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Proposed design and construction guidelines for labour-intensively built rubble masonry concrete structures with particular reference to arch bridges

Rankine, Roderick Graeme Duncan

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All too frequently RMC bridges are built as part of rural road upgrading projects on old drift sites which were deliberately situated on wide sandy stretches of the river where the flow is shallow. These sites are often far above good founding material. In such cases, the construction of adequate foundations becomes very difficult and is likely to compromise the financial viability of most RMC bridge designs. Founding on unstable silty material may be dangerous, owing to the tendency for the riverbed material to become liquid, a phenomenon which has been reported to occur at depths as great as between two[8.4] and ten[8.5] metres below riverbed. While some designers[8.2] might argue that it is often not possible, or indeed economic, to found RMC bridges on bedrock, Shelton reminisces that never once was he unable to find a site which offered exposed rock right across the river as a foundation[8.3].

Where a bridge is built as a skew crossing or on a curve of a river, the designer runs the risk of his structure being damaged or made completely redundant. The skew crossing tends to channel the stream towards one of its banks, particularly when it becomes blocked by debris during floods. The chance of that abutment being damaged is greatly increased if it is not founded on bedrock. Centrifugal force of a river at a bend tends to channel the flow towards the outside bank particularly during floods, and the presence of the structure merely serves to accelerate this flow and enhance erosion. In such cases, rivers have been known to cut complete new channels around bridges, and remain permanently realigned, necessitating the construction of new structures[8.5,8.6].

In cases where river-bank gradients require significant modification, the viability of the structure is likely to be compromised. Earth fill approach embankments are a tempting shortcut but are notorious for being washed away by floods[8.4,8.5]. Any built-up approach produces appreciable obstruction to the river's flow as it acts like a weir. Therefore, built-up approaches should be treated as a continuum of the bridge structure. They should be provided with apertures to reduce their obstruction and be as structurally competent as the bridge itself. Excavated approaches "in cut" are preferable, but are liable to become choked with debris during floods, necessitating clearing afterwards.

8.2.1 Founding on rock

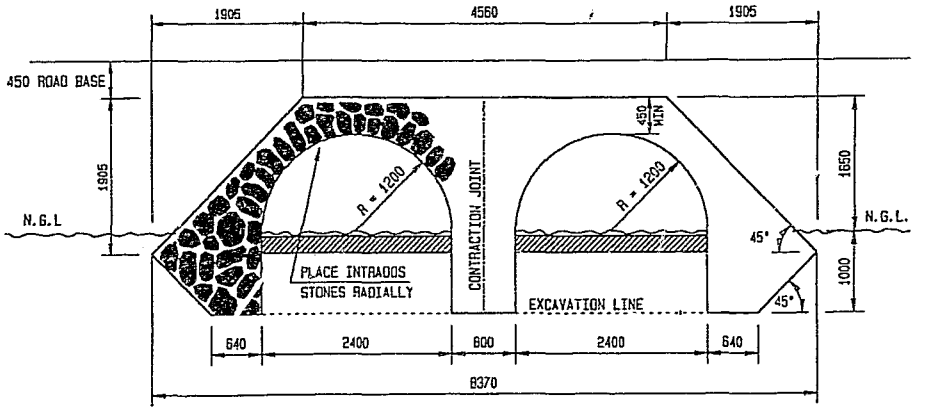
Where bedrock occurs at abutment sites, the economy of an arch bridge is likely to be excellent since the problem of providing a suitable foundation to resist not only its mass but also its thrust, has already been solved by nature. Where this rock is reasonably plane and perpendicular to the line of thrust, arches can be sprung directly from the rock surface. Where this is not the case, the casting of concrete bases to levels upon which centring (formwork) can be erected is recommended. In both cases anchorage via socketing and/or dowelling is recommended.

8.2.2 Founding on material other than rock

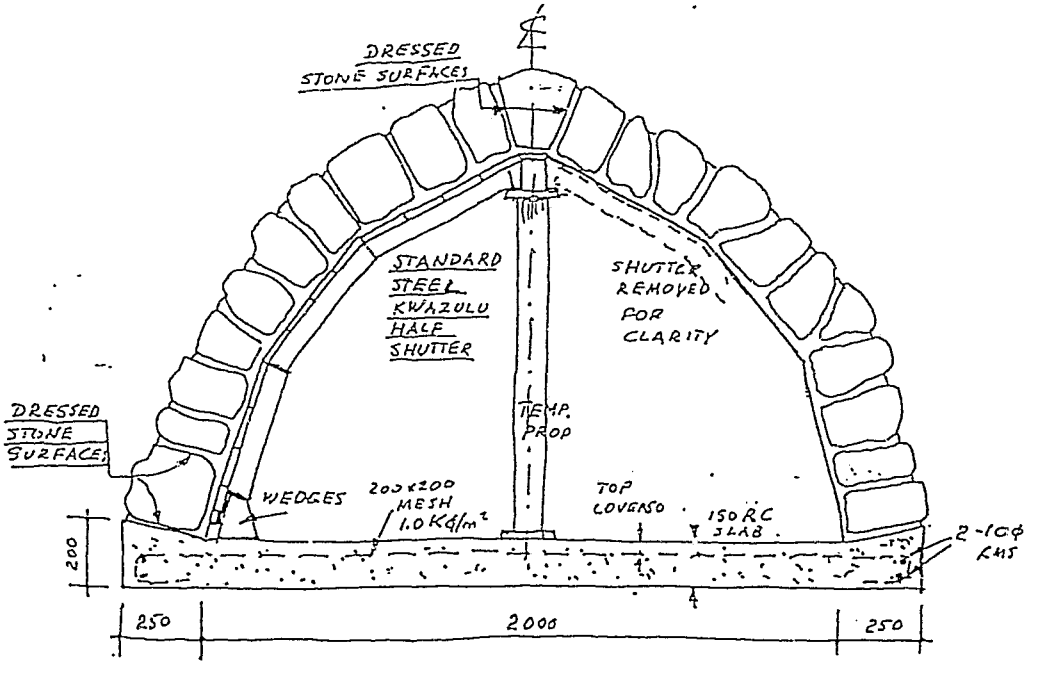
Where it is not possible to found on bedrock (due to its non-existence at an accessible depth) and the stability of the riverbed, below the foundations, can be ensured, then there will be a requirement for the use of bases, reinforced concrete raft foundations or piles. Consideration should be given to the maintenance of the horizontal reaction to arch thrust at abutments in case the structure, or indeed, the riverbed should move. Figure 8.2a proposes one practical way in which this may be achieved where a bridge is founded on bases. Kowalski[8.25] has proposed the use of a reinforced raft foundation to tie both abutments together as an alternative means of maintaining the resistance to the arch's thrust (see Figure 8.2b). Where piled foundations are used, some raked piles inclined sub-parallel to the line of thrust may achieve the same result. Resistance to transient loading may also be achieved by the use of raked piles and/or by increasing the width of the structure. Bases should be designed to minimise the danger of undermining by scouring during floods, and also to provide the required bearing area, so as not to exceed the bearing strength of the river bed. To guard against the scouring action of the foundations, scour protection from one abutment to the other, terminating at cut-off walls on the leading and trailing edges is required[8.2]. The cut-off walls, particularly on the downstream side, must be built to a depth below which stability of the riverbed material is assured; and no less than about two metres. In many rural areas, such as the Northern Province in South Africa, gabions are deliberately not used as scour protection because the wire is frequently stolen. RMC, rip rap or grouted stone pitching has been preferred instead.

8.2.3 Founding partly on rock

Although bridges with alternate abutments founded on rock and other material respectively do exist, the practice is not recommended. A 3,7 m radius arch bridge, in the Sekhukhune area in the Southern District of the Northern Province of South Africa, founded partly on rock and partly on clay, was reported to have been severely damaged during a flood in 1995[8.7]. The abutment founded on clay is understood to have been undermined.



a.



b.

Fig 8.2 a. A proposed arch bridge design for founding on soils. The inclined foundation abutments are intended to maintain the reactions to horizontal thrust in the event of subsidence. Wing walls and scour protection have been omitted for clarity. **b.** Kowalski's[8.25] proposed reinforced raft foundation used to maintain the thrust equilibrium reaction.

8.3 FORCES TO BE RESISTED

Arch bridges and their foundations need to withstand transient water loads during floods, self-weight, traffic loads, internally generated arch thrust, and possibly load redistributions caused by foundation distortions.

