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Masonry properties for assessing arch bridges

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1. INTRODUCTION.

In assessing the strength of masonry arches, for example by the mechanism method, it is a common assumption that the material from which the arch is built is rigid and of infinite compressive strength. Observation of the mode of failure of arch bridge structures, however, suggests that this assumption may not always be valid and that strength checks may be necessary. This will require not only information from compression tests under axial load but also from tests under eccentric or concentrated loading simulating the condition at a hinge. Since assessment procedures may depend on estimation of masonry strength from small samples or cores, means of relating the results of compression tests on such samples to the masonry of the bridge are also needed.

Analysis by finite elements and certain other methods requires knowledge of elastic moduli and the stress-strain relationship for different types of masonry.

The strength of multi-ring brickwork arches may be limited by separation of the rings as a result of internal shearing forces and thus a knowledge of the shear strength of this type of masonry may be required.

The objectives of this paper are thus (i) to discuss the factors affecting the compressive strength of masonry which are relevant to arch structures (ii)to review available information from the literature and from codes of practice relating to the estimation of compressive strength and stress-strain relationship and (iii) to make practical recommendations arising from the above.

2. STRENGTH OF MASONRY IN COMPRESSION.

2.1 GENERAL.

The masonry in an arch vault may be either natural stone or brickwork. Natural stone masonry may take the form of carefully shaped voussoirs with thin joints, rubble with squared blocks laid in courses or roughly shaped blocks set in thick mortar joints. Brickwork arches consist of a number of rings in which the bricks are laid on edge with mortar between them and between the rings. The mortar joints will be of the order of 10mm as in other forms of brickwork construction. As will be shown, the joint thickness is an important factor and in the case of brickwork arches, the mortar joints between the rings constitute potential surfaces of weakness.

2.2 FACTORS AFFECTING COMPRESSIVE STRENGTH OF MASONRY.

The following factors are of importance in determining the strength of masonry in an arch structure:

Masonry unit:	Strength of material Shape of unit
Mortar :	Strength Deformation characteristics relative to those of masonry unit Thickness of joint relative to thickness of unit
Masonry :	Type e.g. ashlar, rubble etc. Loading condition e.g. eccentric or concentrated.

2.2.1 DEFINITION OF UNIT STRENGTH.

Considering these factors, the first problem to be encountered is the definition of the unit strength as the result obtained in a compression test will depend on the test conditions and on the shape of the unit. The test conditions specifed for masonry units of different materials are not the same and also different apparent strengths would be obtained for the same material depending on the ratio of height to thickness of the specimen. Thus in BS. 5628 Part 1 [1] there is a set of tables relating masonry strength to unit and mortar strength for different unit sizes and materials. The draft Eurocode EC.6 [2] attempts to overcome this difficulty by the use of a formula in which the unit strength is standardised with reference to a unit 200 x 200mm, height x thickness. This is achieved by the introduction of a "shape factor", δ , which is given with the formula in Appendix I. Although the scope of this code excludes bridges and there has been difficulty in arriving at generally valid constants and indices in the formula there is clearly great advantage in having such a formula which would be applicable to the masonry in arch bridges.

2.2.2 EFFECT OF JOINT THICKNESS.

The EC.6 formula has been devised in relation to masonry used in the walls of buildings and does not take into account the effect of joint thickness, at least as a specific variable. This may be satisfactory in the case of walls since the numerical constants have been chosen to agree with wall test results but in the case of stone masonry arches joint thickness may vary considerably. Several theories have been developed which together with experimental results give a quantitative indication of the importance of this particular factor. Such theories are based either on consideration of the deformation characteristics of the component materials [3,4] or their strength under multi-axial stress [5 -7]. The deformation theories are based on the assumption that the masonry unit will be caused to fail in tension resulting from the restraint which it offers to the lateral deformation of the mortar. In general, mortar has lower strength and higher lateral deformation under compressive stress than the unit. Although the deformation characteristics of mortar approaching failure are not accurately known, this theory is sufficient to show that the relative thickness of the mortar joint is an important parameter. The strength theory is possibly more reliable, at least for the materials for which the relevant properties are known. Thus fig.1 shows the effect of relative joint thickness on masonry prism strength as calculated by a formula derived by Ohler [7]. The thickness ratio of 0.15 represents 10mm thick joints in brickwork with 65mm thick units whilst 0.01 would be the ratio for thin (5mm) jointed stone masonry with 500mm thick units. There is a ratio of 1.6:1 between the strengths of the thin and thick jointed masonry for the same materials. (It should be noted here that fig.1 refers to prism strength which is generally considerably higher than "masonry strength").

2.2.3 TYPE OF MASONRY.

The strength of masonry clearly depends also on the type of masonry - thin jointed ashlar built with large blocks will obviously be very different in strength from rubble masonry using the same materials. In the former case formulae, such as in EC.6, should be valid but the variety of possible forms of rubble masonry, ranging from roughly squared blocks laid in courses to random stones set in thick mortar beds, will make it difficult to offer general values. A theoretical treatment has nevertheless been advanced by Mann [8] which explores the influence of joint thickness and other variables. However, it concludes that stone strength has little influence on masonry strength since the mortar joint will always fail first. The theory may therefore be useful in estimating initial cracking in the joints but experiments have shown that rubble masonry will carry loads far in excess of this.

2.2.4 ECCENTRIC OR CONCENTRATED LOADING

In standard compression tests the loading is applied to the specimen axially whereas in an arch the loading will in general be eccentric. Approaching failure, the loading on some units will indeed be highly eccentric and in the vicinity of a hinge point will be concentrated on a small part of the area of the unit. There is evidence that the compressive stress at failure under eccentric loading is greater than under axial loading and it is well established that the apparent compressive strength under highly concentrated loading is considerably enhanced.

3. AVAILABLE INFORMATION ON COMPRESSIVE STRENGTH

Extensive test data exists on the compressive strength of brick- and blockwork and to a lesser extent for natural stone walls, some of which is relevant to masonry bridges. Such data is incorporated in codes of practice. Although these are intended for the design of new structures, they can, with discretion, be applied in assessment, provided the properties of the component materials can be established.

BS.5628 Part 1 contains the following clauses:

> " 23.1.8 <u>Natural stone masonry.</u> Natural stone masonry should be designed on the basis of solid concrete blocks of an equivalent compressive strength. Where masonry is constructed from large, carefully shaped pieces with relatively thin joints, its load bearing capacity is more closely related to to the intrinsic strength of the stone than is the case where small structural units are used. Design stresses in excess of those obtained from this code may be allowed in such massive stone masonry, provided that the designer is satisfied that the properties of the stone warrant such an increase.
> 23.1.9<u>Random rubble masonry.</u>The characteristic strength of random rubble masonry may be taken as 75% of the corresponding strength of natural stone massonry built with similar materials. In the case of rubble masonry built with lime mortar, the characteristic strength may be taken as one-half of that for masonry in mortar designation (iv)."

