A COMPARATIVE STUDY ON REINFORCED CONCRETE LIGHTWEIGHT ROOF DECKS

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A B S T R A C T

Nine reinforced lightweight one-way slabs were produced with perlite, styropor and pumice lightweight aggregates as structural roof decks with insulating properties. Those plus three identical Ytong aerated slabs were tested as simple beams under short-term loads until failure. For comparison purposes unit weights (800 kg/m^3) dimensions and reinforcement of the produced slabs were tried to be kept the same as the Ytong slabs. Also, three cylindrical samples were taken from each slab for uni-axial compression tests.

The deflections and mid-span strains were measured for the slabs during the flexural tests. Also, compression and deformation tests were done on the cylinders.

According to the test results, all the slabs showed favorable conditions from the points of load-carrying capacities, deflections, initial crack loads and crack widths with respect to the accepted standards. The slabs made with pumice lightweight aggregate had the highest carrying capacity with the lowest cost of production, therefore being the most economical.

Ö Z E T

Perlit, stiropor ve ponza taşı kullanılarak, her malzemeden üçer adet olmak üzere, tek doğrultuda çalışan ve ısı yalıtım özelliklerine haiz dokuz adet taşıyıcı hafif çatı plağı üretilmiştir. Bunlara, üç adet özdeş Ytong marka gazbeton plağıyla birlikte, kısa süreli yükleme altında basit kiriş şeklinde yükleme deneyleri uygulanmıştır. Karşılaştırma açısından birim ağırlıklar (800 kg/m^3), boyutlar ve donatı Ytong plaklarıyla aynı tutulmaya çalışılmıştır. Ayrıca üretilen her plak için üçer adet silindir numune alınmış ve bunlar üzerinde tek eksenli yükleme deneyleri yapılmıştır.

Eğilme deneyleri esnasında plak ortasındaki birim şekil değiştirmeler ve sehimler ölçülmüş, silindirlerde de basınç çekme (perlit numunelerinde) ve deformasyon deneyleri yapılmıştır.

Bütün plaklar taşıma gücü, sehim, ilk çatlak yükü ve çatlak genişlikleri açısından, gözönüne alınan standartlara göre uygun sonuçlar vermiştir. Bunlar arasında ponza taşıyla üretilen plaklar, diğerlerinden daha ucuza mal olup en yüksek taşıma gücüne sahip olduklarından dolayı, en ekonomik olarak bulunmuştur.

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a	Depth of the rectangular stress-block for the reinforced concrete section
∂	Volume of air in 1 m^3 fresh concrete
a_v	Shear span length
A_s	Main (bottom) steel (Tension reinforcement)
A'_s	Constructive (top) steel (Compression reinforcement)
b	Breadth of the slabs (47 cm)
c	Depth of the neutral axis from the top face of the section
C	Cement content in 1 m^3 fresh concrete (350 kg/m^3)
d	Effective depth of the reinforced concrete section
d'	Distance between the top face and the compression reinforcement
D_b	Compressive force due to flexure in Ytong aerated slabs
E_c	Modulus of elasticity for concrete
E_s	Modulus of elasticity for steel
f'_c, $f_{\ell c}$	Cylinder strength of lightweight concrete
$f'_{c_{cube}}$	Cube strength of lightweight concrete
f_{flex}	Flexural strength of the plain concrete beams
f_{mo}	Compressive strength of mortar
f_y	Yield (limit) of steel
ℓ	Effective span length for the slabs (184,5 cm)
M_u	Ultimate moment for the test slabs
N_{su}	Maximum tie force of the end anchorages of the slabs
γ_i	% of the "i" aggregate in the total volume of aggregates

Pe(1,2,3)	Perlite slabs
St(1,2,3)	Styropor slabs
Pu(1,2,3)	Pumice slabs
Yt(1,2,3)	Ytong slabs
V_A	Total volume of the aggregates in the mix
V_u	Ultimate shear force for the test slabs
$\omega = W/C$	Effective water/cement ratio by weight
W	Water content in 1 m^3 fresh concrete
z	The internal lever arm of the reinforced section
δ	Deflection of the slab under flexural loads
δ_{ai}	Unit weight for the "i" aggregate
δ_c	Specific gravity of cement
Δ	Density of the lightweight concrete
Δ_a	Density of the lightweight aggregate
Δ_{fresh}	Fresh unit weight of the concrete
Δ_{56}	Air-dry unit weight of the concrete at the end of 56 days
$\Delta_{oven\ dry}$	Oven-dry density of the concrete
Δ_{teo}	Theoretical density for the concrete produced
ε_{cu}	Maximum allowable compressive strain for lightweight concrete
ε_c	Compressive strain due to flexure at the top fibre of the reinforced concrete section
ε_s	Strain of steel
ε_y	Yield strain of steel
λ	Thermal conductivity value
ρ_b	Balanced reinforcement ratio
$\sigma_x^m(y)$	Stress formed in the mortar in x-direction and y-axis
$\sigma_y^m(x)$	Stress formed in the mortar in y-direction and x-axis
ν	Poisson's ratio of concrete

ρ = unit weight of the concrete

c' = specific heat capacity

S_{24} = Heat storage property in 24 hours.

s = Depth of the specimen

D = Thermal inertia property.

Q = Quantity of water which flows through the concrete sample

A = Cross section area of the sample

K = Capillarity constant, showing how fast the water rises through the sample

t = Time interval for the water to rise in the sample

W_t = Weight of the sample subject to capillary absorption after a time interval "t"

W_o = Oven-dry weight of the sample

W_a = Percent of absorbed water

CHAPTER I
INTRODUCTION

1.1. GENERAL

Ordinary concrete is a widely used material in structural engineering. Although it has a good load-bearing capacity, its unit weight and therefore thermal conductivity are high. Reduction in the density of concrete is advantageous from economical aspects. The sound-absorbing capacity gets better and thermal insulation is higher than ordinary concrete. Lightweight concrete is produced mainly in three ways(1).

1- Using porous natural or artificial aggregates in place of normal aggregates (Lightweight aggregate concrete).

2- Introducing large quantities of voids in the concrete by physical or chemical means (Aerated-cellular-foam or gas concrete).

3- Omitting the fine aggregate (No-fines concrete).

The advantages and disadvantages that lightweight concrete offers in general are as follows:

Advantages:

1- Because of the reduction in density, formwork need stand a lower pressure for the same volume of the material. Total weight of the materials to be handled is reduced with a consequent increase in productivity and transport.

2- Dead loads are reduced for the structure which in turn reduces the dimensions of the foundations and other structural elements. This is especially important where there is poor ground conditions.

3- Thermal insulation is good.

4- Fire-proofing qualities are better than for normal concrete.

5- Nailing is easier than ordinary concrete. This brings practicality where large numbers of fixings (e.g. in hospitals) have to be made to the concrete.

6- Lightweight concrete has greater suitability over dense concrete for cutting.

7- Less reinforcement is needed since the dead loads and thus the design moments are reduced, especially when lateral earthquake forces are considered, where the forces exerted on the building are directly proportional to the dead weight of the structure.

8- Savings can be made in the application of plaster and other finishes (especially in interior use).

Disadvantages:

1- Because of their porosity, strength is lower than for normal concrete.

2- Insulation against moisture is necessary.

3- Resistance to abrasion is poor.

4- Thicker floor slabs are necessary where heavy live loads have to be carried (e.g. warehouses etc.).

5- An increased cover for reinforcement in externally exposed structures is needed.

6- They have lower permissible shear values than dense concrete.

7- Because of the low modulus of elasticity, deflections are larger, and creep is higher to some extent. Long term behaviour under sustained load is mainly related to the rigidity of the aggregate. Low rigidity increases the creep and also the initial deformation.

1.2. LIGHTWEIGHT AGGREGATE PRACTICE IN TURKEY

Artificial lightweight aggregates like expanded clay, expanded slate and such are not yet being produced in Turkey. Before an artificial lightweight aggregate industry is developed, the utilization of Turkey's abundant reserves of natural lightweight aggregates can be recommended. Natural lightweight aggregates have lower strengths and endurances compared with artificial lightweight aggregates. In spite of this fact, it is possible to produce structural and insulating concretes with natural lightweight aggregates. Pumice, volcanic tuff and volcanic cinders are the most abundant natural lightweight aggregates in Turkey: Especially pumice has great reserves in East and Middle Anatolia in Kayseri-Develi, Niğde, Nevşehir, Van and Bitlis regions, these being approximately 15 billion cubic meters(2). It can be said that natural lightweight aggregates being in great amounts in the cold regions of Turkey, is a good coincidence. Thus, extra energy is not needed to produce artificial lightweight aggregates in these regions.

Perlite also has great reserves in Turkey. The main deposits are around Kars, Erzurum, Van, Erzincan, Nevşehir, Ankara, Manisa and Izmir(2). A big plant for the expansion of perlite ore is in production in Cumaovası, Izmir, since 1982. Perlite deposits are around 8 billion tons in Turkey. Only 8000-10000 tons is worked up annually. This is one millionth of the total deposit. When expanded, this amount is around 90000 cubic meters. Half of this is used in the building sector, and the other half is being utilised in agriculture and industry.

Granular styropor is not used for the building industry in Turkey. It is produced mainly for the packing industry, with the raw material coming from abroad and being expanded here.

1.3. OBJECT AND SCOPE

The object of this study was to experimentally investigate the possibility of making reinforced lightweight structural roof decks with insulating properties using three different lightweight aggregate concretes. For comparison purposes, unit weights, dimensions and reinforcement of these one-way slabs were tried to be kept identical with one type of Ytong aerated slabs.

In the following chapters, first a literature survey about lightweight aggregates and lightweight concretes is given. Then, results of the flexural and uniaxial compression tests are presented with comparisons between these four types of materials and finally some conclusions are drawn from the results of these tests.

CHAPTER 2

LITERATURE SURVEY AND THEORETICAL CONSIDERATIONS

2.1. CLASSIFICATION OF LIGHTWEIGHT AGGREGATES AND CONCRETES

Lightweight concretes are classified according to both density and strength factors. There are different classifications for different countries especially from the point of density.

According to ASTM, concretes which have densities less than 1840 kg/m^3 are classified as lightweight concrete(3). In Turkey and several other countries, the upper limit for the density is 1900 kg/m^3(4). Generally, the practical range is between 300-1800 kg/m^3(1).

According to DIN 1045(5):

Lightweight concrete: $\Delta < 2000$ kg/m^3 (300-2000)
Normal concrete : $2000 \leq \Delta < 2800$ kg/m^3
Heavyweight concrete: $\Delta \geq 2800$ kg/m^3

Another classification is as follows(5):

Lightweight concrete: $\Delta = 400\text{-}1800$ kg/m^3
Lightweight normal concrete : $\Delta = 1800\text{-}2000$ kg/m^3
Normal concrete : $\Delta = 2000\text{-}2500$ kg/m^3

It is also possible to classify lightweight aggregates according to their bulk unit weights.

Bulk Unit Weight of the Lightweight aggregate (kg/m^3)	Type of Concrete Produced
$\Delta_a < 400$	Insulating conrete
$400 \leqq \Delta_a < 650$	Insulating and structural concrete
$\Delta_a \geqq 650$	Structural concrete

The most common way in the production of structural lightweight concrete is to keep the density in the desired range by adding lightweight aggregates. These aggregates can be classified as follows(8):

a) Natural lightweight aggregates: Pumice, tuff, volcanic cinders, scoria, diatomite (all of volcanic origin except diatomite),

b) Artificial aggregates produced from natural materials: Expanded clay, shale, slate, expanded perlite, exfoliated vermiculite,

c) Lightweight aggregates produced from industrial residues: Blast furnace slag, cinders, pulverized fuel ash (fly ash),

d) Lightweight aggregates artificially produced from industrial residues: Expanded blast furnace slag, heated fly ash;

e) Organic lightweight aggregates: Cereal grains, wood chips etc.

Also polymer-based materials, such as styropor, can be added to this list.

2.2. GENERAL ASPECTS ABOUT LIGHTWEIGHT AGGREGATE CONCRETE

Compressive strengths of lighweight aggregate concrete lies between 0,3-40 N/mm^2 (3-400 kg/cm^2) for the range of densities between 300-1850 kg/m^3 compressive strengths up to 60 N/mm^2 (600 kg/cm^2) can be obtained using high cement contents (e.g. 560 kg/m^3). 20 N/mm^2 concrete may require 240-400 kg/m^3 cement while 30 N/mm^2 concrete 300-500 kg/m^3 cement. With lightweight aggregate, cement content varies from the same to 2/3 more than with natural aggregate for the same strength of concrete. There is a limit of cement content in lightweight concrete, which is known as strength ceiling, above which an increase in cement content does little to increase the strength of concrete(1).

The maximum compressive strength which can be reached by an ordinary concrete is primarily limited by the strength of mortar, which depends on age. With lightweight aggregate concrete, the maximum compressive strength is additionally limited by the aggregate. Fig.1 shows the development of strength of lightweight aggregate concrete with four different cement contents buth with the same water content and an aggregate of expanded shale. Between 28 and 56 days all mixes reach a maximum value of compressive strength, recognizable by the horizontal lines. In this case, the lightweight concrete has reached its limit compressive strength(9).

The properties of lighweight aggregate concrete, like those of normal concrete, are affected by the (1) type of aggregate, (2) grading, (3) cement content, (4) W/C ratio and (5) degree of compaction. Main points to watch are (1) work-ability of concrete, (2) its drying shrinkage and moisture movement, (3) strength and (4) thermal conductivity, the last two closely related to density.

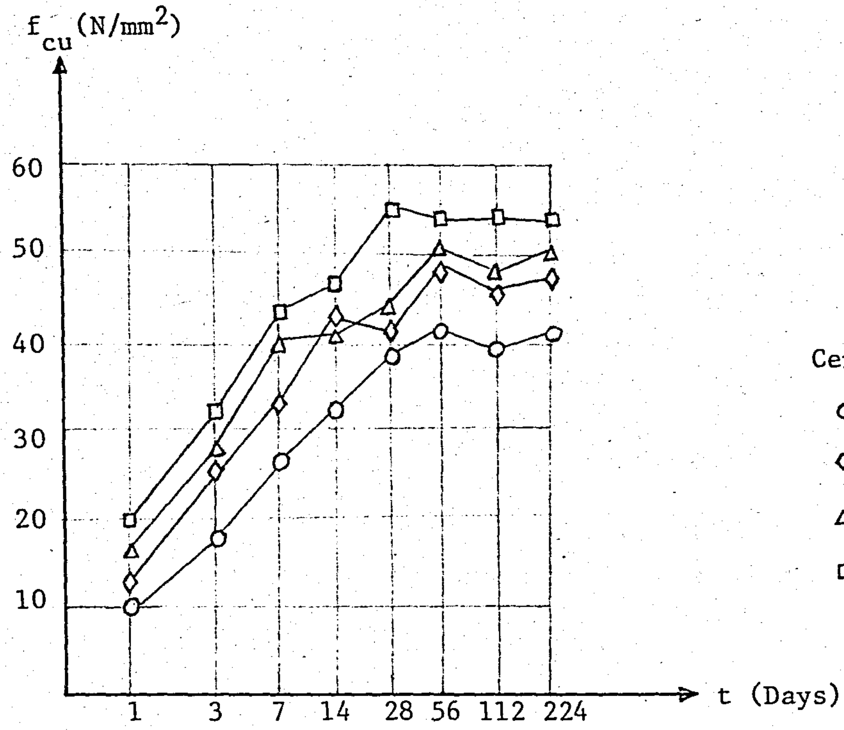


Figure 1- Strength development of a lightweight aggregate concrete with expanded shale aggregate, and four cement contents but constant water content before and after attainment of the limit compressive strength (approximately horizontal line)

Lightweight aggregates are usually angular shaped and have a rough surface. Therefore, they are more suited to plant mixing. Workability can be increased by adding fine aggregate of ordinary weight to the mixture, but this increases the density of the concrete with the insulation properties of the concrete adversely affected. Also, however, with increased workability, water and therefore cement content can be reduced with an increase in concrete strength.

