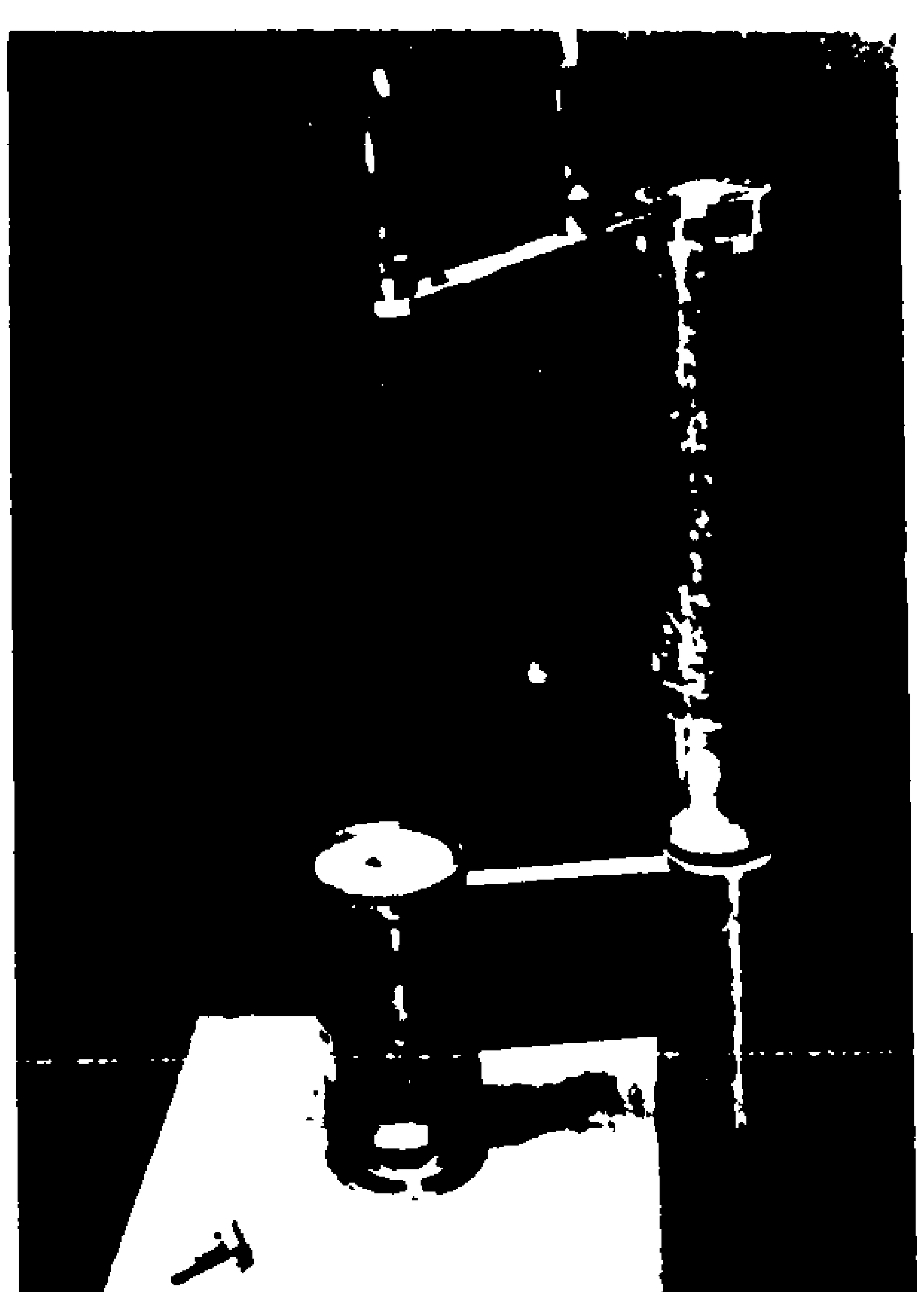

FIG. 1

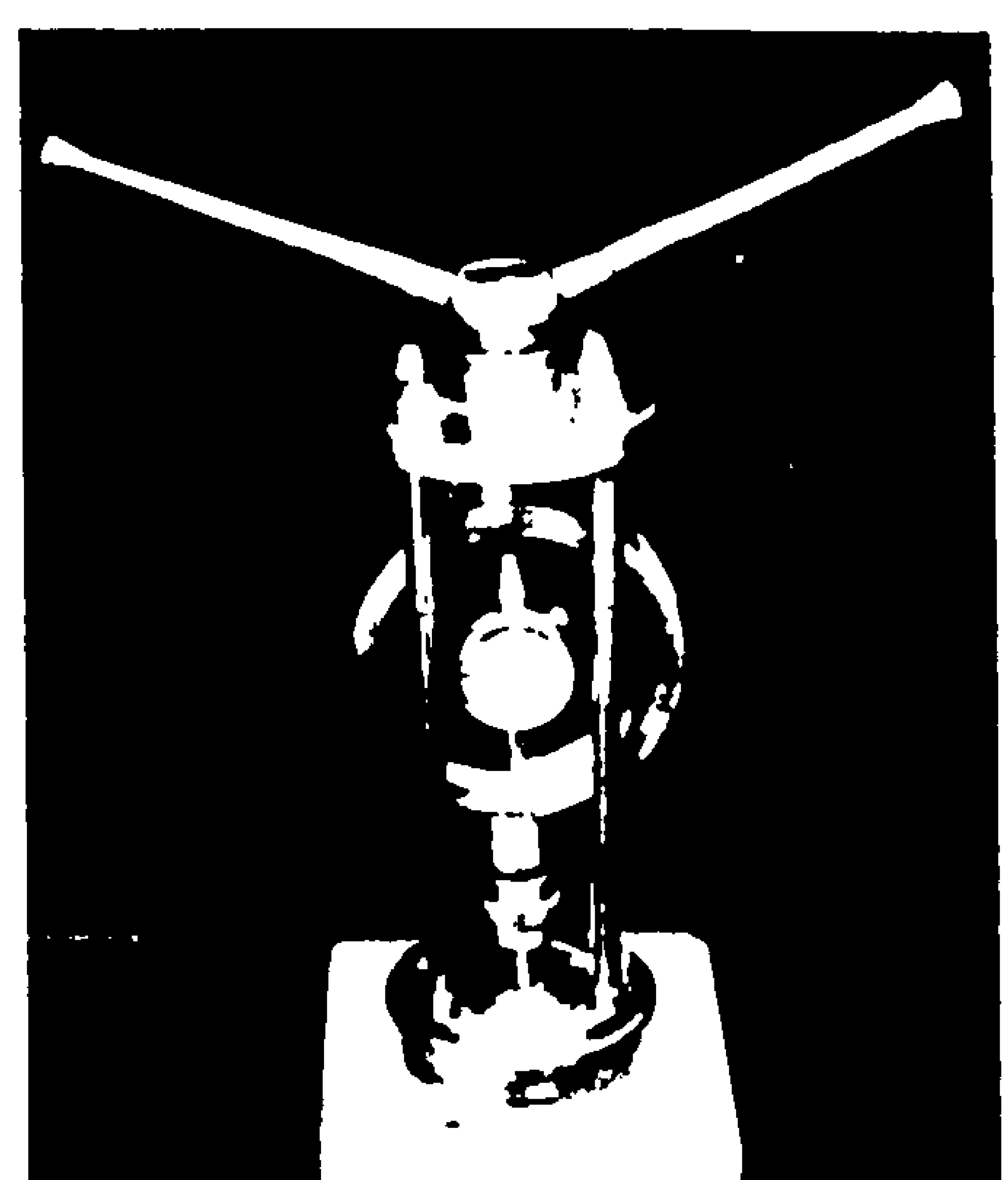

FIG. 2

3. Pull Off (ref. 5) Six pull off tests were performed on a group of three 150 mm cubes. Two tests were carried out on each cube where side faces were considered for testing. The tests were performed by bonding a 50 mm diameter circular aluminium disk to the surface of the concrete under test by means of epoxy resin adhesive. A tensile force was then applied to the disk using commercially available Limpet apparatus (Fig. 3) to cause failure. The pull off strength was calculated on the basis of the pull off force acting over the nominal contact surface area of the disk.