Curves for the compressive strength of brick and stone masonry are given in Department of Transport Departmental

Standard BD 21/84 [9] and in Scottish Development Department Technical Memorandum SB3/84[10] These are shown in fig.2.

A series of tests on rubble masonry piers, sponsored by TRRL and the Scottish Development Department, were carried out at Edinburgh University with the results summarised in Table 1. In fig.3 these results are compared with the corresponding curves from BS.5628, EC.6 and the D.o.T. publication. It would appear that all three sources give reasonably good estimates of the strength of this type of masonry. It should be noted, however, that in the comparison with BS.5628, Table2(a) of the code has been used. This table is intended for brickwork rather than blockwork but the stones used in the test piers were of approximately the same proportions as bricks and the both the unit proportions and strength range in Table 2(d) for solid concrete blocks were unsuitable.

No experimental data appears to exist for the other types of masonry mentioned above, but it would seem reasonable to assume that ashlar masonry is at least 33% stronger than squared rubble. This is in accordance with BS.5628 and is consistent with the calculated results shown in fig.1 for the effect of joint thickness.

The EC.6 formula, using the constants given in Appendix 1 and appropriate values of f_b and f_m, results in a curve which is somewhat less conservative than that given by either the D.o.T. code or BS.5628 but in good agreement with test results up to unit strengths of 100N/mm2.

TABLE 1.

Tests on Rubble Masonry Piers.

Pier no.	Type of stone	Crushing strength (N/mm^2)	Mortar mix and strength (N/mm^2)	Stress at first crack (N/mm^2)	Failure stress (N/mm^2)
First series					
1.1	Whinstone	167.6		6.16	9.86
1.2	Limestone	31.0		3.58	4.88
1.3	Sandstone	38.6	1:2:9	2.77	4.24
1.4	Sandstone	46.8	(2.57)	6.30	6.93
1.5	Granite	130.6		4.94	10.91
1.6	Granite	130.6		5.81	12.32
Second series					
2.1	Sandstone	49	1:2:9	2.77	5.96
2.2	in all tests.	49	1:2:9	3.48	6.38
2.3		49	1:3:12 (1.25)	5.09	7.14
2.4		49	1:3:12 (1.30)	3.96	7.07
2.5		35	1:3:12 (0.95)	2.50	4.09
2.6		35	1:3:12 (1.40)	2.94	4.36
2.7		58	1:2:9 (1.4)	4.37	11.12
2.8		87	1:2:9 (1.18)	3.36	>11.8
2.9		83	1:2:9	4.35	10.14
2.10		83	1:2:9	7.15	10.98
2.11		65	1:2:9 (2.0)	4.95	10.22
2.12		65	1:2:9 (2.1)	5.51	11.16

Piers built in laboratory
Average size of piers 775 x 410 x 930 (*w* x *t* x *h*) mm
No. of courses 5
Average stone size 360 x 200 x 160 mm
Joint thickness 20 mm

It may be concluded therefore that, if the unit and mortar strengths are known, a reasonable estimate can be made of the masonry strength under axial compression.

4. ADJUSTMENT FOR ECCENTRIC OR CONCENTRATED LOADING

It has been found that the apparent ultimate compressive stress under eccentric loading calculated on the basis of a linear stress distribution is consistently higher than that under axial load [11-13] This "strain gradient" effect appears to arise from the non-linear stress- strain relationship of masonry and has been quantified by several investigators. Burns in reference 12 has found that the enhancement of compressive strength increases linearly with eccentricity by a factor between 1.0 for axial loading to 1.5 or more at an eccentricity ratio of t/3. Fattal and Cattaneo [13] have given values between 1.3 and 1.4 between t/12 and t/4 falling to 1.18 at t/3.

Approaching failure, an arch will typically develop up to four hinge points at which the internal forces within its depth will be highly eccentric and also concentrated towards the edge of the masonry section. Investigation of this effect, in relation to beam bearings on masonry, has shown that under such conditions the contact stress may considerably exceed the uniaxial compressive strength of the material [14]. The enhancement factor is related to the ratio of the area of contact to the area of the section and for the loading applied at the edge of the section may be taken as:

$$R = 0.55/\ A_r^{0.33}$$

where A_r is the ratio of the loaded area to that of the section. In an arch the area of contact is somewhat indeterminate but, from observation, a ratio of 0.1 would appear to be reasonable, giving an increase in strength of the order of 20%.

An area ratio of 0.1 corresponds to an eccentricity of about 0.4t so that considering these effects together and having regard to the disparate values quoted for the strain gradient effect, it would seem justifiable to allow a 20% increase in the uniaxial masonry strength if the eccentricity of loading is greater than 0.15 and in the vicinity of a hinge point.

5. TESTS FOR COMPRESSIVE STRENGTH OF MASONRY MATERIALS.

5.1 BRICKS AND CONCRETE BLOCKS.

As suggested in Section 3 above, it is possible to estimate the compressive strength of masonry if the strength of the masonry units and the mortar are known. For bricks and concrete blocks and for mortar, standard tests are available [15 - 17] which must be used in conjunction with BS.5628. Similar standards will established in the near future for EC.6.

5.2 NATURAL STONE.

There is no British or European standard for testing stone and therefore some judgment must be exercised in dealing with this material. For the present, the American ASTM C 17U - 85 [18] could be used with these codes but with some caution in relation to the size of the specimens, which may be as small as 50.8mm diameter or least lateral dimension within the ASTM standard. If the characteristic strength tables for solid units in BS.5628 were being applied to stone masonry, the specimens on which the stone strength is based should ideally be of similar dimensions to the corresponding units. This may be possible for new construction but unlikely for the assessment of existing bridges where reliance will usually have to be placed on results from small specimens.

The effect of specimen size on apparent compressive strength has been investigated for concrete [19] and other materials [20] and is known to be fairly pronounced. This is illustrated in figs.4 and 5 for cubes and cylinders respectively. From these diagrams it is clear that a 50mm specimen would give a substantially higher result than one of side or diameter of, say, 250mm. so that in interpreting the results of such tests due allowance must be made for this effect in estimating masonry strength. If cuboid specimens not less than 90mm side can be tested, the factors to be applied in the EC.6 formula could be used to adjust the stone size to the standard 200 x 200mm.