Replacement of lightweight fine aggregate by sand is usually made on an equal volume basis (partial or total replacement). In the latter case, a reduction in water content of 12-24 % (compared with an all lightweight aggregate mix) has been reported. Concrete with total sand replacement has an 10-30 % higher modulus of elasticity than all lightweight aggregate concrete. Also its shrinkage is reduced by 15-35 %.

Air-entrainment can also improve workability. By introducing air in the mixture, water content is reduced which in turn reduces the bleeding and segregation of the concrete. Limits for the total air contents which do not cause reduction in the strength are, for 19 mm max. aggregate size, 4-8 %; and for 9 mm maximum aggregate size, 5-9 %. Air content in excess of these values lowers the compressive strength by about 10 kgf/cm^2 (1 N/mm^2) for each 1 %, according to Neville(1).

When lightweight aggregate is used in reinforced concrete, special care for corrosion must be taken. The depth of cover can be twice as high as with normal concrete. Generally a larger cover to reinforcement is desirable. Alternatively, the use of rendered finish or coating of reinforcement with rich mortar or cement-latex compound or covering the reinforcement with corrosion protective materials such as bitumen is found helpful.

For same mix composition the shrinkage of lightweight concrete is about 5-40 % higher than ordinary concrete. However, as lightweight concrete has a low modulus of elasticity and a high "tensile strength/compressive strength" ratio, the probability of crack formation during shrinkage is lesser. Nevertheless, contraction joints should be provided against the danger of such cracks(10).

Creep, taken on the basis of the stress/strength ratio is of the same order as for ordinary concrete. Neville(1) suggests that long-term creep is higher than ordinary concret But this fact is not confirmed.

Poisson's ratio is less than that for ordinary concrete. Modulus of elasticity (E_c) is $\frac{1}{2} - \frac{3}{4}$ of that of ordinary concrete of the same strength.

Lightweight aggregate concrete is less sensitive to moist curing. Otherwise, rate of gain of strength is similar to that of ordinary concrete.

The abrasion resistance of lightweight aggregate concrete is not very good. But frost resistance, except when aggregate was saturated before mixing, is excellent.

Sound absorption of lightweight concrete is good, because air-borne sound energy is converted into heat energy in the channels of concrete. Absorption coefficient of sound is twice that for ordinary concrete. But lightweight concrete is not a good sound-insulator, since insulation is directly proportional to the density of the concrete.

Lightweight concrete has a low thermal expansion. This can produce problems if ordinary concrete and lightweight concrete are used side by side. Special care is needed in this situation.

While preparing the mix, a simple way of determining the necessary water is as follows: Grip a handful of concrete tightly, then throw it away and observe the pattern of the grout on the palm. If the hand is well spotted, the mix is correct. If there're only few specks, it is too dry. Hand covered with grout means the mix is too wet, consequently it has a higher density and lower thermal insulating properties.

If at the time of mixing, aggregates are dry, they will absorb the water and reduce the workability. Therefore, if aggregates have a high rate of absorption but low initial moisture content about $\frac{1}{2}$ of the mixing water should be mixed with aggregate.

If the aggregate is saturated before mixing, strength gets 5-10 % lower for the same cement content and workability. When dry aggregate is used, mixing water is absorbed prior to setting by the aggregate, this water having contributed to the workability at the time of placing (like vacuum processed concrete). On the other hand, the density of concrete made with a saturated aggregate is higher, and the durability of such concrete, especially its resistance to frost, is impaired. But when aggregate with high absorption is used, it is difficult to obtain a sufficiently workable and yet cohesive mix, and generally aggregates with absorption of over 10 % are pre-soaked.

2.3. PROPERTIES OF THE LIGHTWEIGHT AGGREGATES USED IN TURKEY

Perlite is a glassy volcanic rock of acidic character. When heated up rapidly to the point of incipient fusion (900-1100^{o}C), it expands 10-20 times its original size owing to the evolution of steam and forms a cellular material with a bulk density as low as 30-240 kg/m^3. Concrete made with perlite has a very low strength, a very high shrinkage and is used primarily for insulation purposes(1).

Pumice is a light-colored, froth-like volcanic glass with a bulk density of about 500-900 kg/m^3. Those varieties of pumice not too weak structurally make a satisfactory concrete with a density of 700-1400 kg/m^3 with good insulating characteristics, but having high absorption and shrinkage(1).

Styropor is the common name of a polymer-based material whose chemical name is polystyrol. The expanded particles are in perfectly round granular shape not exceeding 8 mm in diameter. It is possible to accept this material as ideal holes when used as an aggregate because of its negligible weight compared to the mortar. One of the most important advantages of this material is that its water absorption is practically zero. Therefore, pre-soaking is not necessary and the water/cement ratio is not affected when styropor aggregate is replaced by a natural aggregate. On the other hand, its very poor resistance against fire is a disadvantage of this material for structural purposes(11).

2.4. PROPERTIES OF CELLULAR(Aerated) CONCRETES

Cellular (or foam or aerated) concrete is a concrete weighting from 160 to 1600 kg/m^3 and having a homogeneous void or cell structure(12). This type of concrete is usually made of portland cement, water and foaming agent, and may contain lime, silica, fly ash, expanded shale, volcanic ash or pumice dust. A large amount of air, usually exceeding 25 % in the form of small bubbles, is incorporated to reduce weight(13).

Valore proposed to divide cellular concretes into two major groups:

a) Moist cured cellular concretes,
b) Autoclaved cellular concretes.

These concretes weighting as little as 160 to 320 kg/m^3 may be used for thermal insulation. Densities of load-bearing cellular concretes may range from 550 to 1600 kg/m^3. "Fill" concrete form a third category: Insulation is combined with modest compressive strength in roof and floor fills.

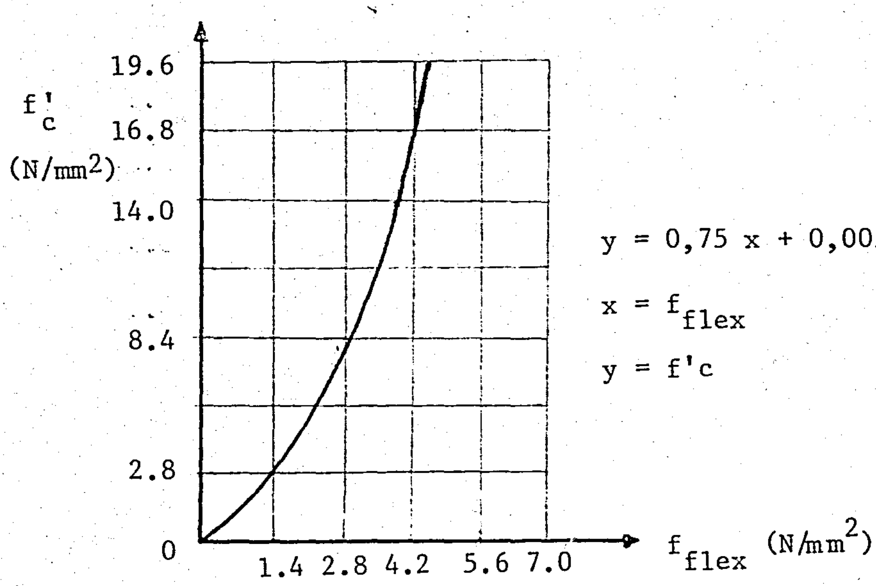
Compressive strengths of cellular concretes are functions of density and ranges from 1,7-7 N/mm^2 for 480 kg/m^3, 2,8-14 N/mm^2 for 640 kg/m^3, 5,6-21 N/mm^2 for 800 kg/m^3.

Average values for a static modulus of elasticity of Swedish cement-silica cellular concrete are reported as in table 1(14).

Table 1- Relationship between density, comp. strength and mod.of elasticity of Swedish cement-silica cellular concretes

Density (kg/m^3)	f'_c (N/mm^2)	E_c (N/mm^2)
520	2,5	1360
640	5,0	2270
740	5,7	2630
800	6,9	3160

Flexural strength is approximately $\frac{1}{5}$ to $\frac{1}{3}$ of the compressive strength. The relation between flexural and compressive strengths obtained by Graf for autoclaved cellular concretes of various compositions and cell-forming processes is shown in Fig.1(14).



$y = 0,75\ x + 0,005\ x^2$

$x = f_{flex}$

$y = f'c$

Figure 2- Relationship of flexural to compressive strength for autoclaved cellular concretes made by various processes

Ytong is the trademark of a widely used cellular concrete based on a Swedish patent. For the production of this material, quartzite is finely grinded, and lime, sand and cement are added to it. After they are thoroughly mixed, water and a special aluminium powder is introduced into the mix. The reaction between the aluminium and the cement releases H_2 and bubbles are formed in the mix, causing it to expand to about three times its initial volume. This material is then cut into pieces and autoclave cured to obtain high strength.

The densities of Ytong is for G 50, 650 kg/m^3 and for G 25, 500 kg/m^3. The least average compressive strengths of cubic samples must be for G 50, 5 N/mm^2 (50 kg/cm^2) and for G 25, 2,5 N/mm^2 (25 kg/cm^2). These values stay within the limits given by TS 453(34).

Its thermal resistance is about 10 times of ordinary concrete. Fire resistance is up to $1200^{\circ}C$(15).

2.5. SOME STUDIES RELATED TO LIGHTWEIGHT AGGREGATES

2.5.1. <u>Effect of Aggregate Strength on the Strength of Lightweight Aggregate Concrete</u>

Grübl(9) showed that concrete strength will be higher with the increase in the tensile strength of lightweight aggregate.

But as stated in ACI 213(6), the strengths of lightweight aggregate grains show big differences according to their origins and types, and there is no qualitative measure to determine the aggregate strength, which means that no clear correlation exists between the aggregate grain strength and lightweight concrete strength.

2.5.2. Effect of the Shape and Surface Form of the Aggregates on Concrete Strength

Budnikov and his friends(16) have stated that aggregates with smooth surfaces are more resistive than the ones having rough surfaces. They found out that the most suitable grains are the ones which have a shape closest to a cube or a sphere; and the weight of the aggregates which have a "shape factor"* greater than 2,5 should not exceed 15-20 % of the total aggregate weight.

Shape of the aggregate has an important effect on the compressive strength of the lightweight concrete, the long and slender aggregates reducing the strength. Also, cornered aggregates with irregular shapes and rough surfaces reduce the workability of lightweight concrete(1).

2.5.3. Effect of Other Characteristics of Aggregates

ACI 211(17) states that by pre-soaking the lightweight aggregate before the production of lightweight concrete, it is possible to reduce the crumbling of the aggregate in the transportation, prevent the segregation, and control and stop the consistency changes in the concrete.

The absorption of water by the aggregate in fresh concrete is not only related to the characteristics of the aggregate, but also to the matrix volume and the W/C ratio of the mix. According to the investigations done by Müller-Rochholz(18), the immediate absorption by the dry aggregate

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* The shape factor as discussed here is defined as "the ratio of the longest to the shortest dimension of the aggregate grain".

in the cement paste is nearly the same as the absorption in 30 minutes when W/C ratio is relatively high (0,60), that this ratio does not change with time when the aggregate is pre-soaked, but there is an increase in the water content of the cement paste during 60 minutes when the water/cement ratio is 0,40. These values are effective W/C ratios, where effective water is the mixing, water for cement and the pre-soaking water used for the lightweight aggregates before the production of concrete is not included.

a) The Effect of Geometric Heterogenity on Concrete Strength

Some researchers have investigated the changes in the compressive strength of test samples due to the variations of the dimensions of aggregates and samples, which is designated as geometric heterogenity (Sample size (D)/Aggregate size (d)). Tanigawa and Yamada(19) showed that geometric heterogenity does not effect the compressive strength of lightweight aggregate concretes, while increasing (D/d) the compressive and tensile strengths of ordinary concrete decreases considerably.

b) Effect of the Continuous Phase (Matrix) on Concrete Strength

Concrete can be simply thought as a two-phased composite material consisting of aggregate and fine mortar phases. In ordinary concrete, strength and modulus of elasticity values of the aggregate are greater than the values for the mortar. Therefore the weaker part, namely matrix is the cause of failure in this case. In lightweight concrete, the strength and modulus of elasticity of the mortar is greater than the aggregate's, and concrete fails because of the aggregate in this case. According to Wesche(20), strength of lightweight concrete is always less than the mortar strength. Therefore increasing the strength of the mortar will not be of much help for this type of concrete for which strength will be increased by the aggregate.

Grübl(21) in his investigations, found out that mortar strength increases the compressive strength of lightweight concretes, but this increase not being very much. His explanations are as follows: When an outer force is applied on the lightweight concrete, tensile stresses perpendicular to the direction of the force are formed on the boundaries of the aggregates (Fig.3). $\sigma_x^m(y)$ shows the stress formed in the mortar in x-direction along the y-axis and $\sigma_y^m(x)$, the stress in y-direction along the x-axis. When the tensile force exceeds a certain value these stresses cause tension also in the mortar phase. In addition to this, a tensile force F_u effects the aggregate. As this stress reaches the tensile strength of the aggregate, it cracks, and the sudden growth of the crack into the mortar causes the failure of the concrete. Therefore, the compressive strength of the lightweight concrete becomes related to the tensile strength of the aggregates.

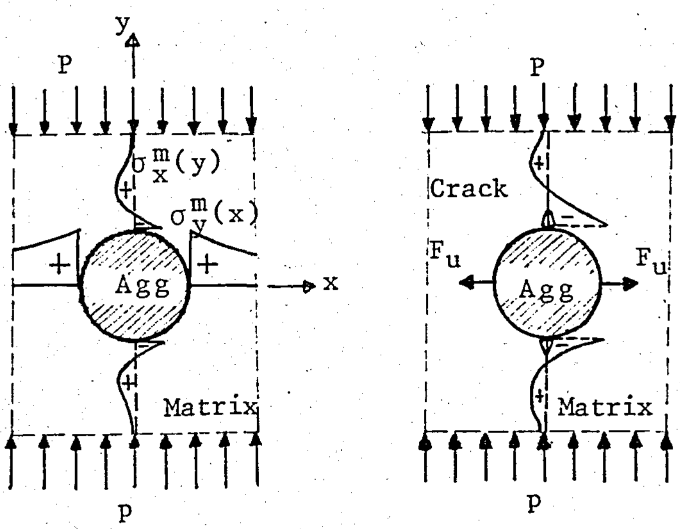

Figure 3- Model for failure of lightweight aggregate concrete

Grübl(9) has stated five types of failure according to the tensile strength of the mortar. If the lightweight aggregate concrete has a strength equal to its mortar strength the failure is from the matrix (Failure type I in Fig.4). If the bond strength between the aggregates and mortar is relatively high, the stress is carried to the aggregate grains. After the bond strength is exceeded, the crack advances along the boundary of the grain. Grübl has called this as combined failure. If the aggregate does not crack until the first crack forms in the matrix, failure types II, III and IV arise. When the matrix tensile strength is higher than this, aggregate will crack before the first crack forms in the matrix. This is the highest compressive strength of the lightweight aggregate concrete (Failure type V).