8.3.1 Transient loading

Bridges with a small aperture fraction (typical of RMC arch bridges) produce an appreciable obstruction to a river's flow, frequently aggravated by driftwood blocking the flow through the apertures, effectively forming a weir. Therefore, unless the bridge is designed as a high-level structure capable of comfortably accommodating the most severe flood-waters, overtopping must be considered as an important loading condition. Overtopping causes significant lateral load, particularly when the apertures are blocked, possibly leading to overturning of the structure, as demonstrated by the collapse of an arch bridge after very heavy local rain reported by Konishi[8.8]. In the event of uncertainty as to the presence of hydrostatic uplift beneath the structure, it is prudent to assume that the entire structure is buoyed-up. Therefore, the overturning moment must be considered in combination with buoyed-up 'density' and blocked apertures but not with traffic loads. Thus, the required moment, to restore stability against overturning, may be calculated by taking moments about the base of the downstream face; assuming the buoyed-up 'density' of masonry to be 1000 kg/m^3 less than the true density of the masonry. Tall single lane bridges are most likely to require anchorage to resist this eventuality. Data concerning actual values of flood-water pressures may be sought from other sources[8.8] or from an assessment of water depth provided by the Department of Transport Guidelines[8.9].

8.3.2 Traffic loading

An assessment of appropriate standards for rural roads[8.10] has considered the traffic that is likely to use the rural roads for which most RMC bridges are designed. The recommendation is made that NA and NB24 vehicle loads, as defined by TMH 7[8.11] (1981), should be used. The maximum vehicle loads that the bridges are required to withstand are thus 16 wheel loads of 60 kN each, consisting of four rows in two pairs. Each row of wheels is separated by a minimum of 2 m, and each row has four wheels spaced 1 m apart across the bridge. The wheel contact area may be defined by a square with a 240 mm side. If more ambitious structural designs are attempted, for example, bridges to carry traffic on national roads, then NB36 loading would appear to be appropriate (where the maximum wheel load is 90 kN and the wheel contact area may be defined by a square with a 300 mm side). Where a structure is to be covered with fill and/or a sub-base, the compaction equipment used may subject the structure to a more severe point load; particularly if a steel roller acts directly on the rough rubble finish.

8.4 GEOMETRICAL CONSIDERATIONS

Thom[8.26] has asserted that: *"A bridge structure should either be a high-level structure, capable of passing a significant design flood, or a low-level stream crossing, designed for overtopping. A structure placed intermediate in relation to these two options forms a barrier to flood-water and greatly increases the risk of damage with cost related implications."* Every effort should be made to minimise the bridge's obstruction to the flowing river, particularly when it is in flood. Large apertures are less likely to become blocked and require maintenance. Therefore, the arch designer should always aim to make apertures as big as possible. Shelton[8.3] recommends a minimum span of at least five metres over any river worthy of its name. Zimbabweans[8.4,8.18] have found that inclining the upstream elevation of the bridge, with a wedge of additional masonry, assists in lifting debris over the top of the structure and clear of its openings. dos Santos[8.19] proposes a catenary bridge deck (lowest at the centre of the river) to encourage overtopping of the structure at midstream to prevent scour from eroding the approaches and abutments. Kerbs and guide-blocks should be kept discontinuous to decrease obstruction to debris and flood water[8.19].

8.5 METHODS OF STRUCTURAL ANALYSIS

Although there appear to be no reported incidents of live-load-induced RMC bridge failures, their reliable analysis is prerequisite for achieving efficient designs of more ambitious structures in future. Numerous authorities including Tellett[8.12], Hendry[8.13], Page[8.14] and de Bruin[8.15] have reviewed a variety of methods of structural analysis of arch bridges, both simple and complex, with which arch bridge designers ought to be acquainted. Traditional design philosophy assumes that arch masonry possesses good compressive strength but zero tensile strength and aims to prevent the formation of tension within the masonry by limiting the resultant line of thrust to lie within the middle third of the arch ring. This is easily achieved in arches which carry either a pure uniformly distributed load or a pure point load. In the former case, for example, where the arch is only required to support its own weight, it would always meet these conditions provided it approximated the shape of an inverted catenary. The inverted catenary arch perfectly traces its line of self-weight-induced thrust; just as a chain sags reciprocally in perfect tension. Where the arch has only to support a point load, it would satisfy these conditions if it had a triangular shape whose apex coincided with the load. However, upon the application of point loads to the catenary arch, tensions quickly develop . Where the point load is static, such tensions can easily be alleviated by distorting the catenary arch shape towards that of a triangle. However, when the point load may be moving, as is usually the case in bridges, the solution is more complex since the arch ring must be capable of accommodating each unique shape of line of thrust as the load moves across its deck. In practice, this is achieved by thickening the arch ring and/or by increasing the dead-weight of

the structure itself and/or by tolerating a small amount of tension within the arch. It is generally assumed that the material is infinitely strong in compression and that, provided the reaction to thrust is maintained, a failure condition is reached when the line of thrust reaches the outer faces of the masonry at no fewer than four points, converting the structure into a kinematic mechanism by the formation of hinge points where the line of thrust alternately coincides with the intrados and extrados of the arch. Three possible kinematic mechanisms are illustrated in Figure 8.3 for elementary point loads, from which it can be seen that hinge points are always formed on alternate sides of the arch. The diagrams show the lines of thrust originating from the point loads as straight lines. This is not only conservative but unrealistic since the line can only be straight if the structure itself is weightless. The true line of thrust is defined by the path followed by the resultant of all the forces acting on the arch across its span, including self-weight and external actions. Hence, the true lines of thrust will be bent favourably away from the intrados. However, the heavier the point load in relation to the weight of the structure, the straighter the line of thrust will become. Determination of the position of this true line of thrust may be obtained by calculation[8.16] or by a graphical method[8.17] of analysis. Figure 8.4 shows advanced stages of collapse in two model arches.

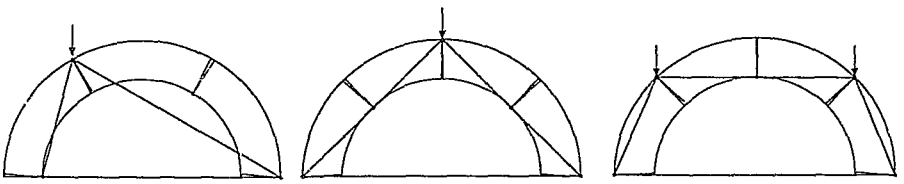


Fig 8.3 Possible arch ring hinge failure mechanisms that may occur provided reaction to thrust is maintained. The straight lines of thrust imply that the structures are weightless.

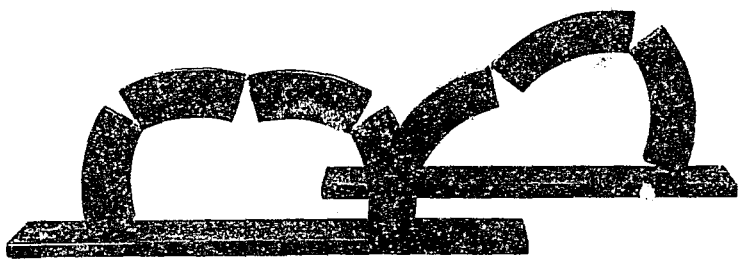


Fig 8.4 Model arches demonstrating advanced five hinged concentric (left foreground) and four hinged eccentric (right background) load collapse mechanisms.

8.5.1 Calculation of the position of the line of thrust

The arch is divided into equal segments with point loads emanating from their centres representing the self-weight of the structure including the road and fill. Reactions are determined by taking moments about estimated reaction hinge positions and the position of the line of thrust relative to the intrados (measured vertically) is determined accordingly. A line of thrust which falls outside of the arch ring indicates an incorrectly estimated reaction hinge position.