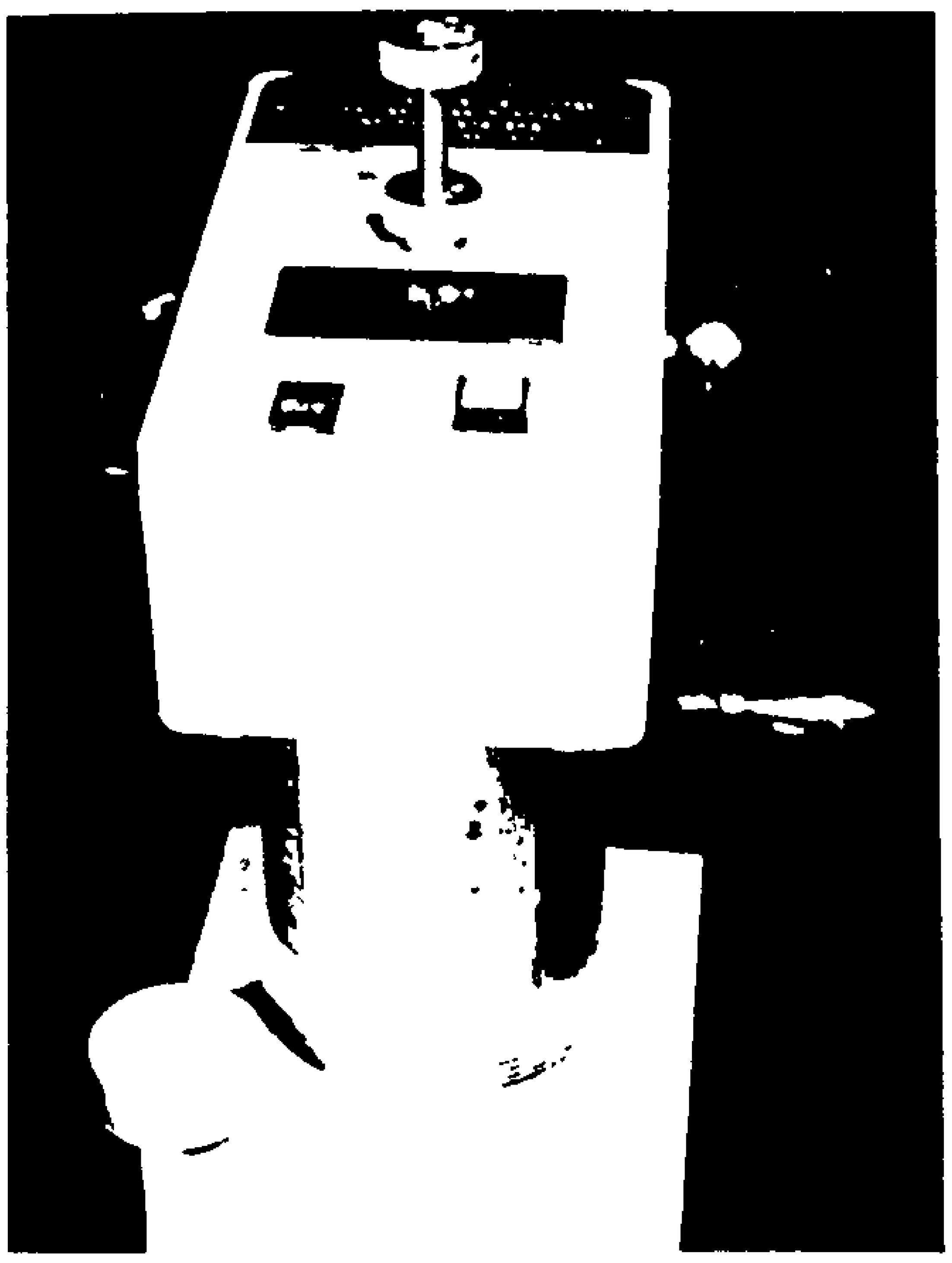

FIG 3

PRESENTATION OF TEST RESULTS

The correlations between the partially destructive test results and the cube compressive strengths are shown in Figs. 4 to 6. Statistical variation analyses based on coefficients of variation together with correlation coefficient and 95% confidence limits have been summarized in Table 3.

DISCUSSION OF TEST RESULTS

The Effects of Aggregate Characteristics on Concrete Testing

Concrete is usually considered to be a two-phase composite comprising the aggregate, which is an included phase, distributed in the mortar matrix phase. The strength of concrete is determined by the properties of the matrix and the aggregate and their interaction. It primarily depends on the strength, modulus of elasticity and Poisson's ratio of both of the components (ref. 6). With normal weight concrete, the aggregates have an essentially higher density and higher modulus of elasticity than the mortar. Consequently, the short-term strength of a normal weight concrete