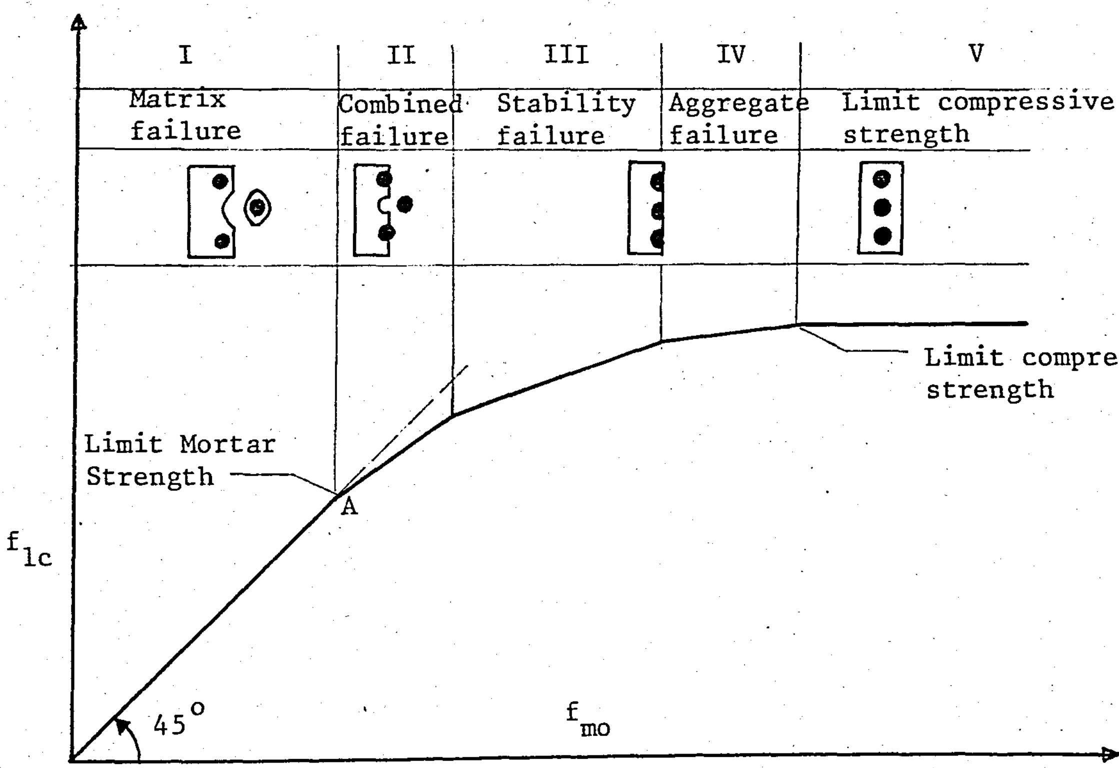

Figure 4- Simplified relation between the compression strength of the mortar, f_{mo}, and the compression strength of lightweight aggregate concrete, $f_{\ell c}$.

2.6. THE STRESS-STRAIN CURVES OF LIGHTWEIGHT AGGREGATE CONCRETES

Popovics(22) has found out that the short-term σ-ε curve of the normal concrete has more curvature than the mortar or the cement paste for the same W/C ratio. The reasons for this are the micro-cracks which form between the aggregate and the mortar in the concrete. According to Popovics, the factors effecting the shape of the σ-ε curve are the type and characteristics of the aggregate, rate of loading and the microcrack development.

The descending part of the σ-ε curve of the lightweight concrete is investigated by Grimer and Hewitt(23) and the following results are found: Because of the different rigidities of the aggregates and mortar, the stress distributions for these two types of materials are not the same. The rate of growth of the microcracks between them is related to the difference between these two rigidity values. As these values approach each other, the σ-ε curve is straighter which is justified with experimental results. Therefore, the curves for the high strength ordinary concretes and lightweight concretes are more like straight lines than curves. According to the experiments done by these researchers, the lightweight concretes of relatively low densities lose their strengths in a very brittle way in uni-axial tests with a sudden fall in the σ-ε curve. For this reason, Grimer and Hewitt concluded that the belief that all concretes are elasto-plastic is not quite true, and a distinction should be made between the types of concretes which follow this rule and which do not.

Wesche(20), who has parallel opinions with the above investigators, has stated that the design of reinforced lightweight concrete should not be made according to the σ-ε curve of ordinary concrete; and for a reinforced lightweight concrete beam, the stress distribution in the compression area can be thought as linear rather than parabolic.

2.7. ULTIMATE STRENGTH DESIGN OF DOUBLY-REINFORCED RECTANGULAR LIGHTWEIGHT CONCRETE SECTIONS FOR FLEXURE

The basic philosophy in the design for flexural capacity is that failure will occur by yielding of the steel rather than by crushing of the concrete. Flexural calculations for lightweight concrete beams are done in a similar way as for the normal concrete reinforced sections, with some variations due to the characteristics of the material, the two most important of these being that the maximum compression strain of the extreme fibre of the beam is to be taken as 0.002 (0.003 for normal concrete) and that the area of the stress block is reduced with respect to normal concrete (Fig.5) (24)(25).

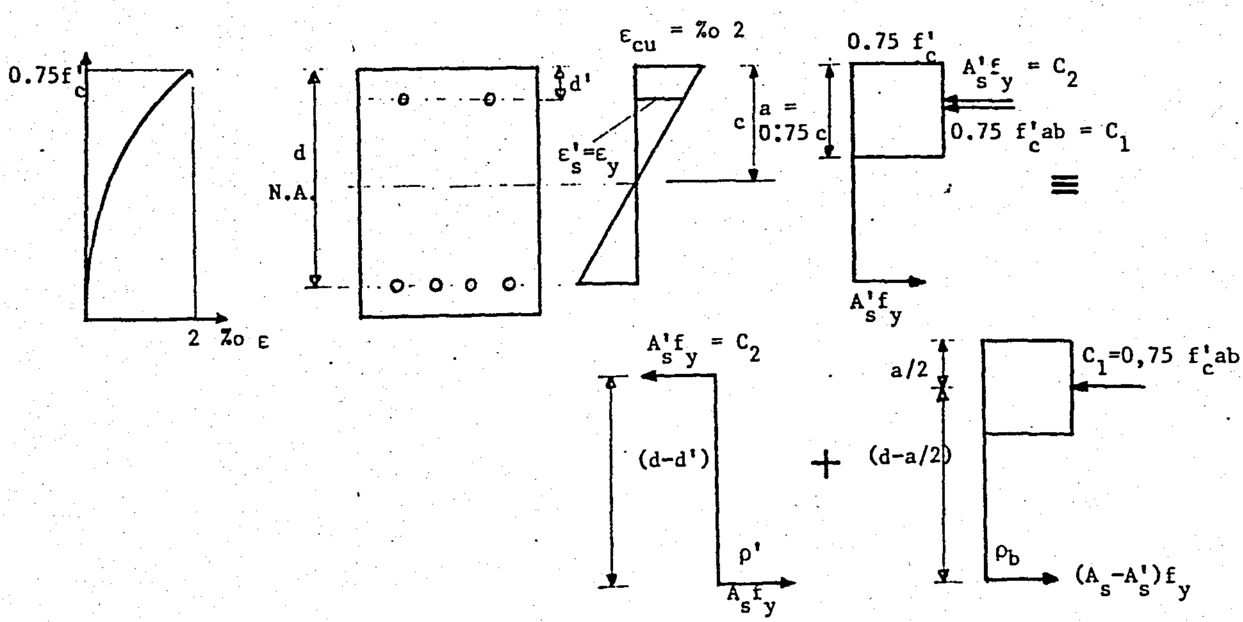

Figure 5- Ultimate strength for doubly-reinforced rectangular beams.

The equations of equilibrium for the analysis of the doubly-reinforced sections can be written from Fig. 5 as follows:

$$0.75\ f'_c\ ab = (A_s - A'_s)\ f_y \tag{1}$$

$$M_u = A'_s f_y (d-d') + (A_s - A'_s)\ f_y (d - \frac{a}{2}) \tag{2}$$

Here, it is assumed that the compression reinforcement yields with the tension reinforcement.

The balanced reinforcement, for which tension and compression failure of the beam take place at the same time, can be written from eq.1 and the compatibility equation given below:

$$\frac{c}{d} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y} \quad , \quad \varepsilon_y = \frac{f_y}{E_s} \tag{3}$$

$$\gamma_b = \frac{A_s - A'_s}{bd} = 0.75 \times 0.75\ \frac{f'_c}{f_y}\ \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y} \tag{4}$$

For Ytong one-way structural roof decks, calculations are done according to the rules given in DIN 4223. The assumptions done for flexure for these slabs are that, plane section remains plane; the maximum strain for the longitudinal reinforcement is %o 2; the maximum flexural compressive strain for this concrete is %o 2; and the stress block can be taken as a rectangle as for normal concretes. In this case, the compression force can be calculated as follows. Here, the compression reinforcement is not included in the calculations(26).

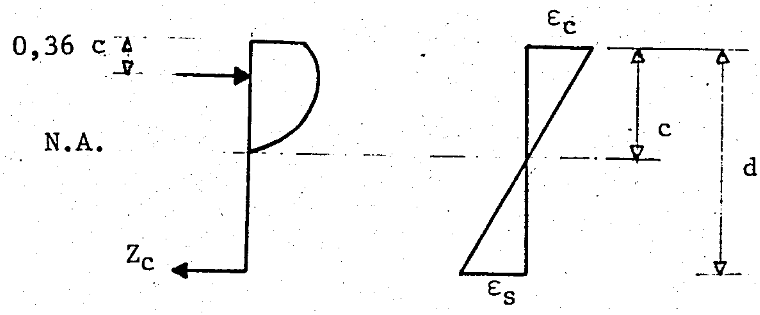

Figure 6- Ultimate strength for Ytong aerated slabs

$$D_b = 0.60\ b\,c\,\frac{2}{3}\,f'_{c_{cube}}\,\frac{\varepsilon_c}{\varepsilon_{cu}} \tag{5}$$

$$z = d - 0.36c \tag{6}$$

Here;

- b = Compression zone breadth of concrete (cm)
- c = Depth of the neutral axis
- $f'_{c_{cube}}$ = Cube strength of concrete kg/cm^2
- ε_c = Compressive strain for the top fibre of the concrete section in %o.
- ε_{cu} = Maximum allowable compressive strain for lightweight concrete (%o 2)

2.8. SHEAR EFFECT FOR REINFORCED LIGHTWEIGHT CONCRETE BEAMS AND ONE-WAY SLABS

Reinforced concrete members are-usually effected by shear forces. But it is rare that this forces exists by itself alone. Besides shear, almost always flexural, axial and sometimes torsional forces effect the element. Especially in flexural elements, shear strength mechanism is closely related to the bond strength, therefore the the anchorage of the reinforcement.

The failure of reinforced concrete elements should be because of flexure and not shear. In other words, flexure must be the determining effect for ultimate load. Actually, shear failure, which takes place as a result of shear forces and bending moments, is usually a result of small deformations and must be avoided because it is brittle and sudden. Especially in earthquake areas, the load-bearing elements in the structures should be designed never to lose their strength due to shear. Therefore shear strength of the element should always be greater than flexural strength.

When the behaviour of beams without shear reinforcement is investigated, it is appropriate to apply the loads on the beam as two concentrated loads symmetrical with respect to the midspan. Research done on such beams has shown that, the type of failure depends primarily on the "shear span (a_v)/effective depth(d)" ratio, a_v being the distance from the support to the nearest applied load. Beams with a_v/d ratio less than or close to 6 lose their strength due to shear. Factors other than this ratio also effect the shear strength of the concrete. These are percent of the tension reinforcement, concrete strength and type of the aggregate. Tension reinforcement prevents the vertical movement of the faces of a cracked section relative to each other. Concrete resists the shear force at the uncracked compression zone and aggregate is responsible for the friction and interlocking force along the surfaces of the diagonal crack. The value of the nominal shear stress, $\tau = \frac{V}{bd}$, quikly rises when M/Vd ratio is approximately less than 2, for a cross-section of the beam which has reached its ultimate strength. When a singular concentrated load is applied on the beam, the critical value for M/Vd form under the load: $\frac{M}{Vd} = \frac{a_v}{d}$(27).

According to Regan(28), who tested reinforced aerated concrete slabs and beams for the shear effect, shallower beams gave higher unit resistances than deeper ones. This factor was greatly influenced by the ratio of main steel. Compression steel appeared to have no influence at this stage.

The slabs tested were 2.0 m long, 600 mm wide and 100 mm deep. Ratio of longitudinal tension steel varied between 0,19-0,67 % and the a_v/d ratios between 2,3-5,4. Dry density of the material was 600 kg/m^3 with an average strength of 4.4 N/mm^2. Reinforcing steel had a yield limit f_y = 324 N/mm^2 for 6 mm bars of plain round mild steel. In the slabs with the lowest percentage of tension steel, flexural cracks were formed in the regions of maximum moment but not for enough out in the shear spans to develop into serious shear cracks. The failures were then flexural with yielding of the reinforcement and subsequent crushing of the compression zone.

Because of the relatively low bond strength, the reinforcement of aerated members often relies on welded cross-bars or stirrups which transfer load by bearing.

In pull-out tests by Sell(29), for which bond was prevented by oiling of the bars and anchorage was only by cross-bars, the load/deformation characteristic was almost elasto-plastic. For a 5 mm diameter cross-bar 80 mm long the limiting bearing stress was about 7 N/mm^2 (approximately 1,8 times the compressive strength of the concrete. This value is for relatively flexible cross-bars. An increase of strength to 9 or 10 N/mm^2 would be likely with a stiffer bar. The greater spacings of the longitudinal bars (e.g. > 100 mm) reduces the stiffness of the cross-bars very considerably and thus make the distribution of bearing stress much less even.

Once shear cracking has occurred, the initial structure of a member without shear reinforcement is a simple arch or strut and tie system and the principal cause of failure is the destruction of the anchorage of the tie. If N_{su} is the maximum tie force that can be sustained(28),

$$M_u = N_{su} \cdot z \tag{7}$$

The internal lever aim, z, can be taken as approximately "0,85 d", whence

$$M_u = 0{,}85\ d . N_{su} \quad \text{or} \quad V_u = 0{,}85 \frac{d}{a_v} N_{su} \tag{8}$$

For a shear span with a constant shear force, the critical section for shear failure can be taken to be at a distance h/2 from the high moment end.

Shallow slabs all had rather high shear resistances than deep beams in the experiments done by Regan(28). This would appear to have been due to the relatively high bond stresses produced in the deeper beams where the main steel was a small number of relatively large bars.

After shear cracking, most members without shear reinforcement can support some increase of load before failure by the arch and tie system. The resistance of such systems is probably of little interest in design partly because of the severe damage accompanying their formation and partly because they place great demands on anchorages which could be difficult to fulfil at practical (short) bearings.

CHAPTER 3
EXPERIMENTAL INVESTIGATIONS

3.1. MATERIALS USED

3.1.1. Aggregate Specifications

a) Sand

The sand used in perlite and styropor slabs was Riva sand. Results of the sieve analysis is given below:

Table 2- Sieve analysis for the sand used

	% Passing (by weight)					Specific Gravity kg/dm^3	Loose unit weight
Sieve openings (in mm)	0,25	0,50	1	2	4	2,66	1,54
	12,8	66	95	100	100		

b) Perlite

b.1) Typical chemical composition of perlite aggregate is as given below(30):

SiO_2 - % 71-75
Al_2O_3- % 12,5-18
K_2O - % 4-5
Na_2O - % 2,9-4
CaO - % 0,5 - 2

Fe_2O_3 - % 0,5 - 1,5
MgO - % 0,1 - 0,5

b.2) The perlite aggregate used is obtained from Pabalk company. Results of the sieve analysis for this is given below:

Table 3- Sieve analysis of the perlite aggregate used.

Openings in mm (Square mesh sieves)	Passing (%) (by weight)
4	100
2	59
1	44
0,5	28
0,25	14

This values conform to the curves given by ASTM C-332 about structural perlite aggregates.

b.3) The unit weight of this aggregate is as given below:

Table 4- Unit weights of the perlite aggregate used.