Graphical determination of the position of the line of thrust

Ideally, this method of analysis is best performed with a computer-aided drawing program to ensure graphical accuracy. The practice of performing the analysis provides the arch designer with an 'intuitive feel' with which to anticipate the consequences of future modifications to various arch design parameters, particularly regarding the contribution of self-weight in maintaining equilibrium. The example shown in Figure 8.5 considers a 90 kN wheel load at mid-span to be a point load shared equally between the two halves. Because of its symmetry, only one half is considered:-

a) The arch is divided into an arbitrary number of equal segments (seven in this case). The weight of each segment (comprising the self-weight of the structure including the road, fill and live-load) is represented as a point load emanating from the centre of each segment.

b) Horizontal thrust at the upper third of the crown is calculated by taking moments about one of the reactions (Assumed at A in this case).

$$H_z \times r = 50 \times 2.1 + 6 \times 1.8 + 7 \times 1.5 + 8 \times 1.2 + 12 \times 0.9 + 12 \times 0.6 + 11 \times 0.3$$

$$\therefore H_z \times 1.5 = 157.2 \text{ kNm (substituting r)}$$

$$\therefore H_z = 104.8 \text{ kN}$$

c) The force diagram is plotted, using Bow's notation (on the left hand side of Figure 8.5).

d) Starting horizontally at the crown reaction, the line of thrust is plotted on the arch profile parallel to the equilibrium vectors on the force diagram; 0-0 to the middle of the first segment; changing slope to 0-50 to the middle of the second segment and so on. The fact that the line of thrust plotted does not exactly intersect point A signifies that

this assumed reaction hinge was not precise.

e) The process is repeated for different load conditions and/or arch shapes and is useful in determining violations of design criteria such as the formation of mechanisms or the middle third rule.

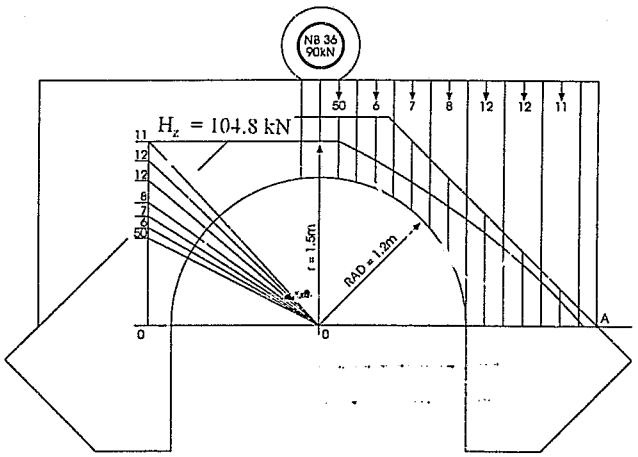


Fig 8.5 Graphical method of determination of the position of the arch's line of thrust. Each of the seven segments is 300 mm wide.

8.5.2 Finite element analysis and the evaluation of a simple model

Until recently, finite element computer models for the analysis of arch bridges were inaccessible to many designers. They typically required very expensive computer hardware and software and the finite mesh had to be generated manually; a process that took much longer than graphical arch design methods. A recently availed two dimensional plane strain program ("Prokon" Groenkloof, SA & London, UK Plane Stress/Strain Analysis) was made available to this initiative for the analysis of RMC arches. Given a grid spacing within the limitations of its capacity, the model is able to quickly generate its own mesh and therefore easily accommodate design changes. Moreover, it is able to model structures comprising more than one type of material and can therefore accommodate additional overburden layers and in so doing, distribute concentrated wheel loads over the masonry. However, additional overburden layers do not normally form part of a RMC bridge structure and there may be better ways of modelling the resistance of fill. The model's limitations include a restriction to assumed linear elastic homogeneous material response and its inability to model independent behaviour in compression and tension, precluding it automatically allowing cracks to develop. Furthermore, it makes the assumption that the foundations are infinitely stiff; an assumption perhaps realistic

when founding upon rock but questionable in soils. Required input includes an estimation of material properties; namely, density, stiffness and Poisson's ratio as well as imposed point loads and UDLs. It assumes the materials to be homogeneous and allows the user to manipulate the mesh size to achieve an acceptable compromise between detail and processing time. The output data include quantified maximum compressive and tensile stresses, together with their coordinates and stress vectors as well as deflections and their coordinates.

8.6 FINITE ELEMENT ANALYSIS APPLIED TO A TYPICAL STRUCTURE AND THE EXPLORATION OF DIFFERENT APERTURE SHAPES

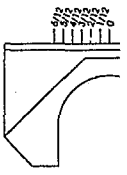
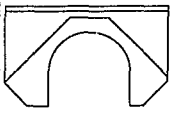
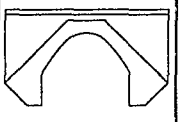
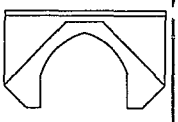
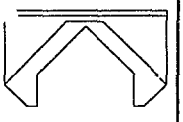
Table 8.2 shows some effects of four different aperture shapes, each with an equal cross-sectional area of 2.3 m^2. A two dimensional plane strain finite element model "Prokon" was used to compare maximum stresses and deflections at midspan. The structure was designed to carry a temporary deviation road as part of a national road rehabilitation program in the Northern Province of South Africa and was therefore designed to withstand NB36 loading. The assumed material properties, including a 150 mm subbase immediately below the road level and a granular fill between the subbase and the road level, are presented in Table 8.1.

Table 8.1 Material properties adopted for theoretical analysis.

Material	Elastic modulus (GPa)	Poisson's ratio	Density (kg/m^3)
RMC	28	0.2	2400
Granular fill	0.12	0.35	1800
Subbase	0.2	0.35	1800

A sensitivity analysis revealed a 100 mm grid interval, for generating the finite element mesh, to yield slightly higher stresses than both larger and smaller spacings. As well as being conservative, this grid size was found to produce acceptable detail without excessively compromising the processing speed. A one metre wide elevational section was assumed to carry the entire 300 mm wide NB36 wheel load. This assumes a load spread of only 350 mm, on either side of the wheel, over a depth of about 800 mm.

Table 8.2 Maximum masonry tensile and (compressive) stresses and deflections at mid-span (μm) in a RMC bridge with a constant aperture of 2,3 m^2 and NB36 wheel-loading at various distances (defined in terms of fractions of arch spans) from mid-span.

 Position of load	 Circular 2.4 m span	 Parabolic 2.7 m span $y = -0.692 X^2$	 Gothic 2.6 m span	 Triangular 3.0 m span
Load zero offset	0.267 MPa 30.3 μm	0.210 MPa 28.7 μm	0.310 MPa 30.6 μm	0.083 MPa 28.6 μm
Load 1/12 offset	0.223 MPa 29.0 μm			
Load 2/12 offset	0.201 MPa 26.1 μm	0.187 MPa 23.8 μm	0.199 MPa 25.0 μm	0.445 MPa
Load 3/12 offset	0.228 MPa 22.4 μm			0379 MPa
Load 4/12 offset	0.275 MPa 18.6 μm	0.035 MPa 15.9 μm	0.260 MPa 16.5 μm	0.406 MPa
Load 5/12 offset	0.291 MPa 14.9 μm		0.270 MPa 13.0 μm	0.415 MPa
Load 6/12 offset	0.285 MPa 11.8 μm			
Max compressive stress	(0.573) MPa @ 4/12 off	(0.803) MPa @ 4/12 off	(0.547) MPa @ 5/12 off	(1.070) MPa @ 4/12 off

8.6.1 Calibration and interpretation of the model's output

A glance at the maximum stresses (presented in Table 8.2) confirms their very low order of magnitude. A comparison of these values with recorded live-load-induced stresses (measured with electronic resistance strain gauges), in a similar structure[8.2] (see Appendix 1) confirms that the model is predicting stresses of a realistic order. Unfortunately, the measuring apparatus used was found to be insufficiently sensitive to record the minuscule live-load-induced strains with much accuracy. Consequently, an improved mechanical strain amplifying device (comprehensively described elsewhere[8.20] {see Appendix 6}) was developed and refined to permit accurate calibrations.

In essence, the device (shown diagrammatically in Fig 8.6) consists of a thick walled, hollow steel tube with a reduced cross sectional area (onto which electronic strain gauges are bonded) over a fraction of its length. Both ends of the tube are fixed to rigid plates to transfer strain from the structure directly to the tube. The entire length of the tube is surrounded by a plastic conduit to preclude any additional load transfer between its ends.