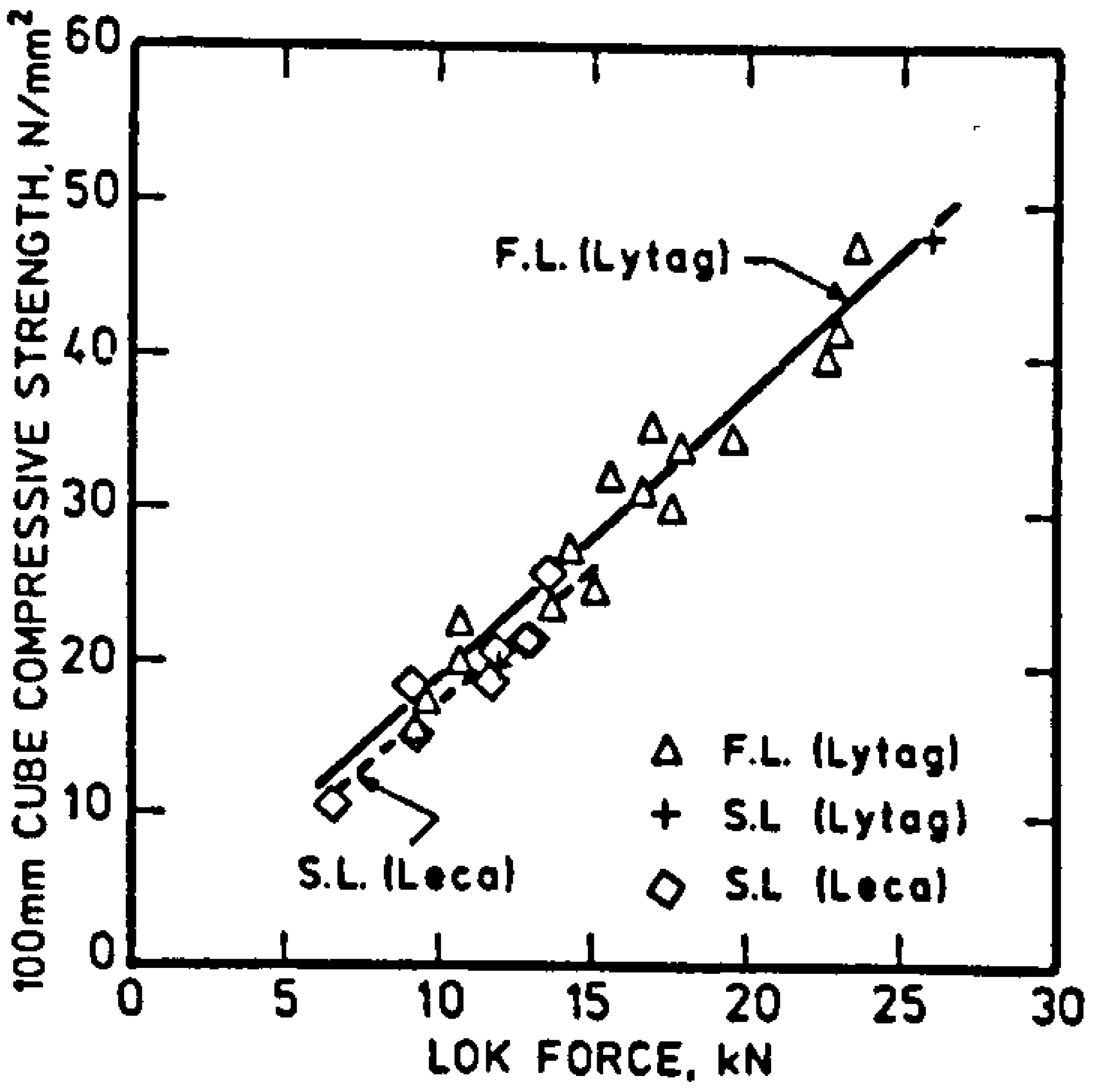

FIG. 4

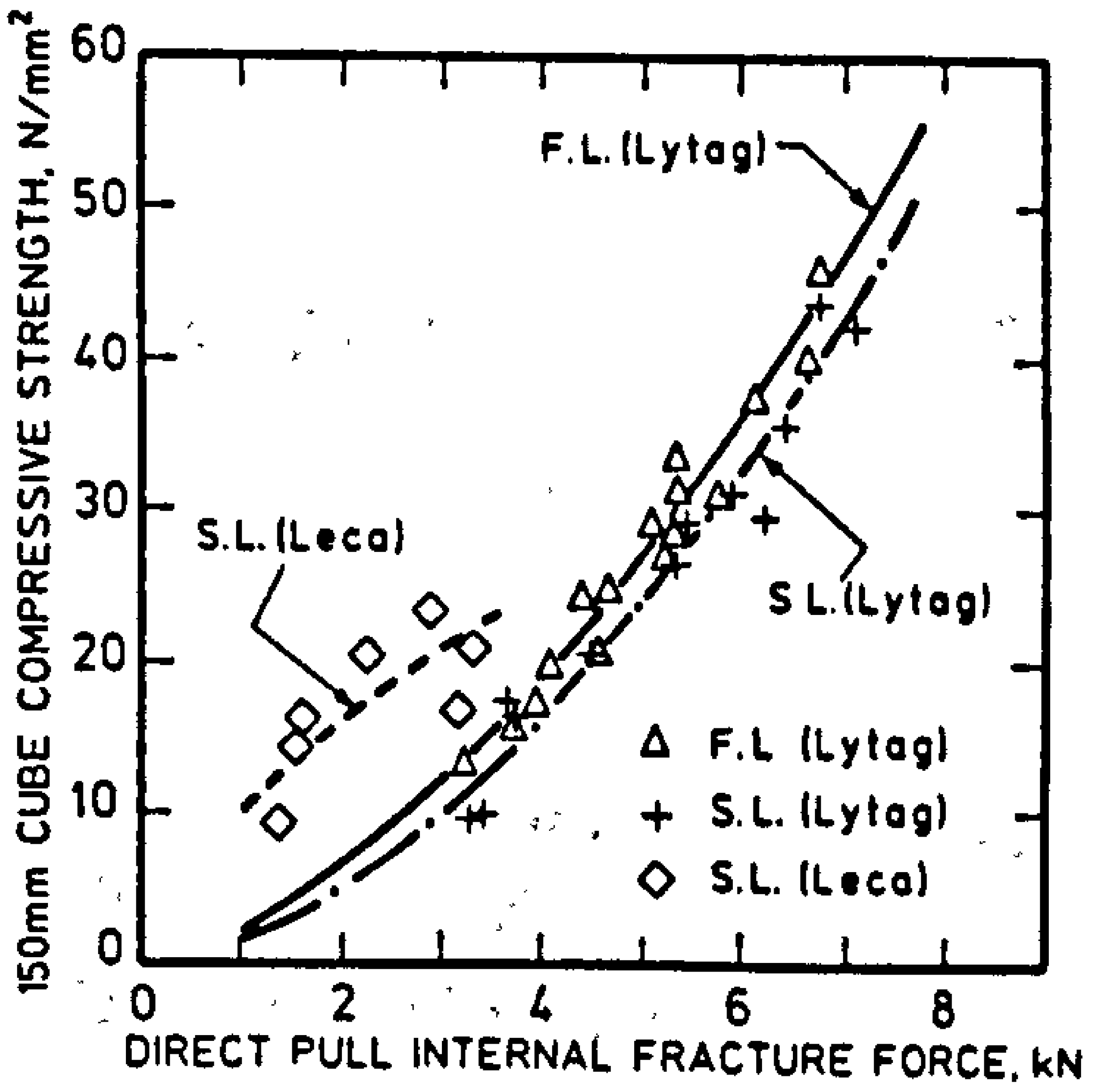

FIG. 5

may be considered principally as a function of the strength of the mortar only. Lightweight aggregates are likely to have a lower strength and a lower modulus of elasticity than the matrix, thus their influence on the strength of lightweight concrete must additionally be taken into account.

In this experimental investigation, Leca was found to be very light and weak compared to Lytag and this has been shown to have a significant effect on test results including the mode of failure. For concrete made from Lytag, it was found that at low strengths aggregate-matrix bond failure predominated whilst at high strengths aggregate fractures were predominant. This characteristic