5.3 TESTS ON CORES.

In most cases where existing bridges are being assessed, stone strength will have to be estimated from cores where again the size and proportions of the specimens are of great significance. Experience with concrete indicates that cores with a height/diameter ratio of less than 1.0

are unsatisfactory. Cores are likely to have h/d ratios between 1.0 and 2.0 and for this range BS.1881 [21] gives the following factors to correct the result to a standard ratio of 2.0:

h/d :	2.0	1.75	1.5	1.25	1.0
Correction factor:	1.0	0.98	0.96	0.94	0.92

If cores are tested, there remains the problem of estimating the equivalent unit strength. It has been demonstrated that the relationship between concrete cylinder, and thus core, strengths and cube strengths depends on the strength of the material. If it can be assumed that the same relationship applies to stone specimens the following formula, due to L'Hermite [21], could be used:

$$\text{Cylinder/cube strength} = 0.76 + 0.2 \log_{10} (f_{cu} / 19.5)$$

where f_{cu} is the cube strength in N/mm2.

In this formula the h/d ratio for the cylinder is 2.0. It is also known that the strength of cylinders with an h/d ratio of 1.0 have almost the same strength as cubes, so that there would be some advantage in testing cores with this ratio and using the result to estimate the equivalent unit strength using the EC.6 shape factor. It has to be said that this suggested procedure has not been proven experimentally but is possibly the most reliable approach at present.

The number of cores available for testing is almost certain to be too small to be statistically valid. Many stones will be non-homogeneous, some having obvious bedding planes. The only possible direction of coring may be such that the specimen can only be tested in a direction different from that in which it is stressed in the structure so that the result may not be representative. For this reason, and on account of the other uncertainties in interpreting the results of core tests, it would be prudent to regard them as at best giving a fair estimate of stone strength.

5.4 COMPRESSIVE STRENGTH OF MORTAR IN EXISTING MASONRY.

The estimation of mortar strength in existing masonry presents considerable difficulties since it will not usually be possible to recover specimens of sufficient size on which to conduct a test. Furthermore, the material is liable to be damaged in removing it from the structure. In these circumstances it is likely that some nominal value of mortar strength will have to be assumed. In old bridges where lime mortar has been used a strength of 0.5 - 1.0N/mm2 would be realistic. Fortunately, tests on stone masonry piers have shown that where weak mortars are used, there is little difference in the masonry strength [23]. This is confirmed by reference to the EC.6 formula in which the mortar strength is raised to the power 0.25, so that an increase in mortar strength from 0.5 to 1.0N/mm2 would increase the masonry strength by about 15%

If a cement mortar has been used it is unlikely to have a strength much in excess of 2.5N/mm2, corresponding to a 1:1:6 cement:lime:sand mix.

6. ESTIMATION OF MASONRY STRENGTH FROM TESTS

Reasonable estimates of masonry strength may be expected on the basis of unit and mortar strength for ashlar, squared rubble and brickwork. Estimation of the strength of masonry in which very roughly shaped stones are set in thick mortar joints is much more problematical and does not seem to have been investigated. In this and similar circumstances masonry strength may have to be determined by testing small piers or prisms. A RILEM specification for such a test will be published in the near future but is only relevant in situations where the necessary materials are available in sufficient quantity and therefore unlikely to be useful in dealing with an existing bridge.

If a test on a relatively small specimen is used it would appear that the result will be greater than that given by a larger one. This has been demonstrated for brick masonry (fig.6) and in a more limited way for rubble masonry piers and prisms. Fig.7 shows the results of tests on such specimens built from the same materials from which it was concluded that, between stone strengths of 30 to 90N/mm2, the prism strength would have to be multiplied by 0.75 to give the pier strength.

7. THE ELASTIC MODULUS AND STRESS-STRAIN CHARACTERISTICS OF MASONRY.

7.1 ELASTIC MODULUS OF MASONRY.

Many measurements of Young's modulus have been reported for brickwork and blockwork on the basis of which BS.5628

Part 2 recommends a general value of $E = 900f_k$ where f_k is the characteristic strength of the masonry. This is intended to apply to short term loading and a reduction to one half of this value is specified for long term effects. It should be noted that the characteristic strength is likely to be considerably lower than the mean strength, depending on the variability of the material strength. Thus a multiplier of 500-600 rather than 900 would be more realistic on the mean result of a limited series of tests. This is illustrated in Table 3 which gives the results of tests on specimens built from four types of brick in 1:1/4:3 mortar [24]. Fig.8 shows similar results quoted by Sahlin [25]. It is clear that at best only an approximate estimate of the elastic modulus is to be expected. Various other formulae have been suggested for the elastic modulus of brickwork [26,27] but do not appear to give any more accurate results than the foregoing.

There appear to be very few measurements of E for stone masonry. From the few that are available, from the tests quoted in Table 2, the multiplier on the masonry strength ranged from 200 to 400. As the masonry in these tests was built with thick joints of relatively weak mortar, it is not surprising that the modulus should be considerably lower than for brickwork with thin joints and strong mortar.

A method for the measurement of the elastic modulus of materials is set out in detail in ASTM E 111 - 82 [28].

TABLE 2.

Experimental Values of Elastic Moduli for Brickwork.
(Powell and Hodgkinson: reference 24)

Brick type	Brick compressive strength N/mm2	Brickwork strength f_m N/mm2	Elastic Modulus N/mm2		
			Tangent	Secant	$550.f_m$
A:16 hole perforated	69.6	19.93	18230	11900	10962
B:Class A blue engg.	71.7	27.65	17370	12930	15208
C:Fletton	25.5	9.33	4960	3740	5131
D:Double frogged stiff plastic	45.3	20.10	16830	11610	11055

Mortar - 1:1/4:3 . Mean compressive strength 15.24 N/mm2

Although this specification is in general terms, it is applicable to masonry stressed in compression.

7.2 STRESS - STRAIN RELATIONSHIP.

The stress-strain relationship for masonry may be required in applying the more sophisticated methods of analysis. This has been established for brick masonry by Powell and Hodgkinson [24] and others [29,30]. It has been shown that this relationship is closely represented by the parabola:

$$\frac{f}{f_{max}} = 2\left(\frac{\epsilon}{\epsilon'}\right) - \left(\frac{\epsilon}{\epsilon'}\right)$$

where f_{max} and ϵ' are respectively the stress and strain at the maximum point of the curve. The initial tangent modulus is given by:

$$E = 2\ f_{max}/\epsilon$$

and the secant modulus at $0.75f_{max}$ is three quarters of this value. The strain at maximum stress is about 0.003 and at failure some 50% higher.