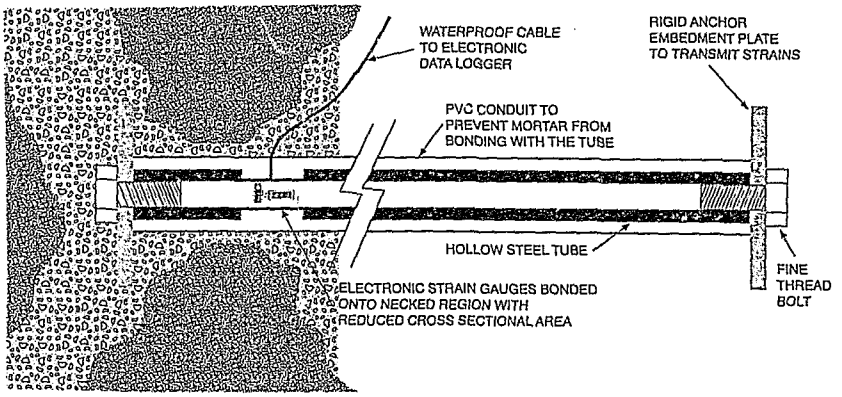
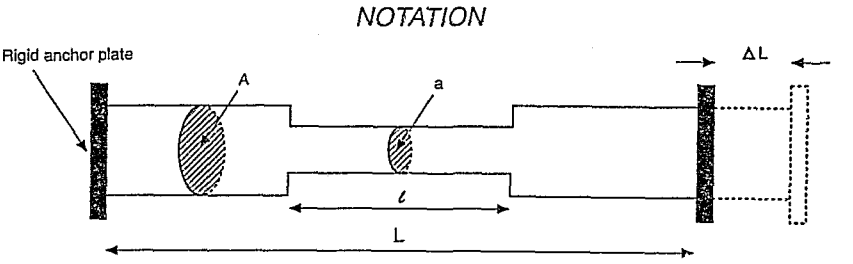


Fig 8.6 Schematic representation of the strain cell amplifier device developed for recording small strains in RMC bridge structures.

Manipulation of the length and cross sectional area of the necked region permits the mechanical strain amplification factor to be adjusted. The formula used to compute the device's amplification factor is derived in Figure 8.7 and mechanical strain amplification factors for a number of neck configurations are presented in Table 8.3.

A one metre long, 16 mm diameter tube with a 3 mm wall thickness, a neck length of 20 mm and an A/a ratio of 10 was used. This geometry afforded sufficient space to accommodate small electronic gauge rosettes and yielded an amplification factor of about one order of magnitude. Sets of these devices were constructed and embedded into three bridge structures during their construction. Unfortunately, flooding, theft of the copper cable and a last minute variation to the design of one of the structures prevented the capture of any strain data from each of these bridges respectively. Nevertheless, the very small live-load-induced strains recorded in the pilot study[8.2] suggest it is extremely unlikely that a structure (with typical proportions; such as the one investigated) would fail as a result of masonry crushing. They are far more likely to fail in tension, in which case, the triangular shape would fail first as its load rolls 1/6th of its span off-centre, followed by the gothic arch with its load mid-span, followed by the circular arch with its load 5/12ths off-centre, followed by the parabola loaded mid-span. The parabolic shape, as adopted by some Zimbabweans for proprietary precast shells[8.3,8.19] as permanent formwork, would appear to be optimal in limiting arch tension. However, the circular arch should probably be retained for ordinary construction since parabolic formwork would be very difficult to construct with the humble resources available to most developing rural communities. An inference, which may be deduced from the insensitivity of these structures to significant changes in aperture shape, is their tolerance of considerable geometric

inaccuracies, such as the exact arch shape and road level as well as formwork deformation. They would most certainly accommodate the magnitude of setting out errors (of about 100 mm) which dos Santos[8.19] anticipates from an unsophisticated workforce with limited supervision.



ε_T = Average longitudinal strain between anchorages
ε_N = Longitudinal strain in the neck
ε_R = Longitudinal strain in the non-necked region
a = Cross sectional area of the necked region
A = Cross sectional area of non-necked region
L = Length between anchor plates
ℓ = Length of necked area
f = Amplification factor

$$\varepsilon_R = \varepsilon_N \times a/A \qquad \text{(EQN 1)}$$

$$\Delta L = \varepsilon_N \times \ell + \varepsilon_R (L - \ell) \qquad \text{(EQN 2)}$$

$$= \varepsilon_N \times \ell + \varepsilon_N \times a/A \ (L - \ell) \qquad \text{(Substituting EQN 1 into EQN 2)}$$

$$\therefore \ \varepsilon_N = \frac{\Delta L}{\ell + a/A \ (L - \ell)} \qquad \text{(EQN 3)}$$

$$\varepsilon_T = \frac{\Delta L}{L} \qquad \text{(EQN 4)}$$

$$f = \frac{\varepsilon_N}{\varepsilon_T} \qquad \text{(EQN 5)}$$

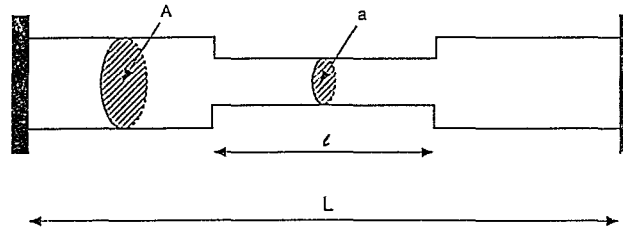
$$= \frac{\Delta L}{\ell + a/A \ (L - \ell)} \times \frac{L}{\Delta L} \qquad \text{(Substituting EQNs 3 \& 4 into EQN 5)}$$

$$f = \frac{L}{\ell + a/A \ (L - \ell)}$$

Fig 8.7 Derivation of mechanical strain amplification factor.

Table 8.3 Mechanical strain amplification factors for a number of neck configurations; computed using the mechanical strain amplification formula derived in Figure 8.7.

ℓ/L \ A/a	1	2	3	4	5	10	20	50	100
1	1	1	1	1	1	1	1	1	1
0.5	1	1.33	1.50	1.60	1.67	1.82	1.90	1.96	1.98
0.2	1	1.67	2.14	2.50	2.78	3.57	4.17	4.63	4.81
0.1	1	1.82	2.50	3.08	3.57	5.26	6.90	8.47	9.17
0.05	1	1.90	2.73	3.48	4.17	6.90	10.26	14.49	16.81
0.02	1	1.96	2.88	3.77	4.63	8.47	14.49	25.25	33.56
0.01	1	1.98	2.94	3.88	4.81	9.17	16.81	33.56	50.25

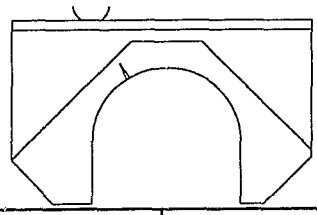
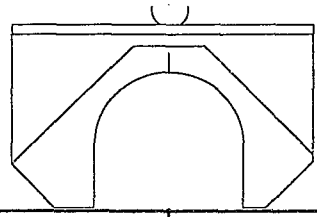


8.6.2 Finite element modelling of tension cracks and collapse mechanisms

Table 8.4 presents data generated by the model when triangular tension cracks are introduced by substituting triangular areas, at points of maximum tension, with a material of greatly reduced stiffness. In both cases, the tensile stresses were inversely proportional to crack length, confirming that the cracks facilitate in relieving tensile stresses. The corresponding increases in compressive stress remained lower than the maxima recorded in Table 8.2. Thus, even after load redistribution, consequent upon cracking, failure by masonry crushing remains extremely unlikely. Figure 8.8 contrasts the tensile stresses before and after cracking. The very small "post-crack" tensile stress at the extrados to the right of the crown indicates the approximate origin of another tensile crack which needs to occur to create the fourth hinge prerequisite to the formation of a kinematic mechanism before the arch will collapse. Figure 8.9 shows this mechanism in action. However, this is unlikely to occur, since this tensile stress is probably

too low to cause a crack, and even if it succeeded, the abutments and fill would be able to resist all but a cataclysmic kick of the hinge. The coarseness of the aggregate interlock in RMC makes voussoir and/or haunch sliding failure mechanisms[8.21] extremely unlikely, even in severely cracked structures.

Table 8.4 Stress and deflection changes as a consequence of cracking at points of maximum tension under wheel load.