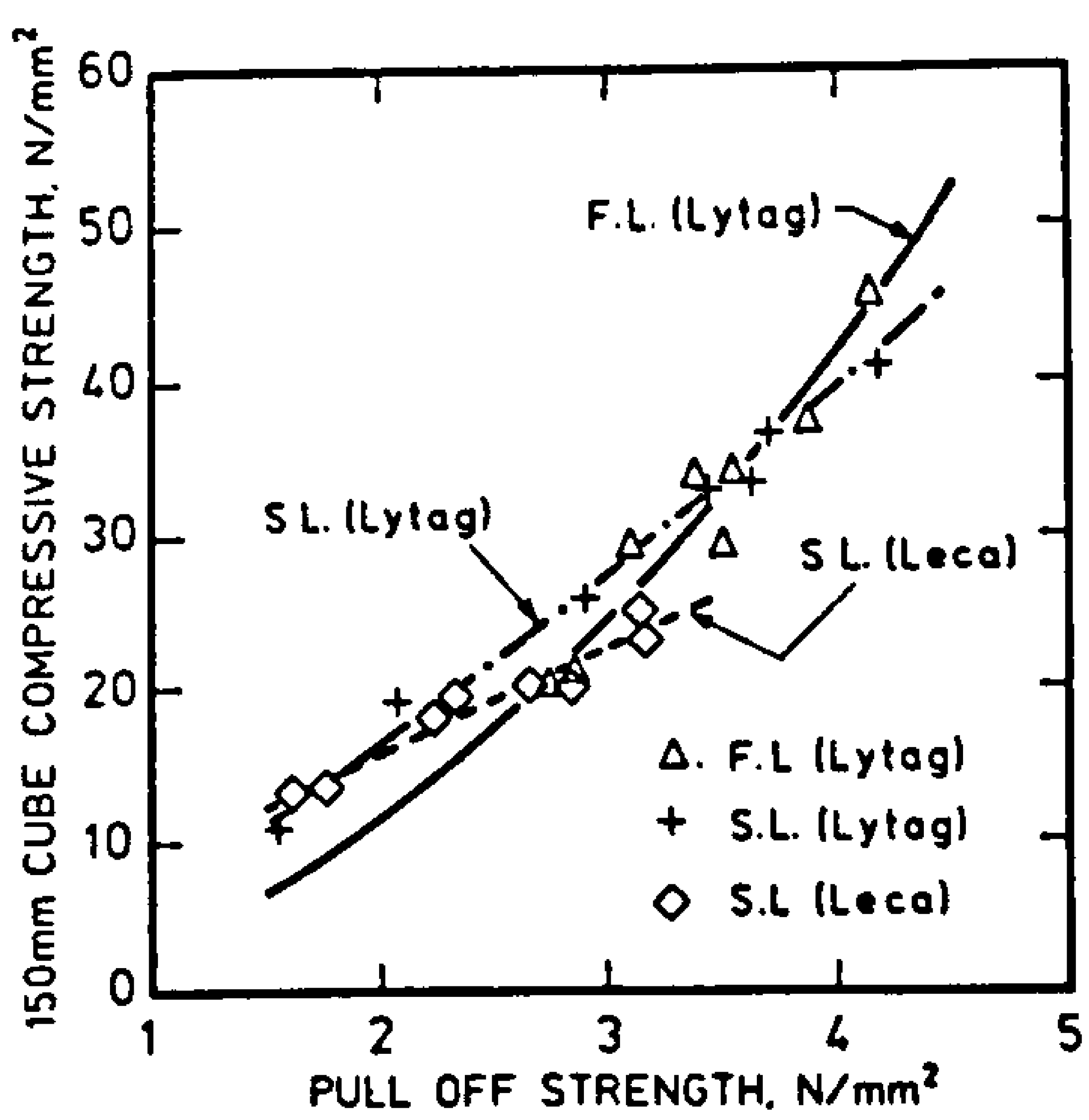

FIG. 6

was not the same for concrete made from Leca where for all cases the failure passed through the aggregates.

Due to the low density of Leca, it was also found that there is a tendency for it to float and migrate to the top of the specimen during vibration which results in a layer of mortar at the bottom of the sample (see Fig. 7). It appears that the thickness of this layer, is dependent on the size of the unit. This problem has not been found with concrete made from Lytag for which more or less uniform concrete was obtained.

As a result of these differing characteristics of lightweight aggregates and their distribution within concrete, the present authors are currently carrying out an extensive experimental programme to study the strength variations within full sized concrete beams made with different types of lightweight aggregates.

<u>Within-test Variability</u>

The within-test variability referred to here basically relates to the single laboratory/operator/material precision of the test methods. Table 3 shows the within-test variability based on coefficients of variation for each of the test methods on the different types of concrete. The results show that the lowest variabilities were obtained for fully lightweight concrete with Lytag with all values below those reported for normal weight concrete (refs. 7,8). With semi-lightweight concrete using Lytag, the data show that the variabilities of pull out and pull off methods are increased and are of the same order as obtained for normal weight concrete but internal fracture results are of reduced variability.

The within-test variability of all test methods related to Leca, as shown in Table 3, is high in comparison with Lytag. This is linked to the nature of the test methods, which are surface tests, and the non-uniformity of aggregate distribution. Tremendous differences in test results were also found for Leca between side and bottom faces of the test specimens, due to different characteristics of these regions. It is thus considered that test results relating to bottom faces may be misleading and they have not been included in the statistical analyses for concrete made with Leca.

The highest within-test coefficient of variation for concrete made with Leca was for the direct pull internal fracture test. This is related to the mechanism of test as well as the aggregate characteristics. Recalling the test procedure, the first step involves drilling, leaving a hole with diameter equal to that of the drill. However, when the soft aggregate was encountered there was a tendency for the hole to become oversize at that point. Furthermore, when tensile load is applied to the anchor bolt this is mainly transferred from the expanded portion of the anchor bolt to the material in which it is embedded. After maximum load is reached, the anchorage typically fails by rupturing the concrete and the formation of a 'pull-out' concrete cone. However this behaviour was not obtained in most cases when testing on side faces of Leca specimens. This is because when the expanded portion came into contact with Leca the wedging forces exceeded the strength of the Leca, resulting in local crushing. This provided increased room for expansion of the clip and resulted in the pulling out of the centre bolt part of the fixing through the outer clip with little or no fracturing of the concrete.

FIG. 7

<u>Relationship Between Partially Destructive Measurements and Cube Strength</u>

Figures 4 to 6 give the correlations between the parameters determined by the partially destructive tests and cube strength for the different types of concrete made from Lytag and Leca. The data points on these figures represent results at different ages and include both wet and dry curing conditions. The relationships between splitting tensile strength and compressive strength at an age of 28 days are given in Fig. 8, indicating the effects of the curing conditions upon these.