8. SHEAR STRENGTH OF BRICK MASONRY.

Although shear strength is unlikely to be a controlling factor in the the strength of masonry arches, there is one situation where it may have to be considered. This is in brickwork arches of multi-ring construction approaching failure where sections of an arch between hinge points will be in compression, as shown in fig.9. The brickwork between the rings will thus be stressed in shear which could lead to separation and thus to a serious reduction in compressive strength. The shear strength of a brick/mortar interface may be taken from BS.5628 as 0.35N/mm2 for mortar designations (i) to (iii) or 0.15N/mm2 for mortar designation (iv). EC.6 will suggest somewhat lower values for clay brickwork but neither code gives a figure for lime mortar. Benjamin and Williams carried out tests on brick couplets [31] using two types of lime mortar and reported shear strengths over 0.3N/mm2. This suggests that lime mortars may be at least as effective as the lower strength cement mortars now in use but real evidence is lacking.

9. RECOMMENDATIONS FOR THE ASSESSMENT OF MASONRY COMPRESSIVE STRENGTH AND DEFORMATION PROPERTIES

9.1 MASONRY STRENGTH FROM CODES OF PRACTICE

Where the strength of units and mortar are known, masonry strength may be estimated on the basis of BS.5628 or, when available, EC.6.

9.2 ASSESSMENT OF UNIT STRENGTH.

Whenever possible, unit strengths should be determined by the standard test methods associated with the code of practice being used.

When small specimens of stone have to be used, they should be not less than 90mm diameter or least lateral dimension. The result should be standardised to 200 x 200mm dimensions by the use of the EC.6 δ shape factors.

Cores should be not less than 1.0 h/d ratio. If this ratio is between 1.0 and 2.0, the result should be adjusted to h/d = 1.0; this may be assumed to correspond to a cube strength and in turn standardised to 200 x 200mm to give an equivalent unit for use in the EC.6 formula.

At least five test results should be obtained and the mean strength adopted for estimating the equivalent unit strength.

Specimens should be tested in the direction in which they will be stressed in the structure. If this is not possible any indications of non-homogeniety must be noted and taken into account in assessing the strength of the material.

9.3 MORTAR STRENGTH.

It will not generally be possible to obtain sufficiently large and undamaged specimens for testing from an existing structure. No reliable in-situ test is known for mortar and therefore it will generally be necessary to use a notional strength e.g. for lime mortar 0.5 - 1.0N/mm2 would be appropriate.

9.4 MASONRY STRENGTH BY TEST.

If materials are available in sufficient quantity, test procedure should follow the RILEM specification. The masonry strength for assessment purposes may be assumed to be 0.75 of the strength so determined.

9.5 STRENGTH ENHANCEMENT FOR ECCENTRIC OR CONCENTRATED LOADING.

The strength of masonry assessed under axial loading may be increased by 20% if the eccentricity of loading exceeds 0.15t provided that the stresses are calculated on the basis of a linear stress - strain relationship.

In the vicinity of a hinge point it may be assumed that the load is concentrated over one tenth of the depth of the section at an eccentricity of 0.4t. The stress on this area may be assumed uniform and may exceed the uniaxial strength by 20%.

In checking the masonry strength at a potential hinge point on the extrados of an arch, it may be necessary to use a reduced ring depth if the mortar is observed to have deteriorated on the surface or the has been recessed by design.

9.6 STRESS-STRAIN RELATIONSHIP AND YOUNG'S MODULUS

The stress-strain relationship for masonry may be assumed parabolic with a strain at maximum stress equal to 0.003 and ultimate strain 0.0045.

The value of E for brick masonry is approximately $900f_k$ or 400 - $600f_m$ where f_k and f_m are the characteristic and mean strengths respectively. These values could also be adopted for thin jointed, coursed masonry but for rubble masonry in lime mortar or equivalent the multiplier should be 200 - 400.

9.7 SHEAR FAILURE IN MULTI-RING BRICKWORK ARCHES.

The segments of multi-ring brickwork arches between estimated hinge points at failure should be checked for possible shear failure. The assumed shear strengths may be taken as 0.35N/mm2 for mortars with an expected strength exceeding 1.5N/mm2 and 0.15N/mm2 for weaker mortars.

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31. J. R. Benjamin and H. Williams "The behaviour of One-Storey Brick Shear Walls" Journal of the Structural Division, American Society of Civil Engineers, paper 1723,ST4, 1958.