				
Crack length (mm)	200	300	200	300
Max tensile stress (MPa)	0.182	0.150	0.221	0.075
Decrease in max tensile stress (%)	37	48	17	72
Max comp stress (MPa)	0.504	0.538	0.488	0.491
Midspan intrados deflection (μm)	10.7	10.1	33.3	33.8
Increase in mid-span intrados deflection	-28%	-32%	10%	12%

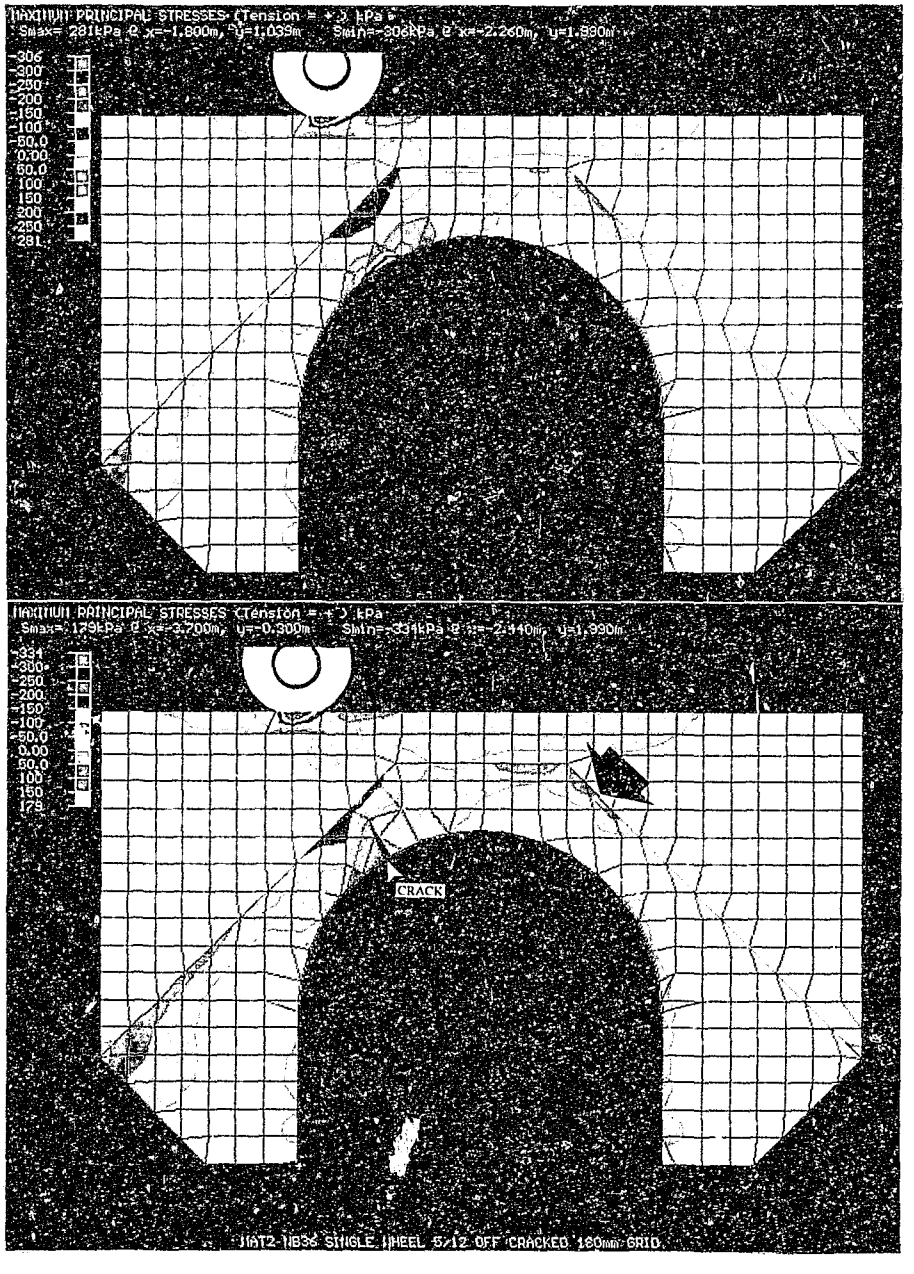


Fig 8.8 Tensile stresses in an arch loaded at 5/12ths of its span off-centre before (above) and after (below) cracking. The arrow pointing to the extrados to the right of the crown indicates a region of increasing tensile stress which needs to crack to form the fourth hinge, prerequisite to the formation of a collapse mechanism.

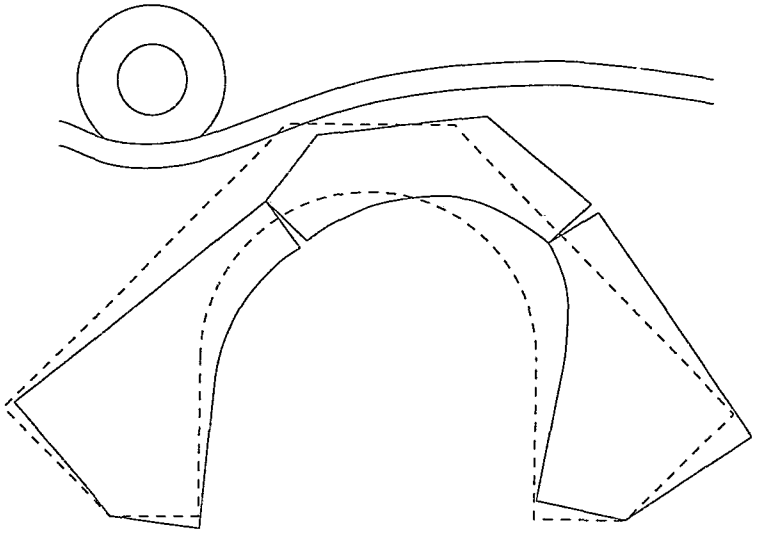


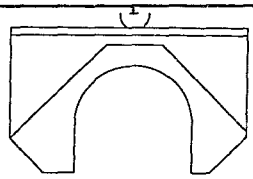
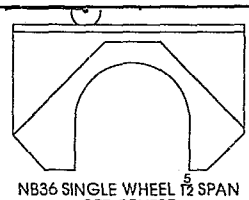
Fig 8.9 Unlikely four hinged collapse mechanism.

8.6.3 Buoyancy compensation in the estimation of self-weight

Self-weight plays a crucial role in the maintenance of arch equilibrium, since the heavier the arch structure itself, the greater its tolerance of point loads. As the arch becomes submerged, upon a rising water level, a proportion of its weight, equal to its displaced volume, may be relieved. Thus, in supporting the structure and making it lighter, buoyancy also makes it weaker. The effect is significant considering that upon immersion, the weight of RMC would be reduced by about 42% from, say 2400 kg/m^3, to 1400 kg/m^3. Table 8.5 presents the results of a *finite element analysis which quantitatively explores the changes which occur as the same* arch bridge is relieved of some of its weight (by buoyancy). The analysis was made simply by reducing the material "density" of the immersed materials by 1000 kg/m^3.

The structure loaded at mid-span experiences a 20% increase in tensile stress as it becomes immersed. This is a result of the fabric being called upon to restore equilibrium upon the removal of dead weight. The reason why this increase in tension does not appear to be mirrored by an increase in compressive stress is because the net compressive stress is the sum of both the fabric and gravitational reactions. As the fabric reaction increases in proportion to load, as it must to maintain equilibrium, so the gravitational reaction diminishes at a faster rate. In this case, the rate of gravitational decrease appears to occur at about twice the rate of the fabric increase; reflecting a net decrease. The observed decrease in mid-span deflection (raising of the deck) supports this hypothesis.

Table 8.5 The effect of buoyancy on a RMC arch structure.

	 NB36 SINGLE WHEEL MIDSPAN		 NB36 SINGLE WHEEL 5/12 SPAN OFF CENTRE	
	Dry	Buoyancy compensated	Dry	Buoyancy compensated
Tensile stress (MPa) (% Increase)	0.261	0.312 (20)	0.281	0.316 (12)
Compressive Stress (MPa) (% Decrease)	0.422	0.397 (6)	0.548	0.485 (11)
Mid-span deflection (μm) (% Decrease)	30.1	26.8 (11)	14.9	11.5 (23)

8.7 ANCHORAGE AND RESISTANCE TO TRANSIENT LOADING

Anchorage against sliding and overturning from transient flood-loading may best be achieved by socketing into bedrock and/or grouting pre-bent anchor bars into drilled holes (See section 7.11). Although standard jackhammer drill steels may be capable of forming a hole sufficiently large to accommodate 25 mm nominal diameter reinforcing bars, it may be preferable to use a lesser diameter bar to ensure a good grout surround. Bar spacers obviously help to achieve this cover. The practice of cleaning the hole with high pressure water and air, followed by filling the hole with liquid grout and placing the anchor into the grout enhances bond, cover and resistance to corrosion. If the grout settles, the hole may be topped up with more grout. The projecting bars should be sufficiently long and adequately embedded into the structure to preclude masonry tensile failure immediately above the proximal end. Furthermore, the projecting bar should not be embedded directly in RMC, but a cavity (of about 200 mm in diameter) should be left around the bar and subsequently filled with a cement-rich high slump concrete to increase the bar/aggregate interlock - thereby affording greater pullout resistence. Anchorage may be further improved by linking the anchor bars with transverse reinforcing steel secured within the "crooks" of their cranked radii.