The effect of curing regimes on the relationship between partially destructive tests and compressive strength was studied throughout the tests on fully lightweight concrete and in some

Concrete Type	Test Method	Coefficient of Variation (%)	Correlation Coefficient	95% Confidence Limit on Estimated Strength (%)
Fully-lightweight concrete (Lytag)	Pull Out	5.6	0.968	±17
	Direct Pull Internal Fracture	9.8	0.987	±16
	Pull Off	5.7	0.986	±24
Semi-lightweight concrete (Lytag)	Pull Out	7.5+	-	-
	Direct Pull Internal Fracture	8.3	0.970	±35
	Pull Off	8.6	0.982	±19
Semi-lightweight concrete (Leca)	Pull Out	15.1x	0.936	±23
	Direct Pull Internal Fracture	34.0	0.795	±77
	Pull Off	23.8	0.976	±15

+ Based on averages of groups of 6
x Based on averages of groups of 5

Table 3. Statistical Evaluation for partially destructive tests on different types of lightweight concrete

instances on semi-lightweight concretes with Lytag. Except for the pull off test which requires completely dry conditions for effective adhesive performance, on the remaining test methods the relationships at age 7 and 28 days were found to have only a small dependency on the

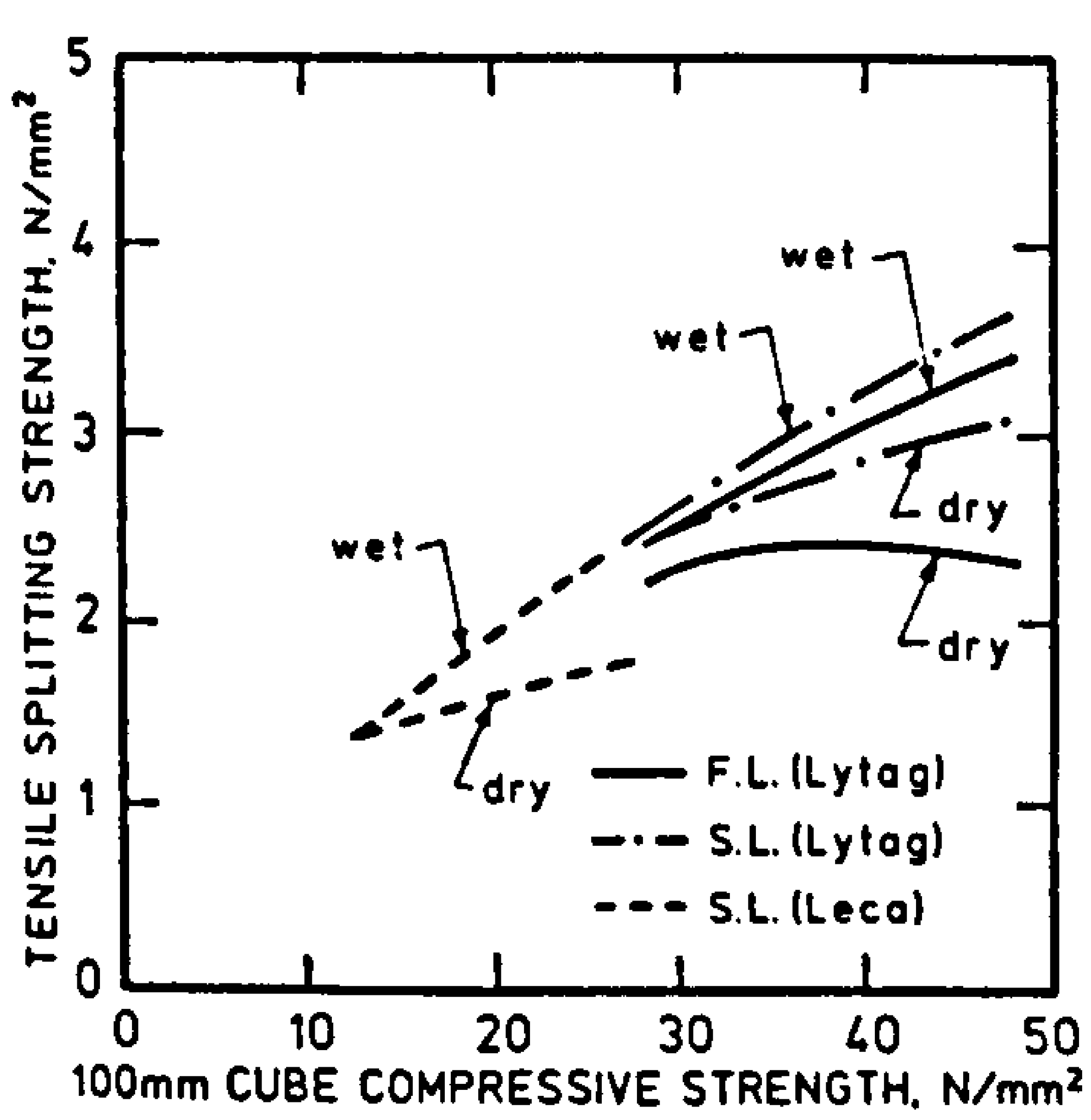

FIG. 8

curing regime. This dependency was found to be greatest at 28 days due to differential moisture distributions, but much less significant than the effect upon the tensile/compressive strength ratio. This may be explained by the different nature of test methods. For partially destructive tests such as pull-out and direct pull internal fracture, the localized failure in the concrete occurs under a complex stress whereas in the cylinder splitting test, the concrete along the vertical diamter is under nearly uniform tensile stress acting over about the middle two-thirds of the specimen. Failure will be in splitting tension due to weakness of concrete in tension. Further analysis was also undertaken to study the effect of curing on the relationship between partially destructive measurements and compressive strength by combining the test results under different conditions. It has been found that a relationship with fairly high correlation coefficient could be expected to result. Hence it was decided to use a wet-cure only on the remainder of the test program and a single relationship was considered for practical purposes.