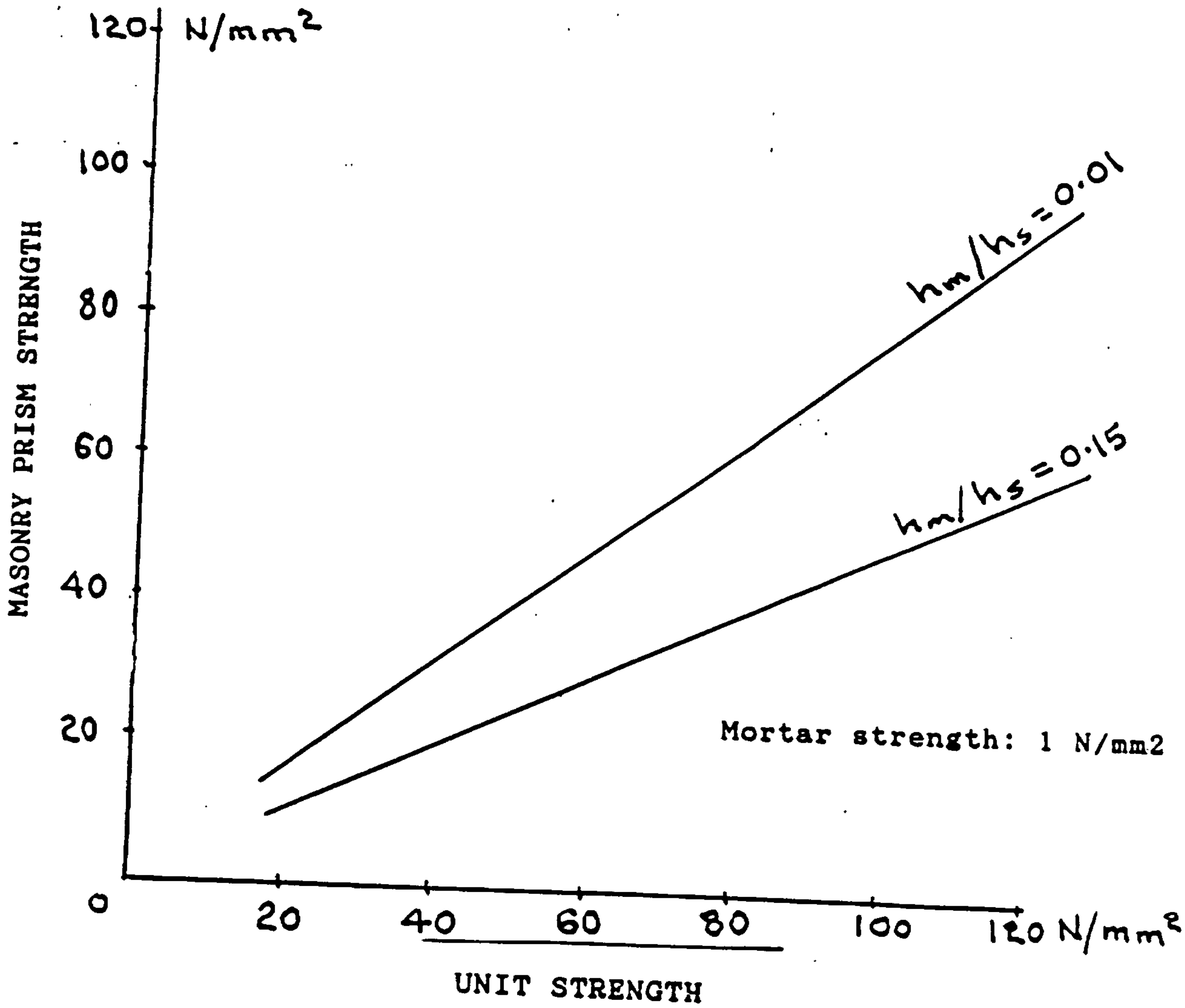

Fig. 1 : Effect of joint thickness on compressive strength of masonry prisms.

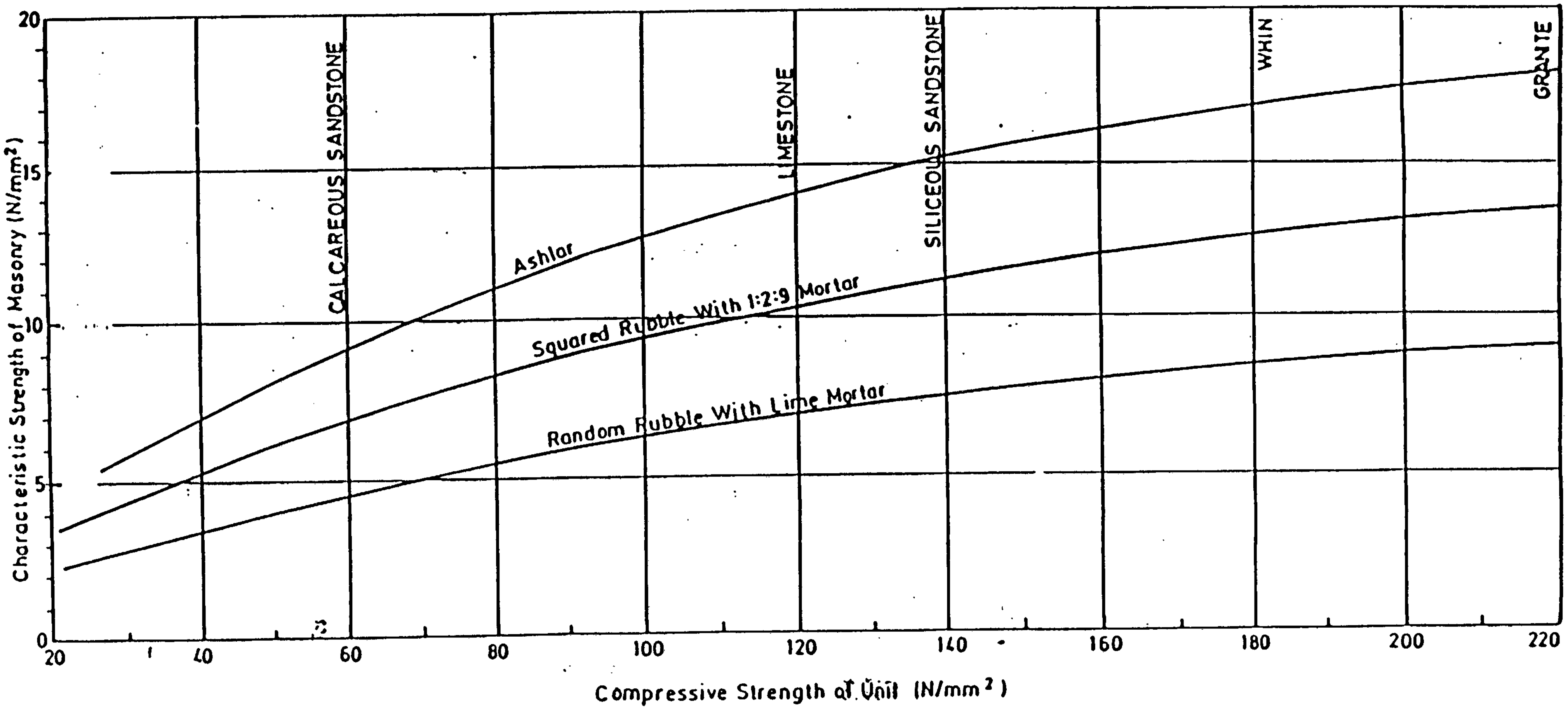

Fig. 2 : Masonry strength v. stone strength.
Dept. of Transport Departmental Standard
BD 21/84 ans S.D.D. Technical Memorandum
SB3/84.