Perlite	Loase unit of (kg/m^3)	Specific gravity (kg / m^3)	Water absorbed by weight (%)
Coarse (0-3 mm)	79	450	174.0
Fine (0-1 mm)	56	420	

According to the manifacturer, this aggregate absorbs approximately % 60 water by volume. This value is guite high with respect to the laboratory values given above(2).

c) Pumice

The pumice lightweight aggregate used was brought from Kayseri-Develi district. The floating particles on water were collected and oven dried completely before mixing. Floating the pumice aggregates also ensured the elimination of all impurities which sank down.

c.1) Specific gravity of this aggregate has been found by Taşdemir as 2410 kg/m^3(11). The unit weights are as given below:

Table 5- Unit weights of the pumice aggregate used.

Agg.part (mm)	Unit weights (kg/m^3) Oven dry	Unit weights (kg/m^3) Saturated	Porosity(%)	Loose unit weights (kg / m^3)
2/4	800	1040	67	410
4/8	780	1000	68	400
8/16	740	930	69	380

c.2) Absorption

The percent of water absorbed by pumice aggregates for various lengths of time with respect to their day weights are given below(11).

TAble 6- Percent of water absorbed by pumice aggregate used with respect to their dry weights

Agg. part (mm)	Water absorbed (%)					
	10 min	30 min	90 min	18 hours	24 hours	72 hours
2/4	-	18,0	22,4	28,1	29,5	37,9
4/8	14,5	17,8	21,6	25,8	29,0	38,0
8/16	15,7	17,3	22,9	25,1	29,9	38,9

These values conform to the ones given in the Turkish Standards(33).

c.3) Modulus of elasticity and compressive strength of this aggregate(11)

According to the experiments done on 3x3x6 cm and 4x4x8 cm prisms, the average "E" was found to be 1700 N/mm^2 and compressive strength 4,7 N/mm^2.

As seen from the experimental values for pumice stone, this aggregate consists of about 70 % porosity. Therefore it is possible to accept it as cavity in the mortar compared with the normal aggregate, since the modulus of elasticity of pumice is about % 2 of the limestone from İstinye district and the compressive strength 5 % - 7 % of the same stone.

c.4) Chemical composition of the pumice agg. used(11):

SiO_2(free)	: 6 %	CaO	: 1,80 %
SiO_2	: 84,30 %	MgO	: 0,60 %
Al_2O_3	: 2,44 %	Loss of ignition (1000^oC)	: 4,10 %
Fe_2O_3	: 0,36 %	Unknown	: 0,40 %

d) Styropor

As mentioned earlier, the modulus of elasticity and compressive strength of styropor lightweight aggregate can be taken as zero compared to mortar strength.

Table 7- Loose unit weights for the styropor aggregate used.

d.1)				
	Agg.part (mm)	1/2	2/4	4/8
	Loose unit weight(kg/m^3)	34	27	16

d.2) Chemical composition of styropor granüles(31) is as given below:

	N_2	O_2	CO_2	CO	H_2	CH_4	C_2H_4	Styrol	Halogen
Styropor P	80	15	4	0,3	-	-	-	Trace	-
Styropor F	75	10	9	4	0,5	0,5	0,2	Trace	Trace

3.1.2. Cement Characteristics

All the cement used was PÇ 325, being the production of Akçimento factory of the same day.

Mechanical characteristics:
(Strengths according to Rilem-Cembureau method)

Compressive Strengths (N/mm^2)		Flexural Strengths (N/mm^2)	
7 days	28 days	7 days	28 days
24.2	39.8	4.8	6.7

Physical characteristics:

Blaine specific surface area(cm^2/g)	3600
Remaining on the 90 μ sieve (%)	5.0
Remaining on the 200 μ sieve (%)	0.2
Specific gravity (g/cm^3)	3.010
Normal consistency for water (%)	28.0
Total openings for Le Chatelier needles (mm)	3.0

3.1.3. Steel characteristics

As the reinforcement used in the slabs was cold-drawn St III b hard steel which was used in the Ytong slabs as welded wire fabric, the yield limit, f_y, is found by %o2 remaining deformation method. The diameter of the bars is the averages of five readings for each bar.

Table 8- Characteristics of the steel used in the slabs.

	Diameter (mm)	fy N/mm^2	fu N/mm^2	Es kN/mm^2	Total Strain (%)
	4.2	660	670	206	8.0
	4.2	665	675	201	7.5
	4.2	660	665	183	7.0
	4.3	635	670	195	10.2
	4.3	675	690	193	8.75
Average values	4.25	660	674	196	8.3

3.2. MIX DESIGN

It was accepted at the beginning of this work that the unit dry weights of the slabs and the cements would be the same for all samples. The cement content was chosen as 350 kg/m^3 and the density of the slabs were aimed to be 800 kg/m^3.

Mix design for lightweight concretes is more complicated than for normal concretes. The main reasons for this are the absorption of water by the aggregates and volume changes (shrinkage) of the aggregates in the mortar. As water is absorbed from the mortar phase, the volume of water and air in the phase changes. This can show variations for different types of aggregates. Therefore it is difficult to calculate the air content as for normal concrete. For fresh lightweight concrete, the volume other than for aggregate and cement is given as the "remainder" (water + air). It is not preferable to give the air content by itself unless calculated by any other method(11).

In this work, "absolute volumes method" is used to calculate the mix proportions, which is one of the most widely used methods. In utilizing the absolute volumes method, the volume of plastic concrete produced by any combination of materials is considered equal to the sum of the absolute volumes of cement, aggregate, net water, and entrained air. Proportioning by this method requires the determination of water absorption and the bulk specific gravity of the separate sizes of aggregates in a saturated surface-dry condition. The principle involved is that the "mortar" volume consists of the total of the volumes of cement, fine aggregate, net water, and entrained (or entrapped) air. This mortar volume must be sufficient to fill the voids in a volume of dry, rodded coarse aggregate, plus sufficient additional volume to provide satisfactory workability(6).

The mix proportions are calculated as follows:

C: Cement content in 1 m^3 fresh concrete (350 kg)
W: Water content in 1 m^3 fresh concrete, dm^3
δ_c: Specific gravity of cement, kg/dm^3
δ_{ai}: Unit weight for "i" aggregate, kg/dm^3
P_i: % of the "i" aggregate in the total volume of the aggregates
V_A: Total volume of the aggregates, dm^3
∂: Volume of air in 1 m^3 fresh concrete, dm^3
w: W/C: Effective water/cement ratio (by weight)

Therefore, total volume of the coarse aggregate is:

$$V_A = 1000 - \left(\frac{C}{\delta_c} + W + \partial\right) \tag{9}$$

If G_i is the weight of the "i" aggregate, the mix is calculated as

$$G_i = W_a \times P_i \times \delta_{ai} \tag{10}$$

$$W = w \times 350 \tag{11}$$

According to this method, the mixes of three different types of concretes are as such:

1) Perlite concretes (1 m^3)

$C = 350$ kg
$W + \partial = 505$ dm^3
Perlite = 108 kg 54 kg fine
54 kg coarse

Sand = 67 kg

The theoretical unit weight of the fresh concrete is thus:

$$\Delta_{teo} = 350 + 505 + 108 + 67 = 1030 \ kg/m^3$$

Since about half of the water is expected to evaporate from the concrete, density at the end of 56 days should be around 800 kg/m^3. The water content is found from the trial mixes mixed by hand in order to prevent the crumbling of the aggregate. This value for water content can be reduced if a mixer is used.

2) Styropor concrete

$C = 350$ kg/m^3 $\frac{W}{C} = 0.45$
$W = 158$ dm^3
$\partial = 20$ dm^3 (20 %o accepted)

Styropor aggregate
Fine (28%) = 6,75 kg
Medium (28%) = 5,36 kg
Coarse (29%) = 3,29 kg
Sand (15%) = 282 + 100 kg (added in practice)

$$\Delta_{teo} = 350 + 158 + 6,75 + 5,36 + 3,29 + 282 = 805 \ kg/m^3.$$

But from the trial mixes, unit weight of the fresh concrete was found to be lower than 800 kg/m^3. Therefore it was decided to put another 100 kg of sand into the mix. For a small volume of styropor concrete (10 lt) this could be reasonable. But when 120 lt of this concrete was poured for the slab, it was seen that the unit weight of the fresh concrete was much greater than 800 kg/m^3, namely 1150 kg/m^3. Therefore it was decided that styropor was compressed under the dead weight of fresh concrete for relatively greater volumes of it. So, for the second slab, sand content was reduced to 70 kg/m^3. In order to balance the volume, styropor aggregate content was increased, leaving the cement and water contents as before. This time, unit weight came up to be around 750 kg/m^3. For the third slab, the mix design for 800 kg/m^3 was taken considering % 10 reduction of styropor by volume. After the mixing, the change in the volume was calculated and was found out that styropor shrinks about % 16 of its original volume for a 130 lt mix of about 800 kg/m^3 unit weight. For greater volumes of concrete, this ratio is expected to rise since compression will be more under greater masses.

3) Pumice concrete

C= 350 kg/m^3 $\qquad \frac{W}{C} = 0,45$

W= 158 lt $\qquad$ Pre-soaking water=%18 of the total weight of agg. = 78 lt.

∂= 171 lt (with air-entrainment)

Pumice agg. (oven-dry weights)

d(2/4)=152 kg (%34)

d(4/8)=144 kg (%33)

d(8/16)=136 kg(%33)

$\frac{0,61}{122}$ = 0,005 (by weight of cement content) air-entraining agent is to be used for 17.1 % air(32).

$\partial = 0,005 \times 350 = 1,75\ kg/m^3 = 1750\ g/m^3$

Pre-soaking water = 78 lt.

$\Delta_{teo} = 350 + 432 + 158 + 78 = 1020\ kg/m^3$

This value is considerably high from 800 kg/m^3, which is claimed to obtain for dry concrete. From the trial mixes, it was seen that more than 20 % of the pumice concrete by weight, is free water when compared with the oven-dry samples. Therefore, theoretical unit weight is taken approximately 20 % more than 800 kg/m^3.

Table 9- Fresh concrete characteristics

Real quantities of ingredients in 1 m^3 fresh concrete

Kind of Concrete	Cement, kg	Total Water lt	Pre-Soaking Water, lt	Sand kg	Total Light-weight aggregate, kg	Air Entrain-ing kg	Fresh unit weight	Effec. W/C ratio	Total W/C ratio
Pe	346	500	250	66	107	-	1018	0,72	1,44
St	402	182	-	324	17,7	-	917	0,45	0,45
Pu	315	213	70	-	389	1,58	907	0,45	0,68

(The values are the averages for each kind of concrete).

3.3. PRODUCTION, MIXING, PLACING AND CURING OF THE CONCRETES

The mixing of perlite and styropor concretes were done by hand on impervious ground. For perlite concrete, this was more convenient than mixer because of the crumbling of the aggregate when harsly mixed. For this concrete about half of the total water was mixed with the aggregate and this was blended thoroughly until all the aggregate was wet, then the remaining water, cement and sand was mixed. The reason for mixing styropor concrete on the ground was its being a very volatile material. A thoroughly closed mixer should be used when using styropor in the concrete production plant. For pumice-concretes, Tricosal LP was used as air-entraining agent to reduce the density of the concrete. First, the aggregate was mixed with the pre-soaking water for about half a minute in a drum-type mixer. The air-entraining agent was mixed in the remaining water until bubbles were formed. Then cement and water was added into the mixer which were mixed for about 60 seconds. The use of air entraining agent increased the workability and cohesiveness, while segregation and bleeding were reduced considerably(33).

Since this work is mainly a comparison test between four different types of lightweight concrete slabs, the dimensions and reinforcement were also held nearly equal to Ytong aerated slabs of dimensions 10x47x198,5 cm. The mold for the slabs was prepared from steel with special prevention against lateral displacements. Although its height was about 5 mm higher than 10 cm, this difference is not considered as a great handicap to influence the comparison results. All the concretes were placed into the mold by compaction with wooden blocks. The sides and bottom part of the mold was greased before pouring the concrete in. Vibrator was not held into the concrete but onto the mold of the slab to ensure the settlement of the concrete in the edges and corners. Also 15x30 cm cylindrical samples were taken from the same mixes for slabs to make compression and deformation tests on them. For these samples, vibrator was held into the concrete but for a very short time to prevent segregation. For slabs, the concrete was screeded and slightly troweled to obtain a flat surface.

After this, they were covered with polyethylene sheets and wet sacks. The molds were taken out in 2-3 days, while the concrete was wet cured for 7 days in an environment of $10^{o}C \pm 2^{o}C$. Afterwards, slabs and cylinders were carried to a room with a relative humidity of 65 % ± 5 % and temperature $20^{o}C \pm 3^{o}C$ and air cured until the day of test.

Tests were done on the 56th day rather than 28th day from the time of mixing. The reason for this was that because until that day, lightweight concretes continued to gain strength and after this the increase of strength was very much reduced(9). Aerated Ytong concretes gained all their strength during the production because of the autoclave process. Therefore strength comparisons would be more sound on samples of 56 days of age. Also, another reason was that the perlite slabs and cylinders were still wet at the end of 28 days, which would adversely effect the test results.

Test slabs were brought to the test room and were put on the supports the day before the test. Cylinders of the particular slab were also brought to this room and stayed in the same conditions with the slabs.

3.4. REINFORCEMENT OF THE SLABS

The mesh reinforcement used in the slabs were made of average 4.25 mm plain, round, cold drawn St III b hard steel wires with welded cross-bars of the same steel. The reinforcement consisted of 4 ϕ 4.25 bars as bottom steel for tension and 2 ϕ 4.25 bars as top steel for compression (constructive reinforcement). The concrete cover was taken as 1.5 cm for the bottom steel and 1.5 cm for the top steel (for h=10.4 cm) or more with increasing height of the slabs.

The top and bottom reinforcement were connected, except in the aerated slabs, with welded vertical legs to control the distances between the top and bottom reinforcement and between the bottom steel and the lower face of the slab. The details for the reinforcement are given below.

All the slabs were under reinforced according to eq.4.