8.8 CENTERING (FORMWORK)

Timber has been used as centering for arches for many hundreds of years. Recently, the high cost of timber has challenged builders to consider other alternatives. One method deployed in the Northern Province of South Africa uses corrugated iron sheeting pre-cranked to the intrados radius[8.2,8.22]. In the case of large radii, these sheets are propped by gum-poles until the masonry becomes self-supporting (see Figure 8.10). A shortcoming of this method is that frequently, upon construction loading, the form distorts. The use of stiff plywood profiles, onto which the ribs and sheets are attached (see Figure 8.11) is proposed as an improvement. The adjustable jacks and short sections (as opposed to one continuous form) are intended to reduce the difficulty of stripping and prevent damage in handling to facilitate maximum reuse.

Another method, claimed to cost 25% of the corrugated iron method, uses 50 mm saplings spanned between temporary masonry profiles over which spent paper cement packets are draped[8.2,8.22]. However, the sag between points of support and the rougher finish is likely increase the risk of blockage and cause unacceptable turbulence to fast flowing water.

Recently, a Zimbabwean company[8.3,8.19] patented a proprietary precast concrete permanent form shell named a "shelvert" which is delivered to site in two halves. Once joined at the crown the structure is stable and capable of supporting load without any reliance upon the strength of the spandrel masonry whatsoever (see Figure 8.12). An advantage of the system is that it is not limited to a circular intrados.

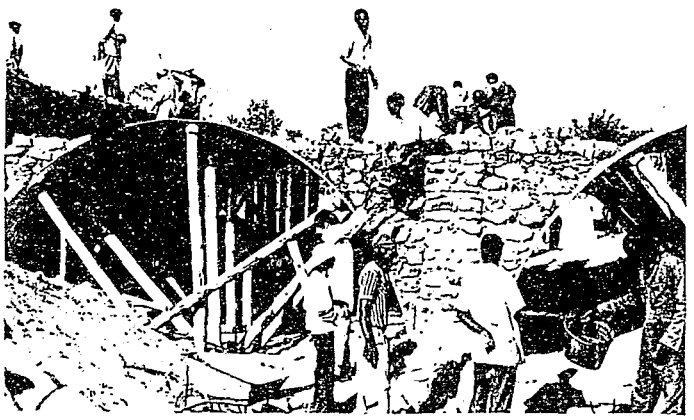


Fig 8.10 Cranked corrugated iron sheeting and gum poles used as centering in the Northern Province of South Africa.

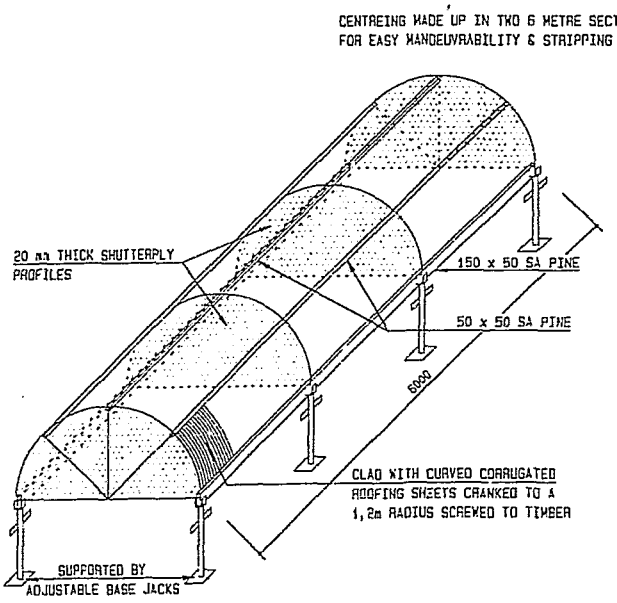


Fig 8.11 Proposed plywood stiffening to prevent distortion of the circular symmetry.



Fig 8.12 Proprietary precast concrete shells used as permanent formwork in Zimbabwe. The parabolic shape is highly efficient in minimising tensile stresses in the structure. Photograph courtesy of Fort Concrete Zimbabwe.

The small bridge (shown in Figure 8.13) at De Vasselot Tsitsikamma National Park, South Africa became unserviceable as a result of corrosion of its steel liner after 25-30 years of service. The top of the liner remains extremely well protected, by zinc and bitumen; however, its bottom, which is subject to abrasion, has corroded away in the high chloride environment. It is proposed that the existing liner be reused as centering for a RMC structure without any additional support. Corrosion will eventually consume the centering, leaving a permanent and attractive RMC arch.

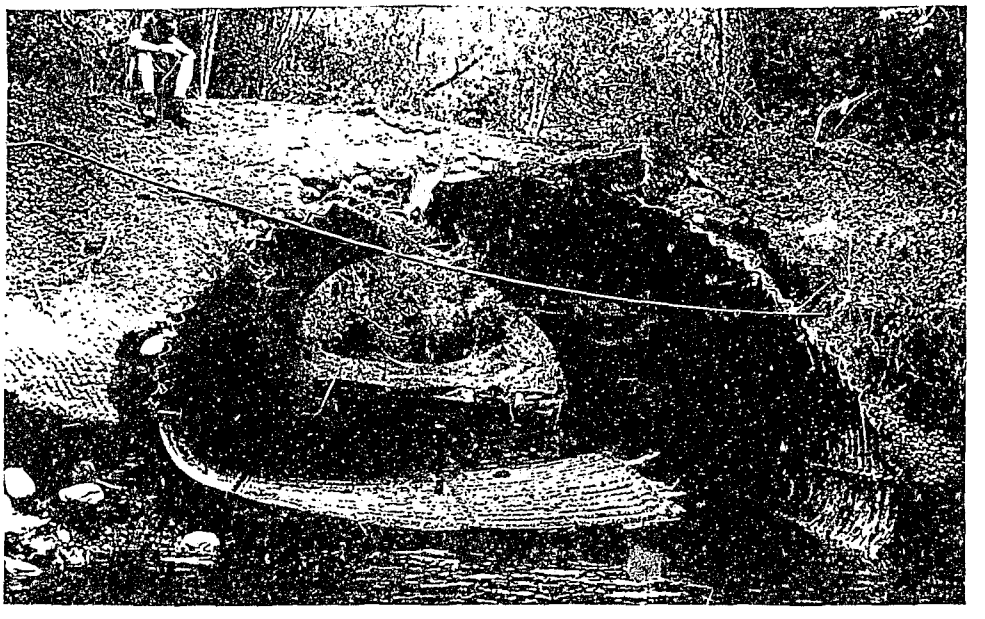


Fig 8.13 This corroded steel liner can be used as centering for a permanent RMC bridge replacement.

8.9 PLACING OF RMC

With the exception of the smallest streams, construction should be restricted to the dry season when many southern African rivers cease to flow, to minimise the risk of flood damage whilst the new structure is most vulnerable. Nevertheless, to guard against the freak occurrence of an out of season flood, large apertures should be left in the formwork to allow water to pass under the structure. The stockpiling of river sand will also ensure that the contractor is able to continue working should the river start to flow.

The radial placing of boulders, with respect to their longitudinal axes, around the apertures (as is shown in Figure 8.1), is recommended to derive maximum benefit from the bilateral constraint afforded by the flatter surfaces. As far as possible, the placement of RMC should commence from both sides simultaneously to prevent asymmetric distortion of the centering. The placing of boulders in horizontal layers (with their longitudinal axes sub-parallel to the principal stress trajectories), as is presently the custom, has been shown (see Chapters 3 and 7) to reduce the compressive strength, because the stiff inclusions tend to cleave the matrix apart rather than tie it together.