The correlations between partially destructive tests and compressive strength are generally similar in form in all cases, except for the internal fracture test using Leca. The correlation coefficients range from 0.936 to 0.987, which are comparatively higher than those reported for normal weight concrete by some investigators (refs. 3,4,5). The poor correlation for the internal fracture test measurement versus the compressive strength with Leca is because in most cases the maximum applied load has not reached the failure criteria of concrete as discussed above When using other test methods on Leca however, the tests were performed successfully, although the scatter of results was high due to non-uniformity of the concrete. The accuracies of strength estimation based on 95% confidence limits for strength levels of 30 and 20 N/mm^2 are shown in Table 3 for Lytag and Leca concretes respectively. From this, it seems that the pull out and pull off tests provide the most promising approach for assessing the insitu equivalent cube strength. Pull out results for semi-lightweight Lytag concrete were limited, and are not included in Table 3, but were found to agree closely with the correlations obtained for the fully-lightweight material.

Figures 4 to 6 also graphically compare the calibrations for fully and semi-lightweight concretes. In Fig. 4, for the pull out test, marginally higher pulling resistance was detected for semi-lightweight concrete. This is somehow related to the failure mechanism of the pullout test which seems to be very complex, as reported by Krenchel and Shah (ref. 9). However one possible reason for this might be linked to sand replacement which improves the physical behaviour of the concrete (ref. 10). Similar behaviour has been shown in Fig. 5 for the direct pull internal fracture test, apart from the results for Leca.

Comparing the relationships for pull off tests with different types of concrete (Fig. 6) indicates that the pull off strength for a given compressive strength varies from one type of concrete to another. With Lytag, the semi-lightweight concrete provides lower pull off strengths than fully-lightweight concrete up to a compressive strength of 37 N/mm^2. This may be due to the high porosity of fine Lytag resulting in a greater surface porosity which may permit deeper adhesive penetration below the surface of the fully-lightweight concrete (ref. 7). This behaviour is reversed at higher strengths when it appears that the tensile strength of concrete as shown in Fig. 8 then dominates the measurements of pull off strength yielding a higher tensile strength for semi-lightweight concrete. Comparison of the concretes made with Lytag and Leca indicates that a higher pull off strength will be achieved for Leca. This behaviour, apart from the adhesive penetration effect, might be influenced by non-uniformity of the concrete. Increased overbreaking was also detected during these experiments with Leca, and since the pull off strength is calculated on the basis of the surface area of the disk this will result in an apparently increased pull off strength. Apart from differences in behaviour between concrete made with different types of lightweight aggregates, the test results also showed that in all cases pull off strengths were higher than those reported for concrete made with natural aggregates (ref. 7).

CONCLUSIONS

It has been demonstrated that partially destructive test methods may be successfully applied to the strength determination of concrete made with lightweight aggregates. In most cases, high correlation coefficients were found to exist between the compressive strength and the parameter determined by the partially destructive tests considered. As a result of different characteristics of lightweight aggregates, correlation with strength was found to vary according to the type of aggregate and whether natural or lightweight fines are used. It is thus essential that a correlation is prepared for the material involved in any specific investigation. The effect on correlations of curing conditions ranging from wet to dry was however found to be small and of no practical significance despite having a major effect upon the tensile/compressive strength relationships.

Particular difficulties were encountered with the direct pull internal fracture test applied to concrete made with Leca, and this method cannot be recommended for this type of lightweight concrete. The pull out and pull off methods however were found to perform satisfactorily for all the types of lightweight concrete examined.

The lowest variability of results was found for fully lightweight concrete made with Lytag which exhibited lower variability than reported for normal weight concrete. By contrast, semi-lightweight concrete made with Leca showed the highest variability on the test results due to non-uniform distribution of aggregates. Tremendous differences in the test results on semi-lightweight concrete made with Leca were detected between side and bottom faces of the laboratory cube specimens. Hence, the bottom faces of structural elements would not be recommended for assessing insitu strength of this type of lightweight concrete.

Among the three test methods examined, it appears that both the pull out and pull off approaches can provide a reliable estimate of insitu equivalent cube strength of generally comparable accuracy to that expected for dense concretes. It must be noted that dry conditions are required for the pull off test in the form used, and there is some evidence that when lightweight aggregates are present the extent of adhesive penetration may be greater and more variable than with concrete made with natural aggregates.

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