(All dimensions are in cm)

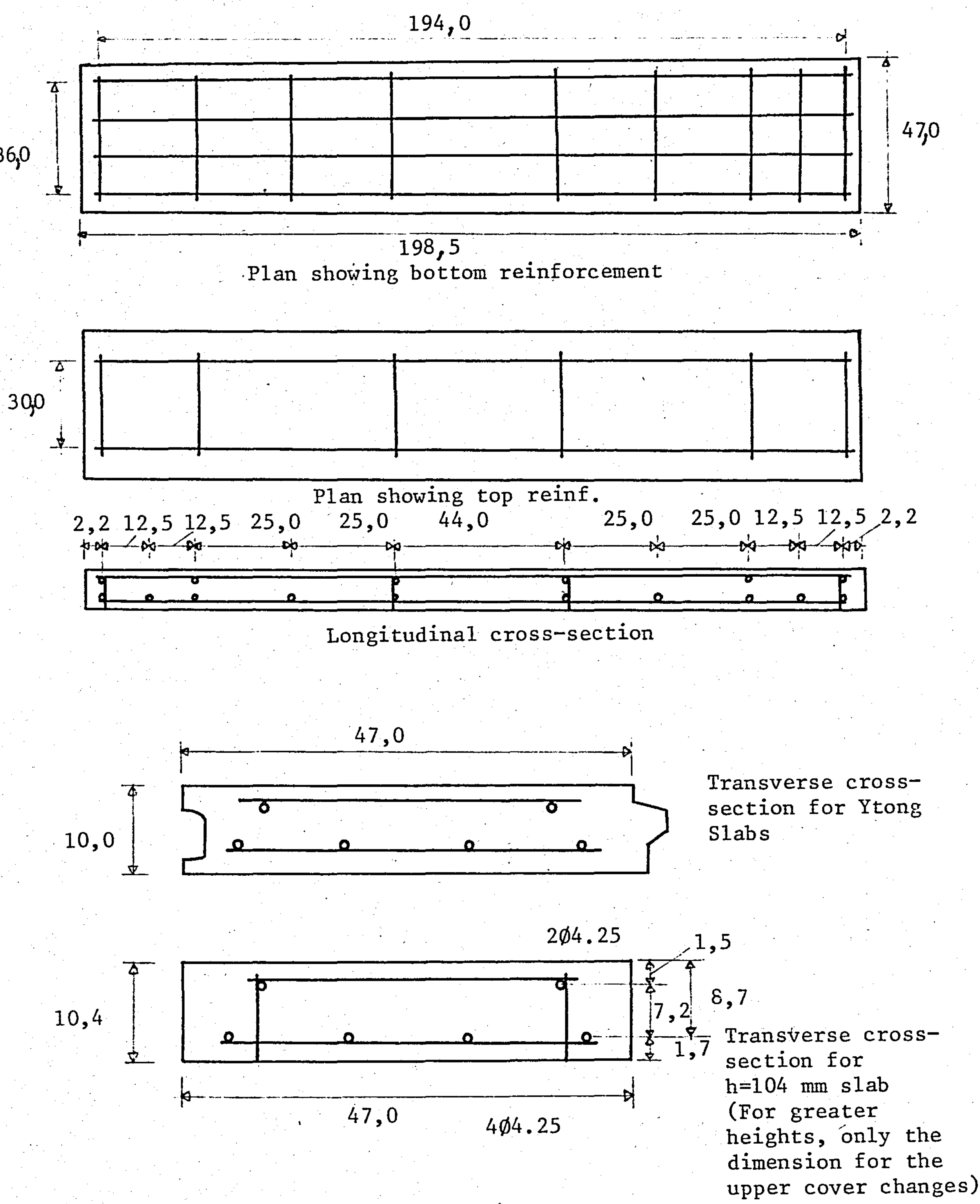

Fig.7- Details for the test slabs.

3.5. FLEXURAL TESTS FOR THE SLABS

Flexural tests were performed on the slabs using quarter-point loading as explained in TS 453(34) and TS 2823(35). Test set-up is as shown below (see also Fig.14):

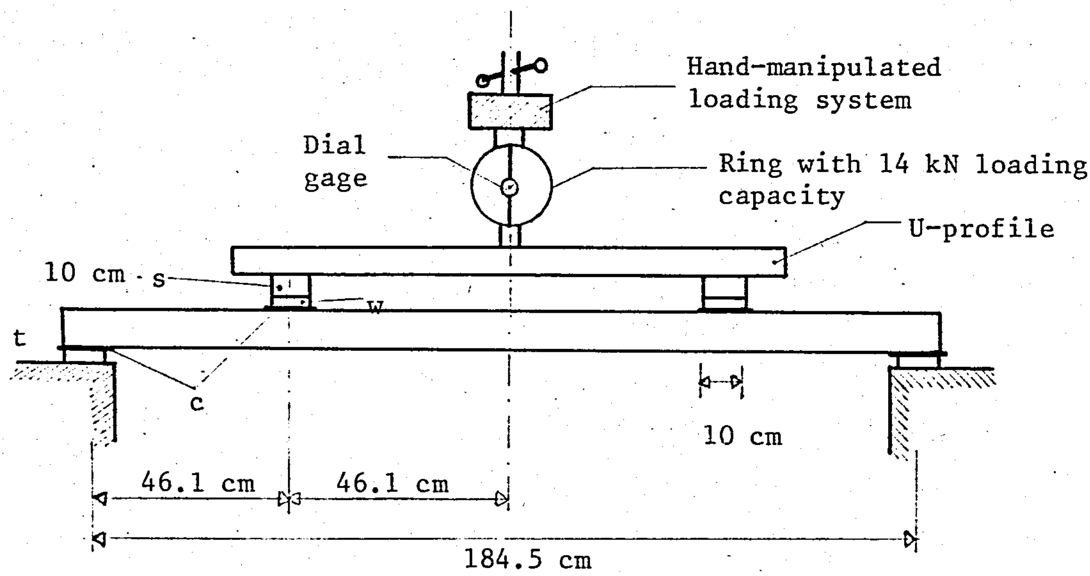

Figure 8- Set-up for flexural tests of the slabs

s = steel plate of 18 mm thick and 500 mm long.

w = wooden plate of 13 mm thick and 500 mm long.

c = corrugated cardboard used for the loading to be uniformly distributed.

t = steel plate of 10 mm thick and 500 mm long.

For the calculations, 1/3 of the width of the support is added to the clear span, leaving the remaining support width as 7 cm.

The total weight of the U-profile and the steel plates on the slab were 25 kg. This weight was not included in the calculations in order to stay on the safer side unless otherwise stated.

Loading of the slabs was done according to the procedur given by CEB(25), with the duration between subseguent loadings modified for our case. CEB gives the duration between two loadings for the beginning of the test as 15 min. This was taken as 6 mm. as the loads were very low compared to the ones for normal concrete beams. Loading program used for the tests is given in fig.9 below.

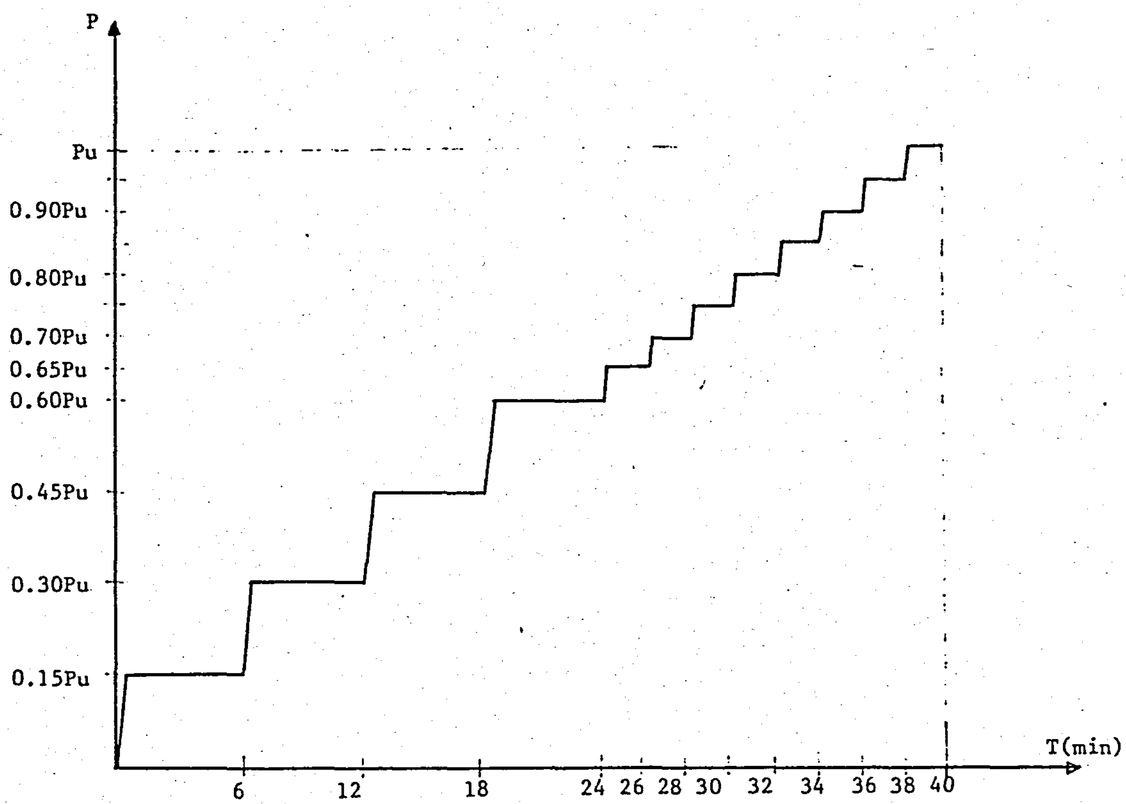

Figure 9- Loading program for the flexural tests of the slabs.

It was verified that the materials used for loading and for support would not undergo any serious deformation under the test loads.

After every loading, cracks were marked on the slab and crack widths were read with a Feeler Gage of 0,1 mm sensitivity.

At the end of every loading period, deflections for the quarter and middle points of the slabs were read with 0.01 mm sensitive dial gages.

For each loading, longitudinal strains were read on five locations along the height of the slabs using Demec points on the surfaces of the concrete, in the middle part of the span, on a 200 mm gauge length. A strain of $8x10^{-6}$ could be real in one division of the scale on the dial.

Also, short-term compressive tests were done on 15x30 cm cylinders taken from the batches for every slab. From these test results, stress-strain curves were drawn and modulus of elasticity values were calculated. For the compression tests, a universal hydraulic testing machine was used.

CHAPTER 4
RESULTS AND DISCUSSIONS

4.1. RESULTS OF CYLINDER TESTS

The values of compressive strengths and air-dry unit weights for 56 days old samples with the oven-day and fresh unit weights are given in the table below. The modulus of elasticity values are the averages of three tests for each type of material. All samples were 15x30 cm cylinders. The symbols given in the beginning of the table are the respective slabs for the samples.

Table 10- Results of the cylinder test

Slab No. and Type	f'_{c56} kg/cm^2 (N / mm^2)	Δ_{fresh} kg / m^3	Δ_{56} kg/m^3	$\Delta_{oven\ dry}$ (kg/m^3)	$E_{c_{av}}$ kg/cm^2 (N/mm^2)	Average Moisture Contents (%)
Pe 1	22.0(2.2)	985	700	610	21800	
Pe 2	23.3(2.23(	1020	755	637	(2180)	0.183
Pe 3	25.5(2.55)	1050	760	625		
St 1	33.5(3.35)	1150	1060	1035	-	
St 2	14.7(1.44)	710	680	655	20600	0.045
St 3	19.7(1.97)	890	805	745	(2060)	
Pu 1	44.8(4.48)	890	855	785		
Pu 2	50.8(5.08)	920	865	795	35300	0,088
Pu 3	42.1(4.21)	910	810	745	(3530)	

(The value for the unit weights are approximated to the nearest 5 kg/m^3).

The "E" values are calculated from the "σ-ε" diagrams, as average slopes of the secants, connecting points corresponding approximately to the $\frac{1}{3}$ f'_c values to the origin. Typical σ-ε curves are given for these concretes in Fig.12.

Also, splitting tensile strength tests were done on two cylinders from mixes of Pe 2 and Pe 3 slabs, which failed due to shear, and the values found for "f_{ct}" were 3,0 kgf/cm^2 (0,3 N/mm^2) and 2,7 kgf/cm^2 (0.27 N/mm^2) respectively.

The cylinders for Pe 1 was weighted in three weeks after oven-dried, and their moisture content came up to be % 2.4 with respect to their oven-dry weights.

4.2. RESULTS OF FLEXURAL TESTS FOR THE SLABS

The loads of failure given for the perlite and Ytong aerated slabs were the complete failure loads, causing diagonal shear failure-for the perlite slabs and complete breakdown for the Ytong slabs. For the other two types of slabs, namely styropor and pumice slabs, finding an exact load of failure was not so easy because of the ductile behaviour of the slabs. Therefore, loads causing excessive deformations and causing the break off of one of the tension steels were accepted as ultimate loads. Beyond those, the slabs were still able to carry extra loads.

Twelve slabs were tested, being three for each type of material, until failure. Two among these were not taken into evaluation, these being Pe 2 which was damaged during its curing period and Yt 2 for which the set-up was spoiled during the test.

The characteristic loads with the dimensions and unit weights of the slabs are given below:

Table 11- Results of the flexural tests of the slabs

Slab Type and No	Dimensions (cm)	Weight of the slab (kg)	Unit Weight of the slab (kg/m^3)	Initial* Crack load (kg)	Load for 0.4 mm crack width (kg)	Load of Failure (kg)
Pe 1	10.6x47x198,5	81.1	820	250	250	800
Pe 3	11.0x47x198,5	85.5	835	170	210	770
St 1	10.4x47x198,5	109.3	1125	300	560	1275
St 2	11.0x47x198,5	69.3	675	270	665	1060
St 3	10.6x47x198,5	83.1	840	230	420	980
Yt 1	10.0x47x198,5	68.3	730	450	630	970
Yt 3	10.0x47x198,5	68.9	740	400	665	950
Pu 1	11.0x47x198,5	85,8	835	200	525	1350
Pu 2	11.0x47x198,5	84.5	825	220	490	1400
Pu 3	11.0x47x198,5	85,5	835	270	630	1450

4.3. FAILURE MODES FOR THE TEST SLABS

All the slabs except the perlite ones lost their carrying capacities by flexure. This can be seen from the Fig.10 below.

One of the main differences between the types of failure of Ytong aerated slabs and the others was that while the section of failure was near the mid-span for the Ytong slabs, in all others it took place under one of the applied loads at the quarter points. This was because of the higher rigidity of the aerated slabs compared with the others. Generally two major cracks developed under both loads, one of

* Initial crack loads are not exact loads but approximations since the cracks were investigated after every loading and not during the loading.

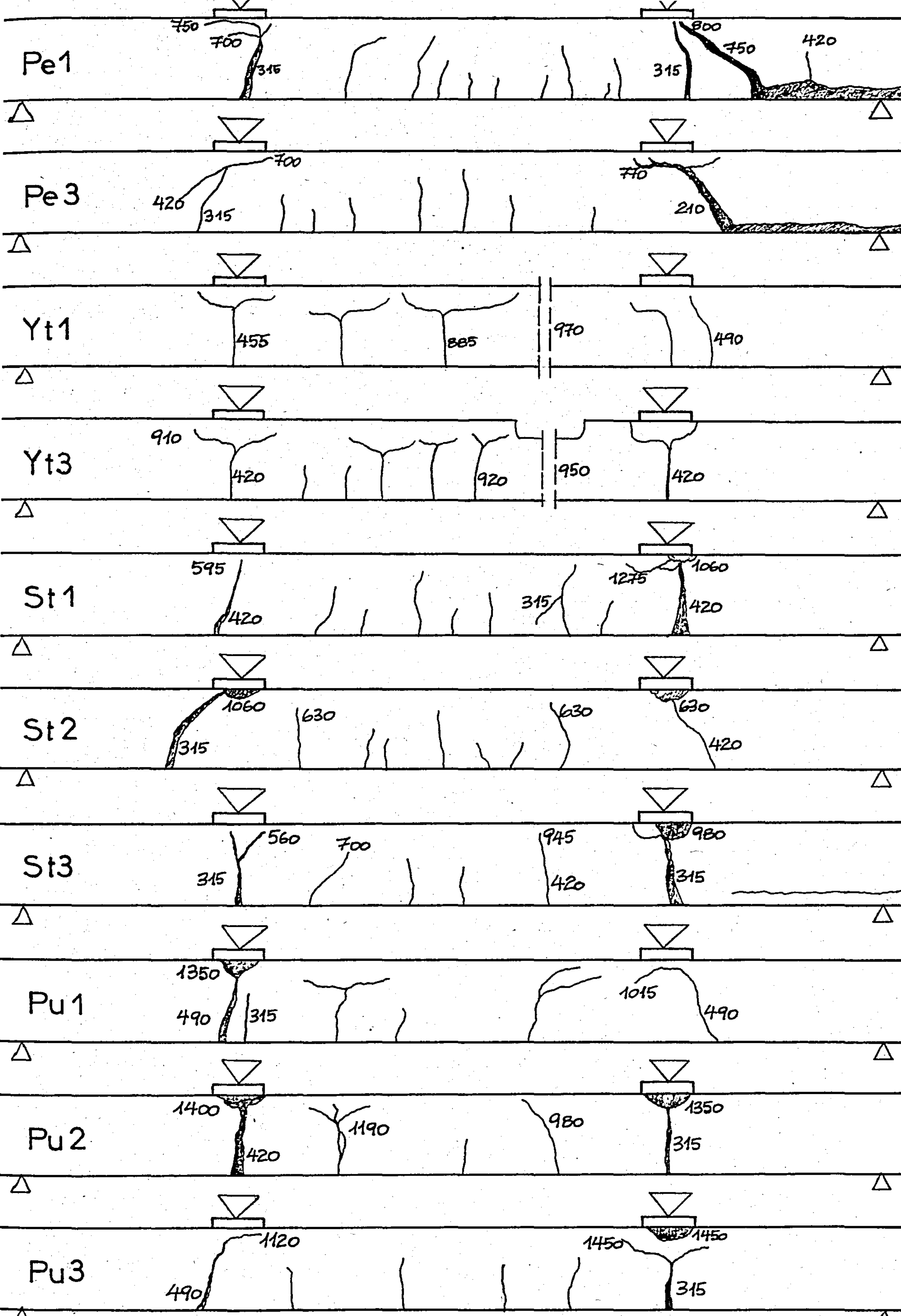

Figure 10- Failure modes of the test slabs.

them growing excessively for higher loads and causing tension and then compression failure, or shear failure as for the perlite slabs.