8.10 JOINTS IN MULTIPLE SPAN BRIDGES

Although joint details in multiple span RMC bridges have seldom been implemented in practice and no reported structural failures appear to have been attributed to their omission, there is some argument for their insertion. Shelton[8.6] cautions against the possibility of the formation of inclined tension cracks created as a consequence of foundation movement and/or temperature drop. The cracks would naturally tend to occur near the thinnest section at the crown and with continual thermal pumping may cause significant misalignment of the deck (see Figure 8.14). Varkevisser[8.23] recalls a RMC bridge in Makumbura, Zimbabwe which was cracked right through its crown, probably as a result of differential foundation settlement, yet it remained serviceable for many years. The provision of vertical contraction/movement joints at the crown of the arch or from foundation to bridge deck between successive openings is an obvious solution to the problem. Such joints have been formed by building alternative sections of masonry and then painting the joint surfaces with lime-wash before building in the remaining intermediate masonry. Although discontinuity of moments and tension is desired at the joints, aggregate interlock should be fostered for the transfer of shear to preclude arch sections from sliding relative to their adjacent neighbours. Consequently, the practice of building these joints against smooth shutters or by the inclusion of planer bond breakers is not recommended.

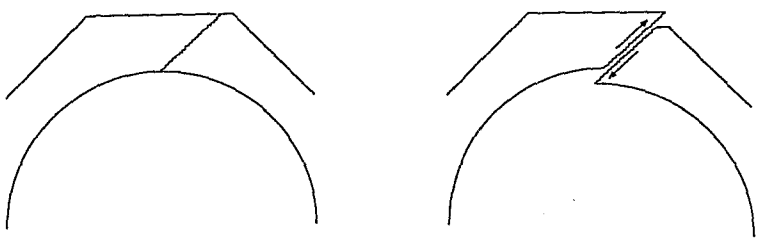


Fig 8.14 Possible inclined tension crack caused by contraction and/or foundation movement.

8.11 ROAD SURFACES

In the absence of layer-works, the RMC finish is typically too rough and uneven for use as a running surface for traffic. Screed toppings have been used[8.2] but they tend to crumble and wear quickly[8.24]. The Zimbabweans recommend a 30 MPa concrete slab about 150 mm thick (unreinforced[8.3]) and 100 mm thick if mesh reinforced[8.18].

8.12 MAINTENANCE

Even apertures in the best structures are prone to becoming fouled and need to be cleared of driftwood and other debris from time to time. Inspections for blockages of the apertures, scour and structural damage should be undertaken routinely; at least after significant rainfall.

8.13 CONCLUSION

The competitive advantage of RMC arch bridges is often dependant upon their situation. Ideally, a bedrock foundation and a straight section of stream with moderately inclined banks is required. The positioning and geometry of the structure must be designed to minimise obstruction to the flow of water and debris. A graphical method to determine the position of the arch's line of thrust and a plane strain finite element model for arch structural analysis were demonstrated. This finite element model was used to explore the sensitivity of a structure to four different shape apertures (all of equal area) and the structural response of a circular arch to its overtopping and cracking. Of the shapes considered to date, a parabolic aperture was found to be optimal in limiting tensile stresses but is unlikely to succeed the circular arch for the latter's simplicity of construction. The effect of buoyancy due to overtopping was demonstrated to increase tensile stresses within the arch and thereby reduce its capacity to support point loads. Failure by cracking in tension under wheel-loading was simulated theoretically, revealing that the cracks may serve a useful role in alleviating tensile stresses. Other modes of failure, including a hinged mechanism, were considered and shown to be unlikely to occur in the type of structure investigated. Hence, indications are that the critical case of the RMC arch is governed not by material strength, but by equilibrium. Once the problem of equilibrium is satisfied, compressive stresses automatically tend to remain within acceptable levels. The robust sections of typical RMC structures make them sufficiently tolerant of the sort of geometrical inaccuracies expected from an unsophisticated labour-force without the provision of reinforcement.

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CHAPTER 9

CONCLUSION

Previous chapters contain their own concluding summaries within the ambits of their individual objectives. The purpose of this final chapter is to reiterate and collate the most significant of these findings within the broader context of this thesis. These overall conclusions indicate the success of this initiative in fulfilling its objectives, areas in need of continued investigation and some personal reflections.

The objectives of this study were declared in Chapter 1. Essentially, they were to:-

1) Investigate the potential of RMC construction as a technically competent and feasible employment-creating technology.

2) Explore the mechanical properties and physical behaviour of RMC.

3) Propose material specifications and guidelines for the design and labour-intensive construction of RMC structures with particular reference to arch bridges.

9.1 SUMMARY OF SIGNIFICANT FINDINGS

9.1.1 The competitive advantages of RMC and its potential to provide sound infrastructure and create employment

The literature surveyed (Chapter 2) indicates that the competitive advantage of RMC before the 20th century evolved primarily as a consequence of its low energy requirement. Its

competitive advantage today appears to emanate as a consequence of its effective use of abundant local resources and its relative independence of costly mechanised plant and transported materials' acquisition. The success of many recent labour-intensive RMC projects (which proved to be cheaper than conventional alternative designs) clearly indicates the potential of RMC to provide cost-effective infrastructure and employment opportunities. The cost savings realised in these projects (over more conventional alternatives) also confirm that this material is more tolerant of low productivity from the labour-force than many other labour-intensive technologies. In addition, growing awareness of the need to create employment for the unskilled destitute, and the need to provide ecologically acceptable infrastructure, are factors that are likely to favour the choice of RMC in future. However, lack of knowledge of the mechanical properties and physical behaviour of RMC have precluded engineers from rationally designing structures of known reliability. Instead, they have been forced to make reference to precedent. Examination of the evolution of RMC structures over the past 200 years shows an apparently unjustified growing dependance upon the reserves of this material's strength. Furthermore, designers and contractors often fail to appreciate the requirements for humans to labour effectively on labour-intensive RMC projects. These factors threaten the potential application of RMC in future.

9.1.2 The mechanical properties and behaviour of RMC

In retrospect, this second objective was an ambitious undertaking. Firstly, there was a minimal existing basis of knowledge to build upon. Secondly, it necessitated the physical exploration of a material not conducive to conventional testing. Ideally, test specimens, for exploring such a complex and coarse heterogeneous material, should have been bigger and more numerous than the ones used in this study; yet, these were the biggest and most numerous that could be managed given the resources at hand. Therefore, the configuration of experimental investigations and the interpretation of the data that followed demanded the utmost care. To minimise the chances of misinterpreting data due to inevitable random scattering of a few results, extreme opposite control variables were compared whenever this was possible and only one control variable was manipulated at a time. Thus, rather than an exhaustive study of any one property of RMC, this initiative might more aptly be described as a pioneering effort to explore new territory and discover factors which significantly influence the physical behaviour of RMC.

The investigations which explored the mechanical properties of RMC have revealed that this highly heterogeneous material has the potential to exhibit great strength and stiffness

anisotropy. Accordingly, RMC will probably never be as predictable as ordinary concrete and a high partial material factor will remain a necessary precaution. Contiguous particle interaction and the potential for anisotropy (as a consequence of an inherently predominant orientation of elongated inclusions with respect to the principal stress) appear to account for much of RMC's unique mechanical properties and physical behaviour. Under uniaxial compressive stress, fracture appears to be initiated by debonding of the stone-mortar interface parallel to the principal stress before the material resists its maximum stress. Discontinuities between the phases caused by contaminated rock surfaces, bleed-water lenses and trapped air pockets appear to initiate parting of the phases. Thereafter, failure appears to be governed by mechanisms which form between boulder inclusions. Rock fracture, as a result of high bearing stresses, is common where contiguous boulders interact and bear upon their adjacent neighbours. These bearing stresses increase with increasing boulder size and stiffness. Elongated inclusions have an observed tendency to split, wedge or cleave the matrix apart where their longitudinal axes lie near-parallel to the principal compressive stress trajectories.

In addition to the exponents currently recognised to govern the stiffness of conventional concrete, boulder shape and orientation appear to have a significant effect on RMC stiffness and dilation, probably more so than rock stiffness. Mechanisms which form between the boulder inclusions may cause high proportions of dilation perpendicular to the principal compressive stress. The deliberate orientation of elongated boulder inclusions, perpendicular to the principal compressive stress, appears to be an effective means of combatting this dilation. RMC stiffness appears to increase as the number of inter-particle contacts disposed to carry load within the principally stressed area increases. Thus, many small contiguously placed boulders appear to produce a stiffer composite than a few large suspended plums.