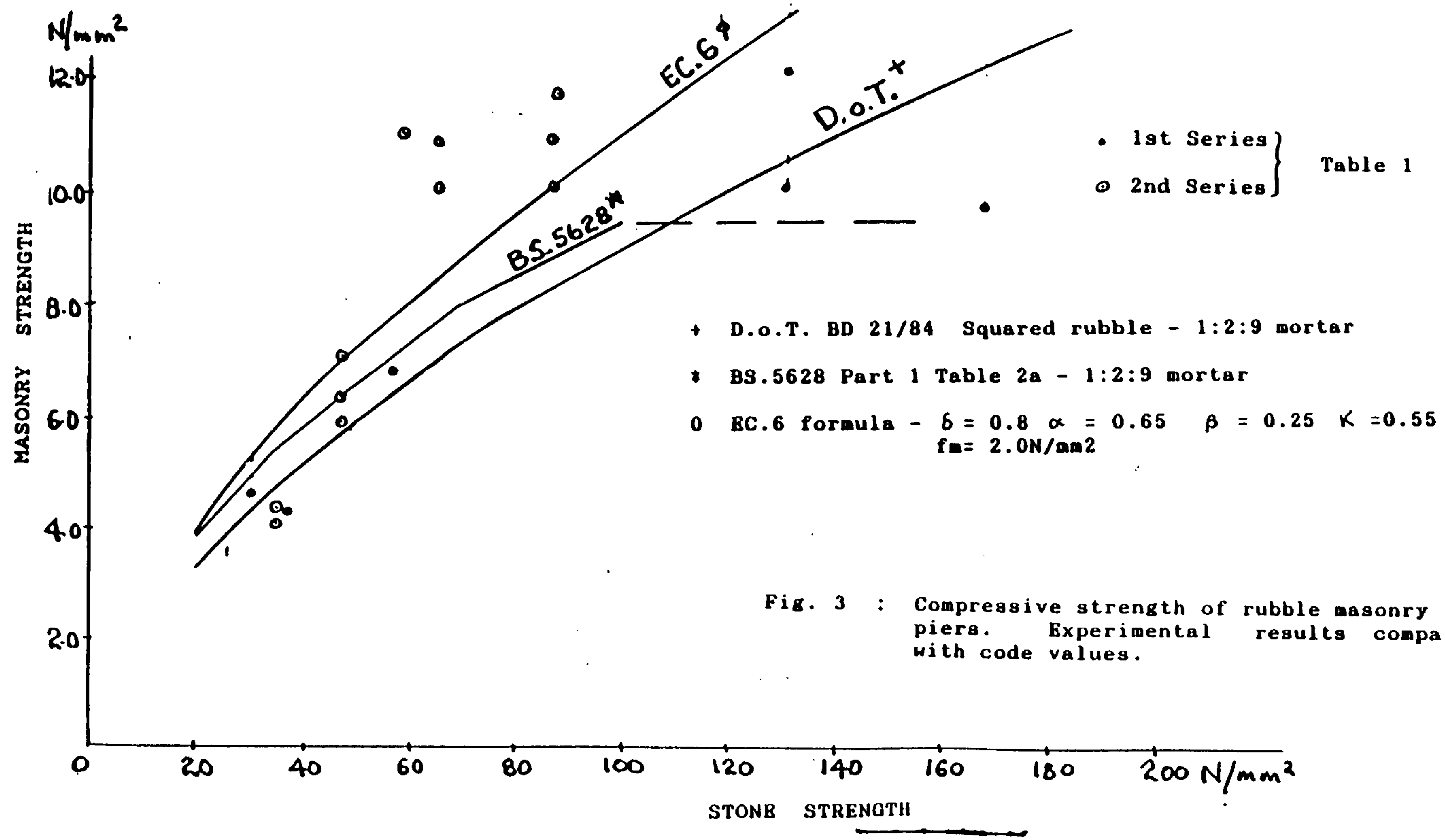

Fig. 3 : Compressive strength of rubble masonry piers. Experimental results compared with code values.

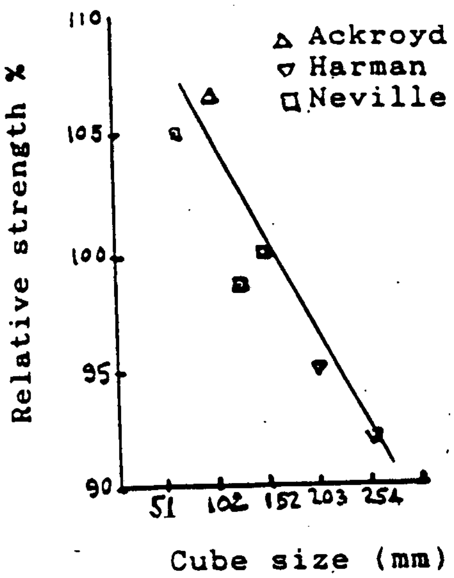

Fig. 4 : Effect of size on strength of concrete cubes.

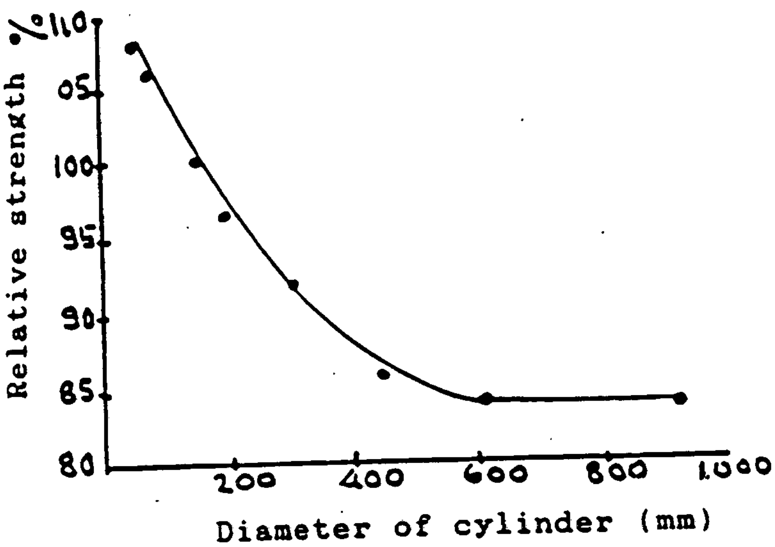

Fig. 5 : Effect of size on strength of concrete cylinders.