All perlite slabs lost their carrying capacity by diagonal shear. Once shear cracking had occurred, the slab was a simple arch or strut and tie system and the principal cause of failure was the destruction of the anchorage of the tie (Fig.15,16). The first diagonal crack very quickly grew up to be the major crack, extending along or near the main steel all the way to the support. Once this happened, the slab was seriously damaged, but it was able to withstand some increase of load prior to failure by splitting the concrete around the end anchorages of the main bars.

For the styropor slabs, flexural cracks were formed in the regions of shear spans but not far enough out to develop into serious shear cracks. The failures were then flexural with yielding of the reinforcement and subsequent crushing of the compression zone. But this was not sudden and destructive and even after one of the main steels for these slabs had broken out, they were still able to carry increased loads. Some splitting of the cover near the supports occurred, but this did not cause the anchorages lose their bearing strength. For St 1, due to its higher density, the carrying capacity was more than the other two styropor slabs, splitting of the cover was minor for this slab than it was for St 3. It is interesting to note that St 2, which had a lower density than St 3, showed much less splitting and a higher carrying capacity than the latter. Seeming paradoxical, the increased amount of sand for St 3 can be the reason for this, since there is no other difference between these two. The sand used for St 3 was six times more than for St 2. It can be thought that sand reduced the ductility of the material making it stiffer, and causing a more brittle failure due to wedge action and splitting of the concrete in front of the tie-bars. Also, another reason for the initial crack load of St 2 being higher, in spite of the increased deflection and decreased height of this slab than St 3, can be the excessive deformation capacity of this high styropor-low sand slab (Fig.17,18 and 19).

For the Ytong aerated slabs, the situation was just the opposite. The compressive strength of this concrete was the highest among the four different concretes (>5 N/mm^2), but showing a brittle character. Since practically there was no bond between the steel and concrete, because of the bituminous coating on the steel, the cracks were sudden and deep. They were almost purely flexural, a small number of them showing slight inclinations from the vertical. Since the ratio of the steel for Ytong slabs was much less than the balanced ratio, the failure of these was due to tension, with all the main steel breaking out nearly at the same time in the section of failure, breaking the slab into two pieces (Fig.20).

All the pumice slabs reached their ultimate strengths by the yielding of the steel followed by the crushing of the concrete in the compression zone (Fig.21). Their failures were also ductile like those of the styropor slabs, making excessive deflections before one of the main steels broke off. After this, they were still able to carry loads for slightly higher loads. Among the four types of slabs they had the highest load-carrying capacity.

4.4. SOME COMMENTS ON THE TEST SLABS

Since the failures of the perlite slabs were due to shear, using stirrups in the critical sections could increase the load-carrying capacities of these slabs considerably. According to the tests performed by Regan on aerated beams and slabs(28), the increase of strength produced by the vertical stirrups was modest, being about 17 % of the strengths of comparable basic beams, where using inclined stirrups ultimate loads were raised by almost 50 %. Also, the use of mild steel for the main reinforcement instead of cold-drawn hand steel would probably decrease the effect of shear cracks of the perlite slabs making them more ductile and the mild tension steel yielding due to flexure before these cracks could cause a sudden failure. But it is also advisable to use hard steel of greater radius than the main steel for the tie-bars, to increase the bearing force of the anchorages and to prevent the early lateral deformation of the tie-bars.

Also, keeping the water content high to increase the workability of the perlite concrete adversely affected the drying time for this concrete. It can be expected that the compressive and therefore tensile strengths were reduced and shrinkage was increased due to this high water content, although shrinkage cracks were not observed on these slabs. Therefore, introducing air-entraining agent into the perlite mix, like for the pumice concrete, should increase the workability and reduce the W/C ratio. If it is necessary to mix this concrete by hand, then the content of pre-soaking water should be kept as low as possible and the perlite aggregate should be mixed thoroughly with this water until a plastic consistency is attained before adding the other ingredients.

The styropor slabs, showing highly ductile and elastic properties and with expected high impermeability against water and moisture, were the most expensive among the four types of slabs produced. Also, styropor having practically no resistance against fire was a great disadvantage of this material from the point of structural use. Since the cost of perlite and Ytong aerated slabs were nearly the same, the most economical of these slabs came up to be the ones made of pumice, which also showed the highest load-carrying capacities and ductility among these four types of slabs. The economy of pumice came from its abundant availability in Turkey without any need of extra energy for its production.

4.5. STANDARDS FOR SERVICE LOADS, DEFLECTIONS AND CRACK WIDTHS FOR THE LIGHTWEIGHT CONCRETE ROOF SLABS

The permissible service loads are taken from the Turkish handbook of practice of Ytong aerated slabs(36). According to this, the minimum additional loads other than the dead weight of the slab that a standard roof deck may carry is taken as 120 kg/m^2 for snow and wind loads plus 45 kg/m^2 for the insulation materials to be applied on the slab against rain and moisture (such as bituminous cover with gravel and sand). Of the 120 kg/m^2 live load, 75 kg/m^3 is assumed as the snow load on horizontal projection and 45 kg/m^2 as the wind load.

Under the loads given above, the deflection of the slab must not exceed $\ell/300$, ℓ being the free span. This value is far more conservative than the limit given in ACI Building Code 318-77, which is $\ell/180$, as short time deflection for flat roofs.

For allowable crack widths, recommendation of ACI is 0,4 mm for elements in normal atmospheres not subject to outdoor effects(37).

For structural roof decks the factor of safety method is recommended by the Turkish Standards(31,32). According to these the factor of safety against failure is;

$$n = \frac{P_k + R + W}{P_A + W} \qquad (12)$$

Where P_k = Failure load (kgf),

P_A = Maximum permissible load which the slab can carry other than its own weight (kgf)

R = Total weight of the loading system on the slab (kg)

W = Weight of the slab (kg)

For roof and floor decks, the factor of safety must be greater than or equal to 2,3 for failure load, and 1.25 for initial crack load. The factor of safety for initial crack is recommended only by the Ytong manifacturers but not by the standards.

The factors of safety of the tested slabs calculated according to Eq.12 are given in Table 12 below:

Table 12- Factors of safety for failure and initial crack loads for the test slabs

Slab No	FS For Failure	Average FS for Failure	FS for Initial Crack	Average FS for in crack
Pe 1	3.68		1.45	
		3.60		1.29
Pe 3	3.52		1.12	
St 1	5.14		1.58	
St 2	4.93	4.81	1.55	1.50
St 3	4.39		1.36	
Yt 1	4.56		2.33	
		4.51		2.22
Yt 3	4.46		2.11	
Pu 1	5.82		1.24	
Pu 2	6.05	6.03	1.32	1.36
Pu 3	6.23		1.52	

The maximum deflections permitted by the Ytong Handbook of Practice is $\ell/300$. This limit is $\frac{184,5}{300}$ = 0.6 cm = 6 mm for our case. The corresponding loads to this deflection value are taken from the load-deflection curves (Fig. 13), added with the 25 kg coming from the materials used for the tests, and are given below (Slab weights are not included in this loads). Also the deflections corresponding to the permissible service loads (other than the slab weight) with their averages for each type and the deflection ratios (δ/ℓ) with respect to span length are given in Table 13 is below.

Table 13 - Deflections of the slabs under service loads

Slab No	P(kg) Load Corresponding to max.permissible deflection(6 mm)	P(kg) Average Loads for Each Group	δ(mm) Deflections under Service Loads (mm)	δ(mm) Averages and δ/ℓ values
Pe 1	325		2.25	2.50
		285		
Pe 3	245		2.75	(1/738)
St 1	420		0.75	-
St 2	295	360	2.75	2.13
St 3	365		1.50	(1/866)
Yt 1	465		1.75	1.75
		425		
Yt 3	385		1.75	(1/1054)
Pu 1	345		1.50	1.33
Pu 2	345	340	1.25	(1/1390)
Pu 3	335		1.25	

4.6. THERMAL CONDUCTIVITY PROPERTIES OF THE TEST SLABS

The value of thermal conductivity, λ, is a specific property of a material and is a measure of the rate at which heat energy passes perpendicularly through a unit area of homogeneous material of unit thickness for a temperature gradient of one degree.

$$\lambda = \frac{W}{m.K} \quad \text{(SI units)}$$

Research on this subject has shown that λ values with respect to oven-dry densities of the various materials show a general dependence of λ mainly on density. According to the German standards(38), the λ values for the four different types of concretes with average 800 kg/m^3 densities, which are tested in this work, stay in the range of 0,21-0,29 $\frac{W}{mK}$ where a value of 0.25 $\frac{W}{mK}$ can be taken as the average thermal conductivity for them, since this range is very narrow. For dense concrete with siliceous or calcareous aggregate, λ is taken as 1.75 $\frac{W}{mK}$. Therefore, thermal resistance for these materials with 800 kg/m^3 unit weights are seven times more than the dense concrete with 2400 kg/m^3 unit weight.

The heat capacity of a material is also an important factor which has to be considered with the other thermal propreties of the material. This factor indicates the amount of heat stored in the material in a certain time interval. The formulas for heat storing and thermal inertia properties are given below(39).

$$S_{24} = 8,5 \times 10^{-3} \times \sqrt{\frac{\lambda.\rho.c'}{s}} \quad , \quad \frac{W}{m^2.K} \quad ;$$

$$D = 8,5 \times 10^{-3} \times s\sqrt{\frac{\rho.c'}{\lambda}} \quad ,$$

where

ρ = unit weight of the concrete

λ = Thermal conductivity

c' = Specific heat capacity

S_{24} = Heat storage property in 24 hours

s = Depth of the specimen

D = Thermal inertia property.

For all sorts of concrete;

$$c' = 0.25 \text{ kcal/kg}^{o}\text{C} = 1045 \frac{\text{W . s}}{\text{kg. K}}$$

For the slabs produced, taking $\lambda = 0.25 \frac{W}{m.K}$, $\rho = 800$ kg/m^3 and s = 0.10 m., heat storage and thermal inertia properties are as given below.

$$S_{24} = 0,19 \text{ and } D = 0,024$$

For normal concrete with a unit weight of 2400 kg/m^3, $\lambda = 1,75 \frac{W}{m.K}$ and s = 0.10 m.;

$$S_{24} = 0,87 \text{ and } D = 0,016.$$

4.7. CAPILLARY ABSORPTION OF THE CONCRETES PRODUCED

Capillary absorption of lightweight concrete is also an important factor from the point of its use in places subject to outdoor effects such as rain or snow. The formula for capillary absorption is given as

$$\left(\frac{Q}{A}\right)^2 = K.t \tag{15}$$

Where Q = Quantity of water which flows through the concrete sample

A = Cross-section area of the sample

K = Capillarity constant, showing how fast the water rises through the sample

t = Time interval for the water to rise in the sample.

The capillarity curves for the lightweight aggregate concretes produced are given in Fig.(11).

If W_t is the weight of the sample subject to capillary absorption after a time interval t, and W_o is the oven-dry weight, percent of absorbed water due to the concrete's dry weight is given by the below formula:

$$W_a = \frac{W_t - W_o}{W_o} \times 100\ \% \tag{16}$$

The values for W_a for perlite, styropor and pumice concretes are found as 34.5 % in 16 minutes, 9.8 % in 64 minutes and 6.8 in 64 minutes respectively. For perlite concrete, water rose from bottom to top face in 16 minutes, whereas for the other two types of concrete, this took 64 minutes.

From the above values and the capillarity curves given in Fig.11, it is seen that rate of absorption for perlite concrete is quite high with respect to styropor and pumice concretes, which have nearly the same rates of absorption. It is interesting that rate of absorption for pumice concrete is slightly lower than for styropor concrete. The reason for this can be the microscopic bubbles formed in the pumice concrete by the air-entraining agent, since these holes act as discontiniuties in the concrete and slow down the rate of capillary absorption.

4.8. COMPARISONS OF TEST RESULTS WITH THE THEORETICAL CONSIDERATIONS

According to Section 2.4, theoretical ultimate loads for the slabs can be calculated from egs. 1,2 and 5,6 (for Ytong slabs); taking $\varepsilon_{cu} = 0.002$, $E_s = 196$ kN/mm^2, $A_s = 56.8$ mm^2, $A'_s = 28.4$ mm^2, $f_y = 660$ N/mm^2, $(d-d') = 72$ mm and $f'_c = 5$ N/mm^2 (for Ytong).

The f'_c values for the other types of slabs are taken from Table 10. In the calculations, weights of the slabs were considered as concentrated forces acting on the quarter points like the applied loads, for convenience. The average theoretical ultimate loads and the average tests results for each type of slab are given in table 13 below:

Table 14- Comparison between the theoretical ultimate loads and the test results for the slabs

Type of slab	Theoretical Ultimate Loads for each type of slab	Test Results - Average Loads of Failure For Each Type of Slab
Pe	11.5 kN(1150 kg)	7.9 kN (790 kg)
St	11.7 kN(1170 kg)	11.0 kN (1100 kg)
Yt	9.5 kN(950 kg)	9.6 kN (960 kg)
Pu	12.5 kN(1250 kg)	14.0 kN (1400 kg)

For theoretical calculations, it was accepted that the failure was balanced, which was not exactly the case for the test slabs. As seen from the table, the ultimate load for perlite slabs can be raised considerably by avoiding their failure due to shear. For the other types of slabs, the relationship between the theoretical and practical values for the failure loads seems reasonable, considering the wide range of f'_c values for each type of concrete found from the test cylinders.

Also, the value of 0.002 for ε_{cu} agrees quite well with the test results, where strains measured at the mid-spans of the slabs were in the range of $(1500-2500)\times10^{-6}$. But for more precise analysis, the measurements should have been done at the sections under the loads (e.g. at quarter points for our case), since all lightweight aggregate concrete beams and slabs with very low rigidities failed at these sections.

From the limited data of the test results on the modulus of elasticity (E) and Poisson's ratio (ν) values for the styropor, perlite and pumice concretes, some general results can be obtained. It was seen that the "ν" values for these three types of concretes was about 0.20, which agrees quite well with the values given for lightweight concretes in various standards.

The "E" values, on the other hand, calculated from the formula given by the ACI Code(40) as

$$E_c = \omega_c^{1.5}\ 0.14\sqrt{f'_c} \quad \text{(metric)} \qquad (17)$$

for values of ω between 1440 and 2480 kg/m^3, stayed below the values found from the test results, since the density was 800 kg/m^3 for our case, which was much lower than the range given above. It is found that by changing the coefficient of 0.14 in the above formula to 0.20, the formula

$$E_c = \omega^{1.5}\ 0.20\ \sqrt{f'_c} \quad \text{(Metric)} \qquad (18)$$

agreed well with the test results.