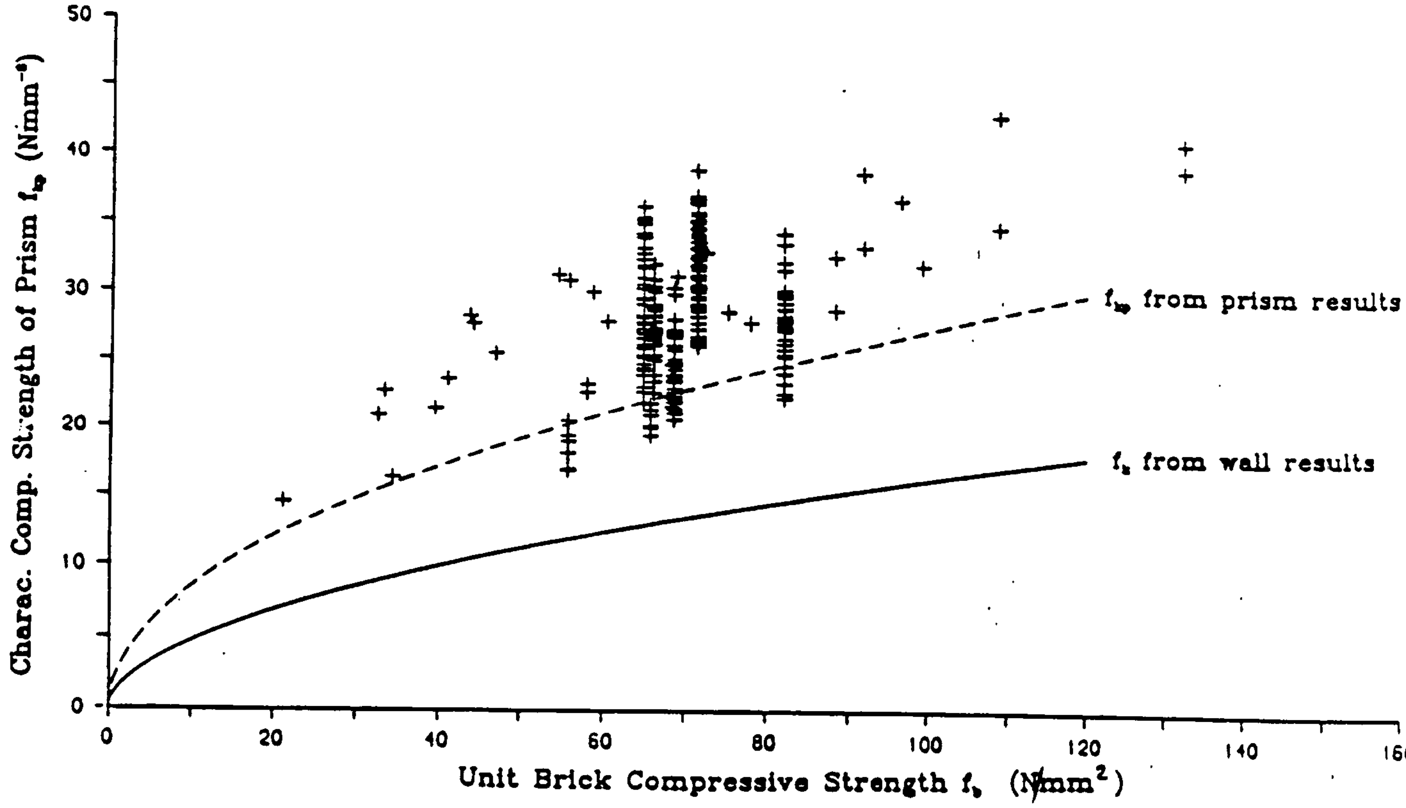

Fig. 6 : Characteristic strength of masonry from prism tests compared with strength from wall tests.

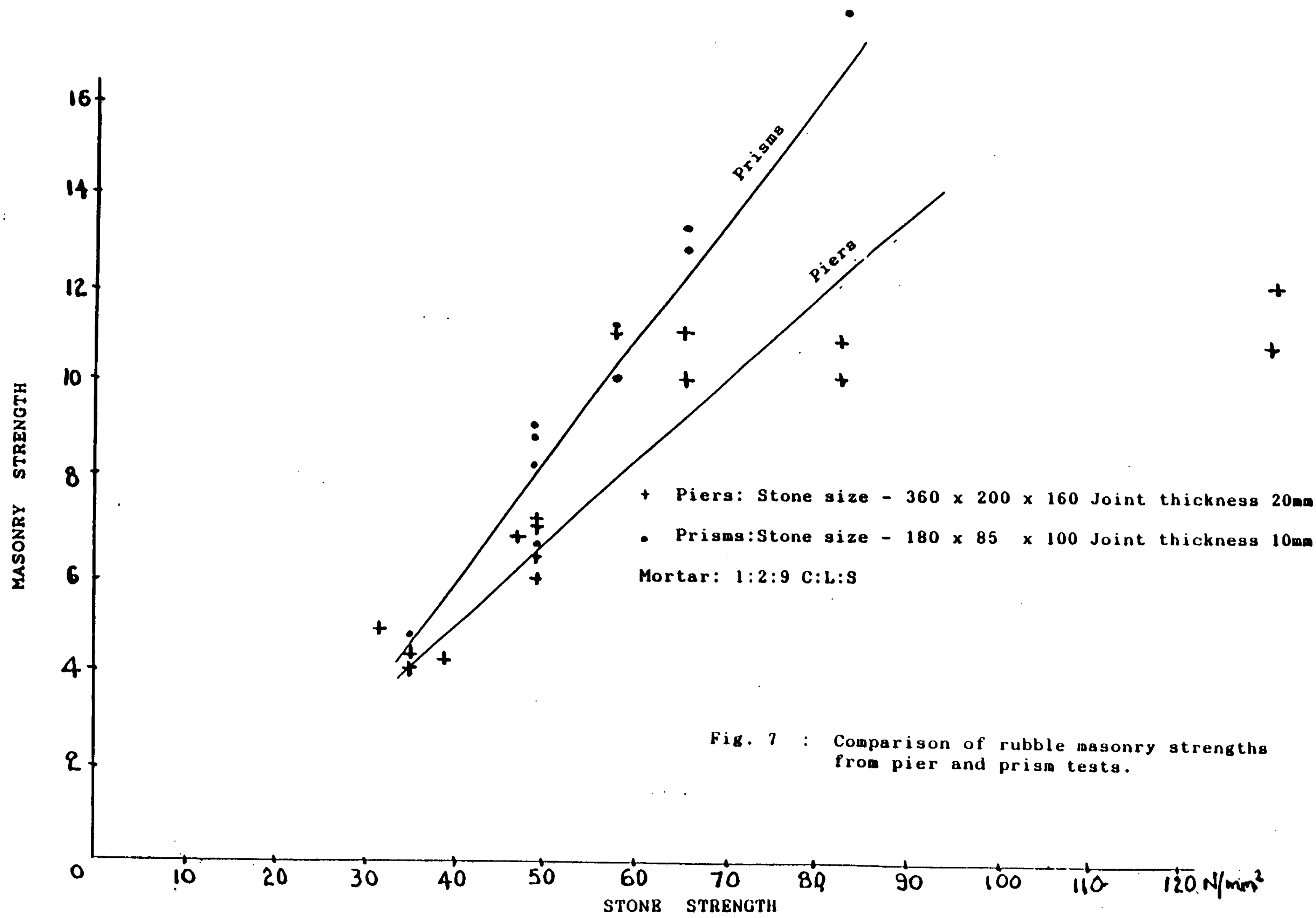

Fig. 7 : Comparison of rubble masonry strengths from pier and prism tests.

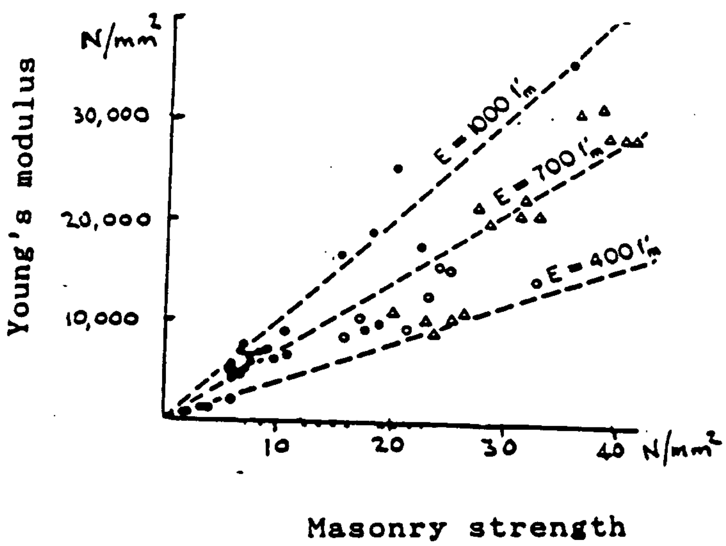

Fig. 8 : Experimental values of E for masonry (Sahlin).

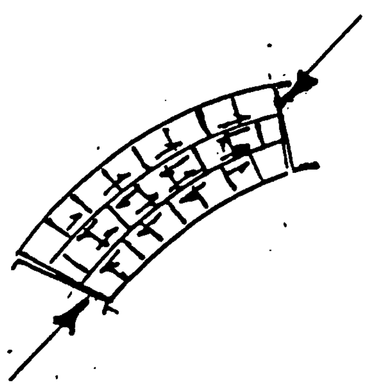

Fig. 9 : Shear in section of multi-ring brick-work arch between hinge points.

APPENDIX 1.

Eurocode 6 Formula for Determination of Compressive Strength of Masonry.

The following is an extract of the relevant clause of the draft Eurocode 6 (Reference 2):

3.2.2.3 Determination of f_k from the Compressive Strength of the Units and Mortar.

The characteristic compressive strength of masonry, f_k, may be determined from a knowledge of the compressive strength of the masonry units and from the strength of mortar used.

It can be assumed that the characteristic compressive strength of a masonry wall of thickness equal to the width of the unit will not fall below the values given by equation (3.1).

$$f_k = K \cdot f_b^{\alpha} \cdot f_m^{\beta} \tag{3.1}$$

where f_b is the mean compressive strength of the unit in the direction of the applied loading normalised to allow for the unit being dry and factored by a shape coefficient to allow for the dimensions of the unit

f_m is the mean compressive strength of the mortar

K, α and β are coefficients (see preface for suggested values)

When a masonry wall has a thickness greater than the width of the unit, such that there is a longitudinal mortar joint through part of the length of the wall (see Figure 1), the value of f_k obtained from equation (3.1) should be reduced by 15% or by an amount based on tests on the type of masonry.

No values are given in the text of paragraph 3.2.2.3 of EC.6 but some tentative figures are included in the Preface. These have not proved to be satisfactory and revised proposals are quoted by Edgell in a paper entitled The Characteristic Compressive Strength of Masonry (Masonry International, Vol.3 No.1, 1989). These are as follows for masonry built in solid units:

$$f_k = 0.65(\delta f_b)^{0.65} f_m^{0.25}$$

and where the masonry contains a joint parallel to the face:

$$f = 0.55(\delta f_b)^{0.65} f_m^{0.25}$$

where δ is a shape factor to allow for the dimensions of the masonry units and is given in Table A.1 below.

These values are still subject to revision but are likely to be reasonably accurate for thin jointed block masonry, coursed rubble masonry and brickwork arches.

TABLE A.1
Shape Factor δ.

Height	Width (mm)				
	90	100	150	200	250
50	0.70	0,65	0.60	-	-
65	0.75	0.70	0.65	0.60	0.55
100	0.90	0.85	0.80	0.70	0.65
150	1.05	1.00	0.95	0.85	0.80
200	1.20	1.15	1.10	1.00	0.90
250	1.25	1.20	1.15	1.05	1.00

APPENDIX 2.

SUGGESTIONS FOR FURTHER RESEARCH.

TESTS ON MASONRY COMPRESSIVE STRENGTH.

The recommendations made in this report for the estimation of masonry strength in the assessment of arch bridges are based on available information but, as observed at various points, only limited work has been carried out on stone masonry. There is therefore scope for further direct investigation of the compressive strength of stone masonry. Tests so far carried out have been on rubble masonry so that there is a need for some further tests on thin jointed (ashlar) masonry and on rubble masonry in which roughly shaped stones are set in thick mortar joints.

CONCENTRATED AND ECCENTRIC LOADING.

The recommendation made in relation to concentrated loading at hinge points and eccentric loading in general is based on inference from tests on other forms of masonry. Verification of these effects on stone masonry would be desireable.

CRUSHING STRENGTH OF STONE.

Standardisation of a method for determining the strength of stone is desirable. This might be requested as an appendix to BS.5390, Code of Practice for Stone Masonry, which refers to crushing strength (Para.42.Testing) but does not give a method. Such a request would, however, be more effective if accompanied by some data on the effect of specimen size and other information which would enable BSI to draft a suitable test. A similar request could be made to RILEM, Committee 127 MS, which is concerned with the development of masonry testing methods.

SHAPE FACTORS FOR STONE BLOCKS.

Work is currently in progress on shape factors for masonry units to be incorporated in EC.6 but no work has been done to confirm that these factors are applicable to natural stone. It would therefore be useful to undertake tests on different shaped blocks to obtain information on the effect of this variable on apparent stone strength. This would be done as part of the work necessary to standardise a test method suggested above.

CORE TESTS.

In view of the importance of being able to interpret the information from core tests, a suitable programme of tests on the relationship between core test results and those from block tests would be valuable. This would include consideration of the size and proportions of core specimens. As observed in the report, the only information on these matters at present comes from corresponding work on concrete.

MORTAR STRENGTH TEST.

Assessment of in-situ mortar strength presents considerable difficulties. Tassios et al* have suggested that a combination of two methods gives an accurate basis for the estimation of mortar strength. The first is a scratch width test and the second a tension test on small fragments of mortar. This may form the basis of a mortar strength test but requires evaluation and possibly further development before it could be recommended for routine use.

* T.P.Tassios, C.Vachliotis and C. Spanos "In-situ Strength Measurements of Masonry Mortars", Proc. Int. Conf. on Structural Conservation of Stone Masonry., Nat. Tech. Univ. Athens, 1989.