CHAPTER 5
CONCLUSIONS

In this study, lightweight slabs produced as structural and insulating roof decks made with three different types of lightweight aggregates, styropor, perlite and pumice and Ytong aerated slabs were tested in flexure until failure. For comparison purposes dimensions, reinforcement and unit weights for these slabs were kept nearly equal.

The results found can be summarized as follows:

1) All the slabs gave acceptable results from the points of load-carrying capacities, deflections, initial crack loads and crack widths with respect to the accepted standards.

2) The failure types of styropor and pumice slabs were highly ductile, where Ytong and perlite slabs failed in a brittle manner. The ductility of the slabs can be considered as an advantage over the brittle failures.

3) All the slabs failed due to flexure, except perlite slabs, for which failure was due to diagonal shear. Using inclined stirrups is expected to increase the load-carrying capacity for the perlite slabs considerably.

4) In order not to lose the high thermal resistances of these slabs, and also to protect their reinforcement against corrosion, they have to be carefully insulated against moisture effects. For this purpose, a cement-latex coating on the reinforcement can be beneficial both to protect it against corrosion and to improve its bond strength with the concrete.

5) Of the four types of slabs, the ones made with pumice showed the highest load-carrying capacities together with lowest cost of production and capillary absorption, therefore being the most economical ones of all. Utilization of pumice lightweight aggregates can be recommended as beneficial for both structural and non-structural elements in buildings.

Since the scope of this study was restricted by the short-term analysis of the lightweight structural slabs, time-dependent effects such as creep and shrinkage were not analyzed. We recommend that these effects be analyzed, since they may change the results found by short-term loadings considerably.

Also, the shape of the stress block and maximum compressive strain for concretes with such low unit weights are not well-known. Therefore, for the purpose of calculations, these values have to be determined with precise experimental methods.

In this study, an economical analysis between these four types of slabs could not be realized as there was no known market or standard unit price for the pumice stone produced in Turkey. Nevertheless, the slabs made with pumice seemed to be the cheapest among the four types of slabs. A detailed economical analysis should be made for this stone since it is expected to play an important role in Turkey's building industry in the near future.

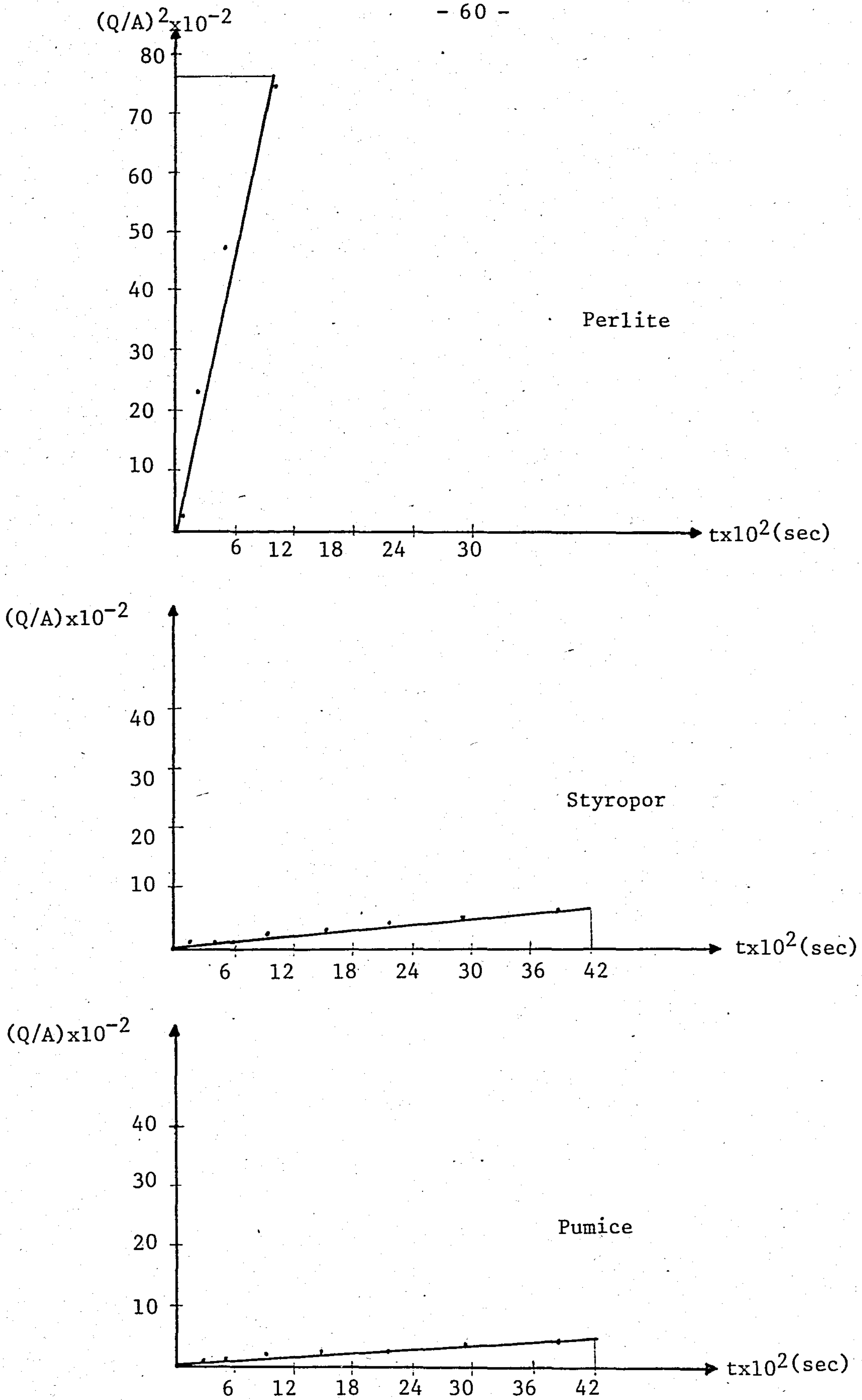

Figure 11- Typical capillarity curves for the perlite, styropor and pumice concrete slabs produced.

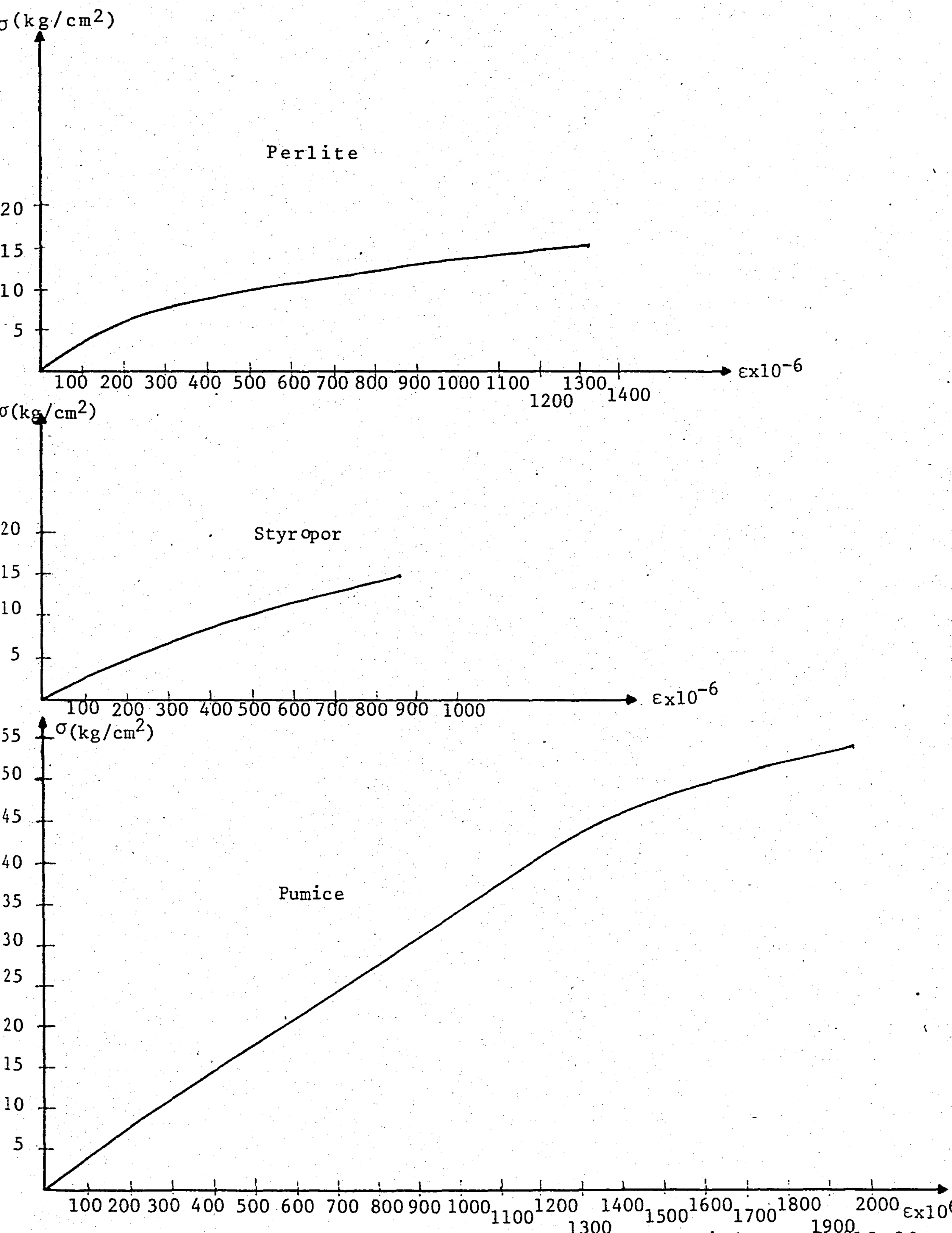

Figure 12- Typical σ-ε curves for the materials used (15x30 cm cylinders)

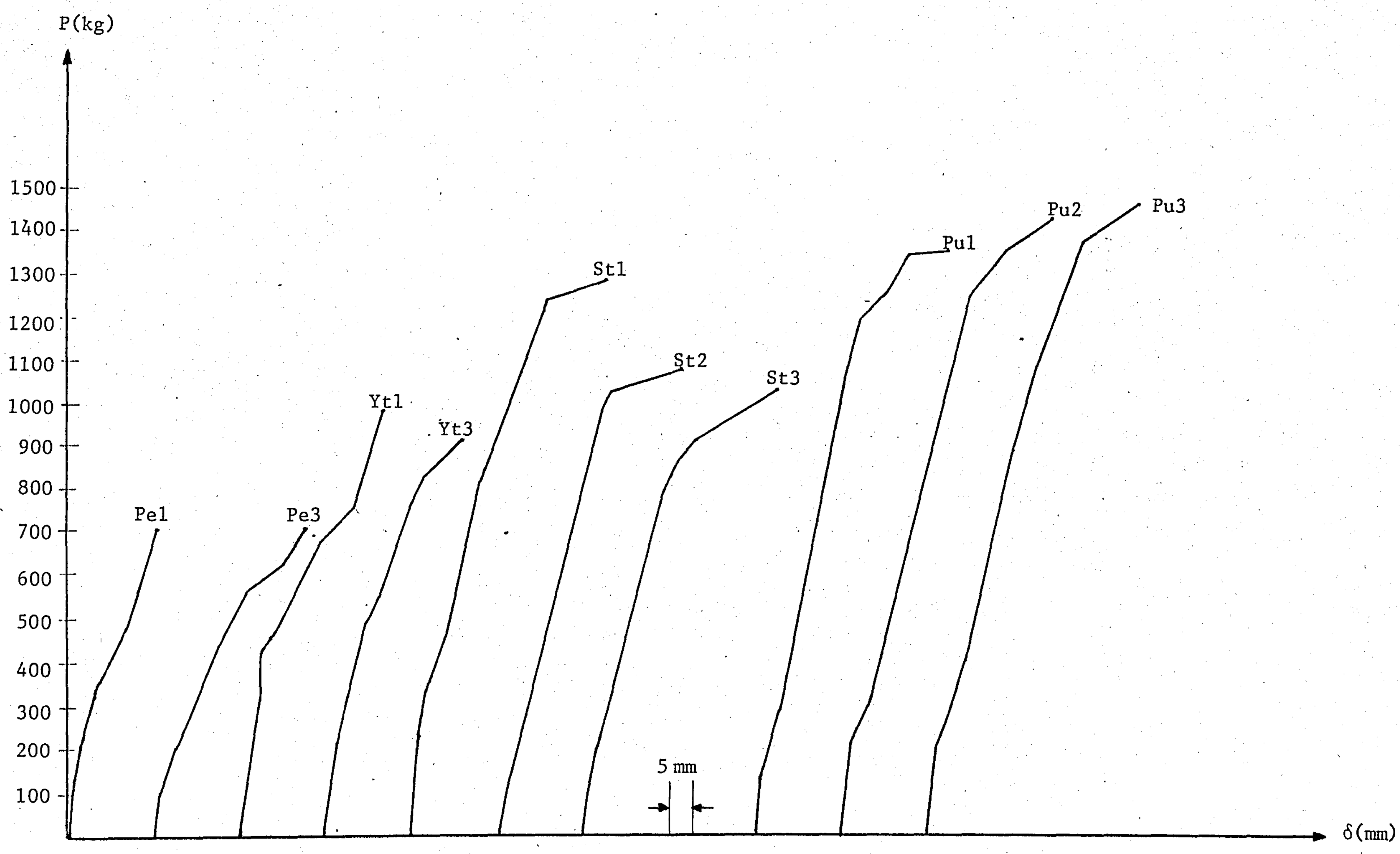

Figure 13- Mid-span deflections of the test slabs.

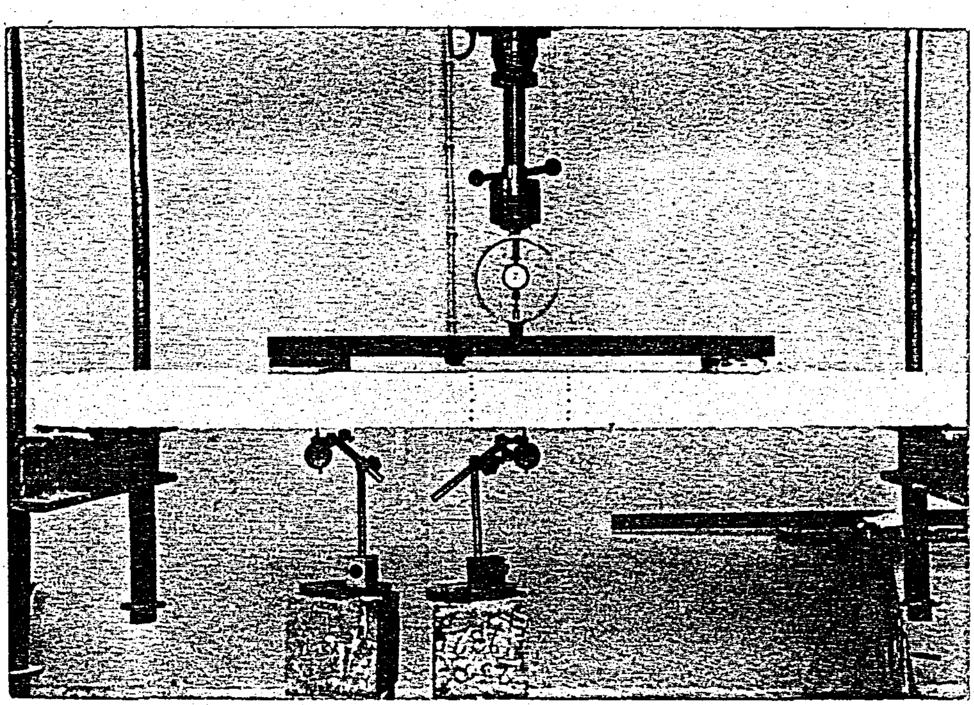

Figure 14- Set-up for the flexural tests of the slabs.

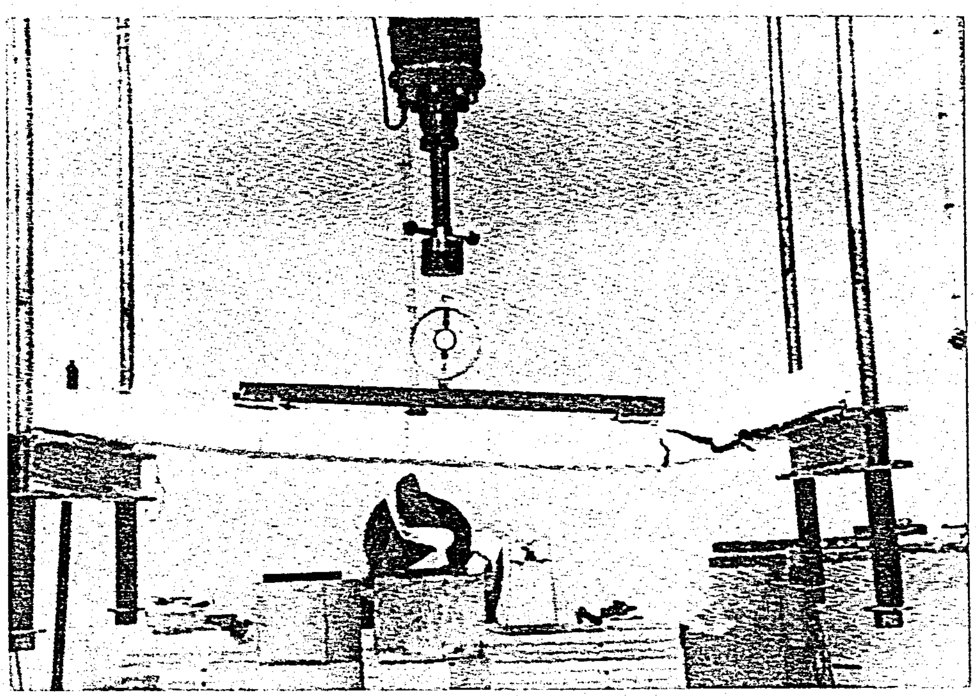

Figure 15- Shear failure of Pe 1

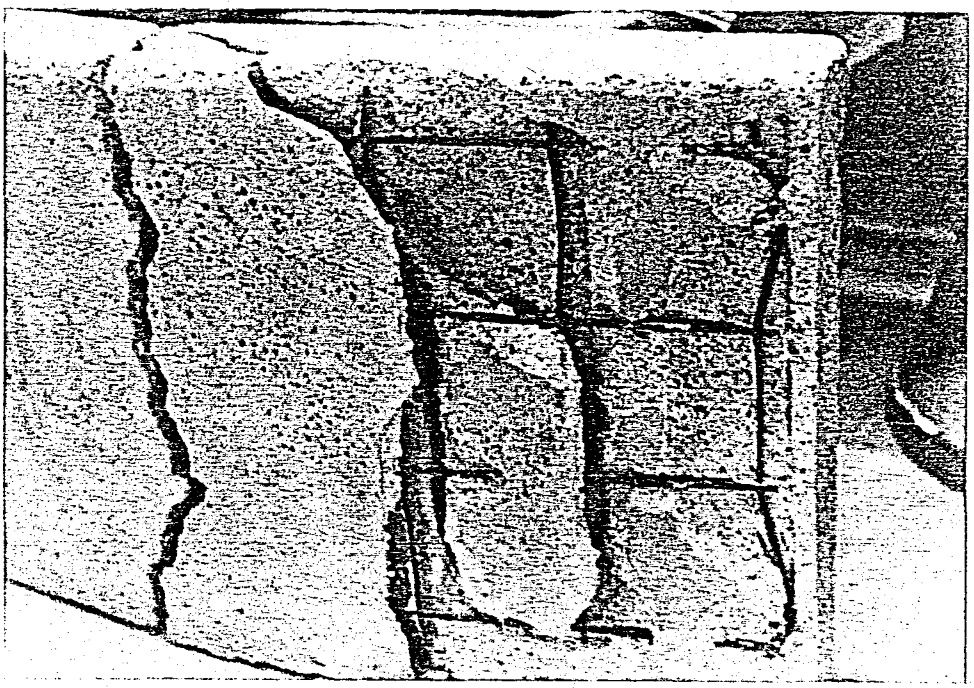

Figure 16- Anchorage failure of Pe 1.

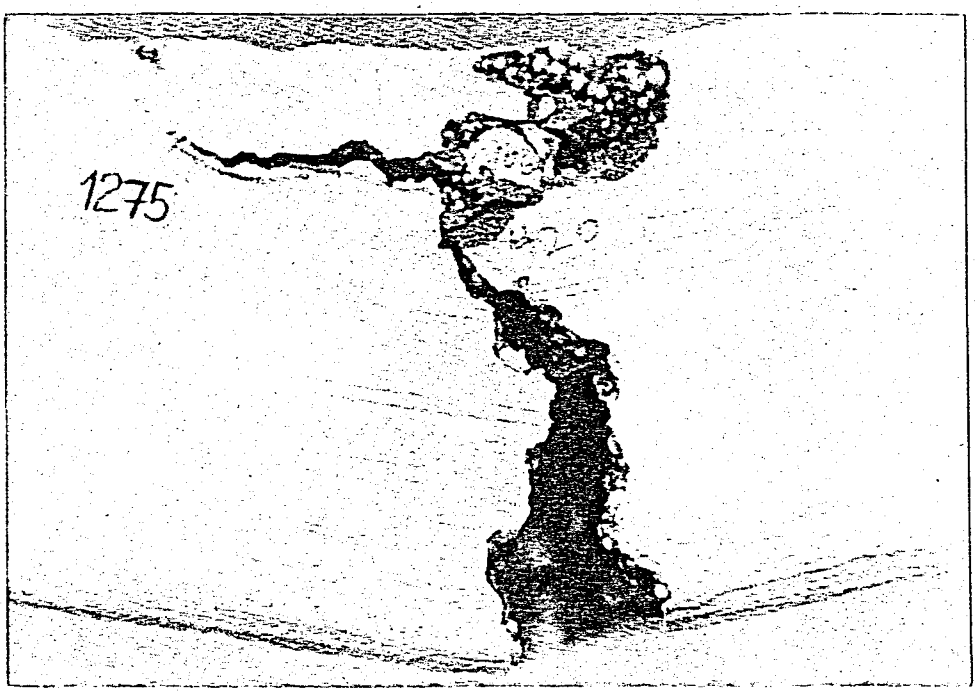

Figure 17- Flexural failure of St 1 under the applied load.

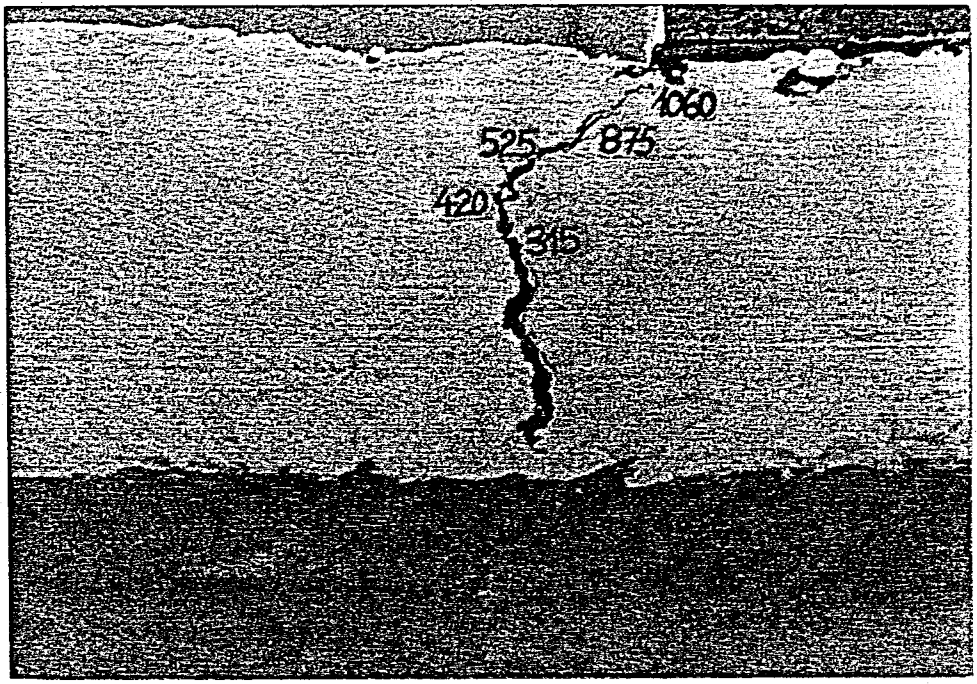

Figure 18- Flexural failure of St 2 (Note that the crack gap is almost closed after the load is removed due to the high ductility of the material)

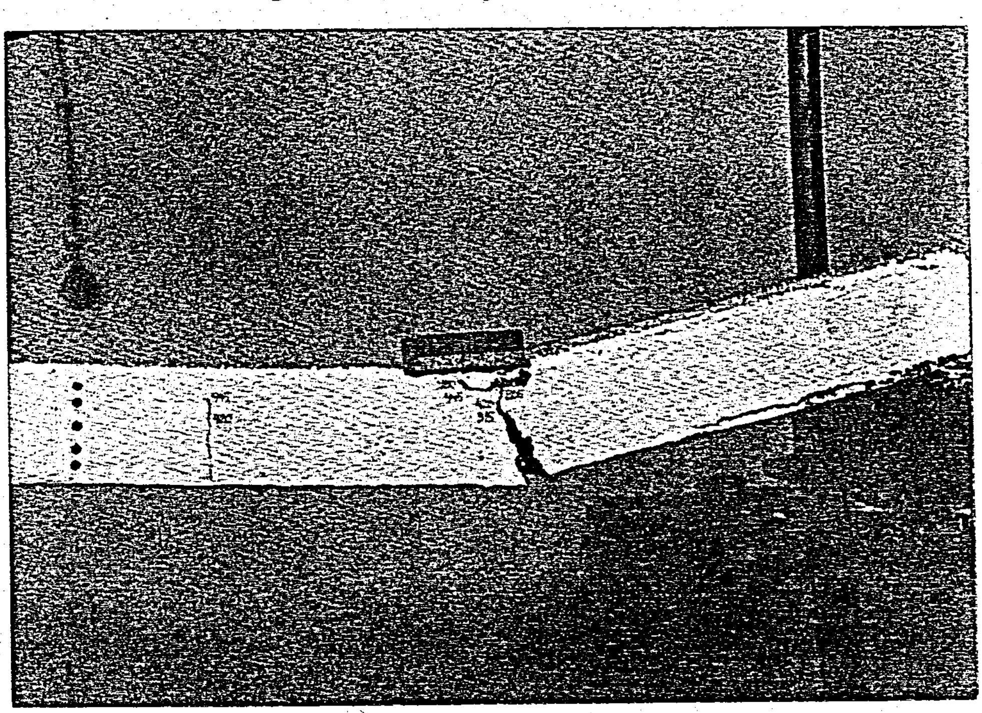

Figure 19- Flexural and anchorage failure of St 3.

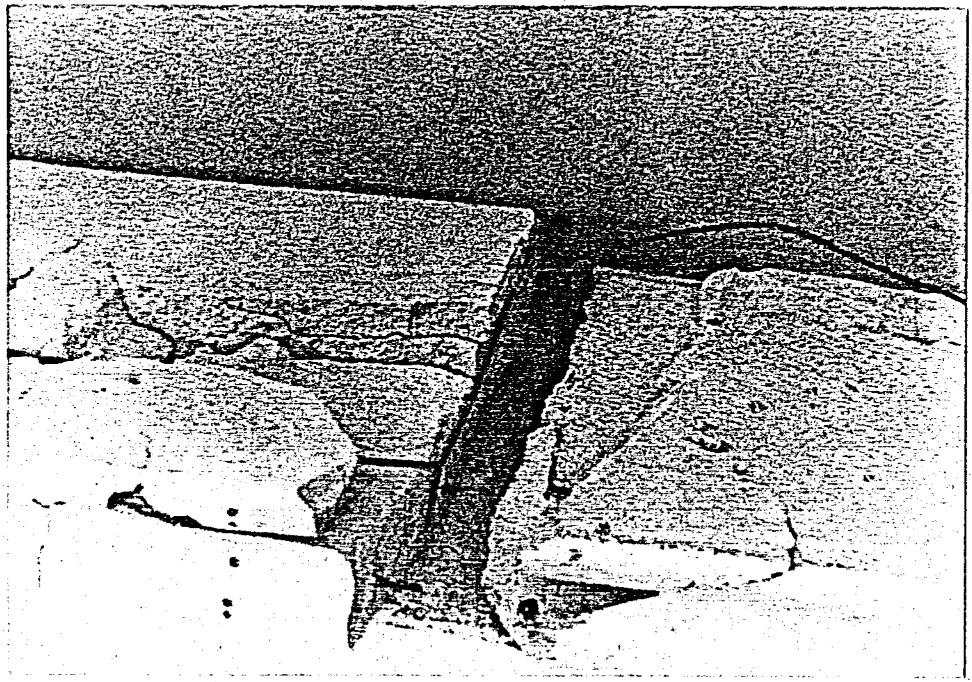

Figure 20- Flexural failure of Yt 1. All the steel failed at the same section in a brittle way since the total strain of the steel was very low and the slab was highly under-reinforced.

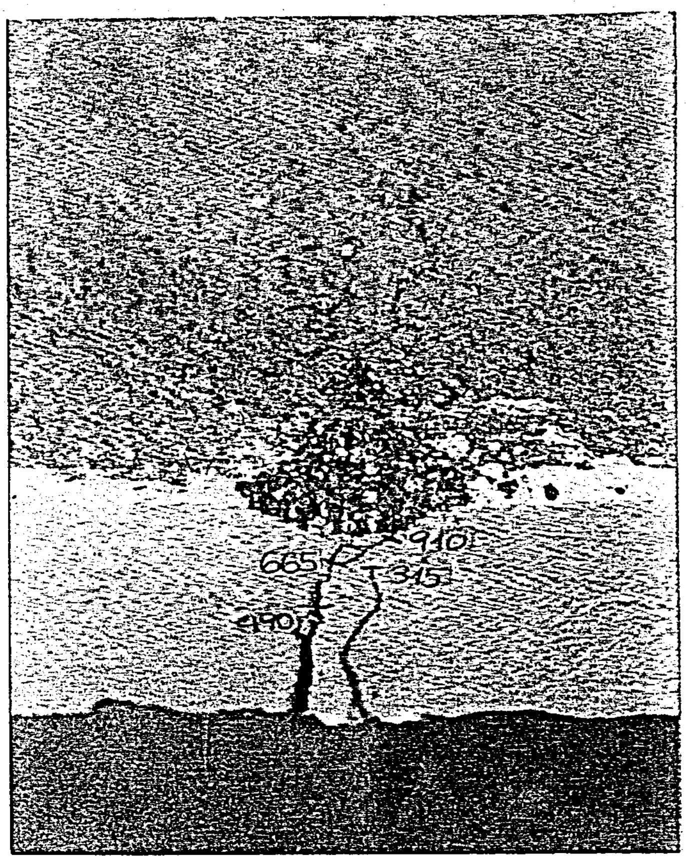

Figure 21- Flexural failure of Pu 1. The failure was balanced and ductile with the successive yielding of steel and crushing of the concrete.

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