In Chapter 5, it has been hypothesized that, in addition to factors known to govern the thermal strain response of conventional concrete, inclusion size and particle contiguity may have a significant effect on the thermal contraction coefficient of RMC. Where boulders are thermally less responsive and stiffer than the matrix phase, contiguous interaction between the boulders may be expected to yield a composite which exhibits different thermal strain responses above and below the placement temperature. Above the placement temperature, the coefficient of expansion is governed primarily by the more active matrix. Below the placement temperature, the coefficient of contraction is checked by the contiguous particle interaction precluding closer packing of the boulder inclusions. This may contribute to the apparent absence of post-hydration thermally induced cracks in wide valley monolithic RMC dams. Precautions to limit thermally induced tensile stresses in RMC, by limiting the placement temperature are recommended. Finally, until reliable quantified coefficients of thermal contraction for RMC

avail, the use of equivalent data derived from conventional concrete and/or predicted values for concrete appears to be the most prudent means of estimating a RMC structure's thermal response.

9.1.3 Human labour considerations of the most needy recipients of employment

Among the most needy recipients of employment are many southern Africans who are not well disposed to selling even their labour; including single women with dependents, the elderly without pensions and the partially disabled. The health and nutritional status of this targeted population has been shown to be far from optimal and declining. These impediments do not enhance their physical capacity to perform hard manual work. Even at the best of times, the available human energy to perform physical work is limited and consequently, if not efficiently harnessed, the economic cost of human energy will be high. Notwithstanding, human labour has a number of competitive advantages when compared to man-made machinery including:- its versatility, its extremely efficient energy conversion efficiency and its disassociation from fixed cost economic inefficiencies. (Conventional machinery is not cost-effective when it is underutilised for its intended purpose. However, human labourers are versatile and can adapt to perform the tasks of many individual machines thereby remaining occupied.) Recommendations to improve the effectiveness of this targeted human labour population include:-

1) The provision of healthcare and nutritional supplementation at the workplace in addition to the minimum financial remuneration as payment for their labour.

2) Reducing the physical demands of 'leisure' activities by assisting in the provision of transport to and from the workplace, providing childcare facilities and carrying water.

3) Scheduling the most physically demanding tasks during cool periods to reduce heat stress on the body.

4) Harnessing gravity to advantage.

5) Promoting developments in hand-tools and human-powered machinery.

6) Design and specification taking ergonomics into account.

9.1.4 Proposed material specifications

Although there can be little doubt that the proposed material specifications and design and construction guidelines, which evolved from the findings of this study, could be improved upon in future, this manuscript appears to contain the most comprehensive collation of technical protocol on RMC practice to date. As an aid to design, it provides a point of departure which is presumably better than nothing. Some of the important recommendations are listed below:-

1) Blasted rock with clean unweathered faces has been observed to offer a better substrate for mortar bonding than rocks with faces that have been weathered or soiled and is therefore recommended for important work.

2) Excavating and stockpiling sand from different depths and different locations in rivers is recommended to achieve better grading.

3) The addition of a small proportion of hydrated lime, as an admixture, may greatly reduce bleeding of RMC and enhance several other desirable properties as well.

4) Simple, straightforward mix proportioning is necessary to avoid errors when employing an unsophisticated labour-force.

5) Specifying a minimum volume fraction of rock (to ensure particle contiguity) and strict adherence to proper curing will help to prevent post-hydration cracking.

6) The faces of construction joints must be designed and adequately prepared to prevent shear sliding failures.

7) Steel anchors, dowels and reinforcing bars should be surrounded by conventional concrete rather than RMC to enhance bond to the steel.

8) Caution should be exercised when specifying the use of South African dolerites as these materials have the potential to precipitate internal bond failure due to their combined high stiffness and thermal stability, coupled with their poor bonding characteristics if weathered.

9.1.5 Arch bridge design and construction guidelines

1) The feasibility of a RMC structure may depend upon its situation. Ideally, a bedrock foundation and a straight section of stream with moderately inclined banks is required.

2) The positioning and geometry of the structure must be designed to minimise obstruction to the flow of water and debris.

3) Measurements of insitu live-load-induced strain have confirmed theoretically predicted low stresses in existing RMC arch bridge structures. The critical case in RMC arch bridge structure design appears to be governed not by material strength, but by the maintenance of equilibrium. Once the problem of equilibrium is satisfied, compressive stresses automatically tend to remain within acceptable levels.

4) Of shapes considered to date, the parabolic aperture was found to be optimal in limiting tensile stresses under the application of wheel-loading. A finite element analysis has shown that tension cracks may serve a useful role in alleviating tensile stresses in a typical RMC arch bridge subjected to wheel-loading.

5) The thick sections, typical of RMC structures, are sufficiently tolerant of the sort of geometrical inaccuracies expected from an unsophisticated work-force.

6) These structures may be designed without tension reinforcement.

9.2 THE NEED FOR AN IMPROVED DESIGN PHILOSOPHY

An acceptable proposal for a RMC structure needs to satisfy more than structural competence and economic feasibility. It has to be socially, politically and ecologically acceptable and it has to be conceived to be assembled by human hands. A number of decisions that the designer makes will impact on factors other than just the structure. For example:-

1) The period chosen to build the structure will determine the risk of flooding of the works, the final placement temperature of RMC and the level of heat-stress placed on the labour-force. The dry winter months (over most of southern Africa) offer the most favourable conditions in terms of these factors.

2) The specification of a maximum and minimum boulder mass will not only limit internal stresses in RMC but it will impact upon the productivity of the labour-force.

3) The sources of materials and the way in which they are stockpiled and managed will affect the quality of the RMC and the effectiveness of the labourers.

9.3 THE FUTURE SCOPE AND POTENTIAL OF RMC TO SERVE MANKIND

Although RMC appears increasingly attractive as an employment-creating technology, capable of delivering sound infrastructure, not every project lends itself to the construction of a RMC structure. To be optimally competitive, RMC structures require a foundation of bedrock, a local abundance of boulders and river-sand as raw materials and an underemployed local community. RMC may therefore not offer the best solution to every need and to use it inappropriately might considerably damage its reputation. For example, in 1997, the author was assigned to design a series of wide RMC arch crossings with low apertures to be founded on granular riverbed material. The estimated cost of these RMC crossings was several times greater than that of conventional pipe culverts.

de Beer[9.1,9.2] has been innovative in integrating rigid RMC with more 'pliable' construction materials such as earth-fill and rock-fill. The advent of these 'composite structures' has increased the range of structural applications for RMC in otherwise inappropriate locations and has facilitated projects which would otherwise not have been viable.

Load-induced stresses, in arch dams built to date, have tended to be low as a consequence of the requirement of a minimum wall thickness to safely accommodate workers and the passage of materials by wheelbarrows within the confines of the two external masonry leaves which contain the inner hearting. The Zimbabweans[9.3] consider a minimum wall thickness of 800 mm to be the absolute limit. Where future structures may experience higher stresses, it is suggested that the aelotropic properties of RMC be exploited to engineering advantage by a simple change to the existing method of placing elongated flat boulders. In arch bridges, elongated boulders should be placed radially about the intrados (as illustrated in Figure 8.1). In dams, the maximum dimension of the boulders should be orientated from upstream to downstream.

There is probably a practical limit to the maximum size of future RMC structures. The choice of construction technique may ultimately be governed by the speed at which the structure can be built. For example, it may be critical that a structure is completed during one dry season. The speed at which RMC can be placed is limited by the maximum number of labourers that can be accommodated at the workface. Beyond a threshold, additional workers impede the existing productivity and the problem of transporting materials to the centre of the structure becomes critical. Figure 9.1 shows a high density of labourers during the building of the thrust-block of Maritsane Dam which, in the author's opinion, borders this threshold. (A greater density of labourers would probably have impeded progress). A further factor which may limit the maximum size of monolithic RMC structures may be the evolution of thermally induced stresses upon a temperature drop below that of placement.



Fig 9.1 Congestion of labourers during the building of the thrust block of Maritsane Dam.

9.4 REQUIREMENTS FOR ONGOING RESEARCH

The following recommendations for ongoing research are intended to:- 1) Enhance our understanding of factors which govern the mechanical properties and physical behaviour of RMC. 2) Verify untested hypotheses. 3) Substantiate and improve upon the recommendations arising from this research.

1) A more extensive large-scale specimen testing program is needed to explore ways of improving the mortar bond to weathered rock surfaces. The relationship between mortar matrix strength and RMC composite strength also warrants investigation to validate the hypothesis that a threshold exists beyond which negligible gains in composite strength are achievable as the strength of the mortar matrix is further increased.

2) Although the hypothesis presented in Chapter 5 provides a possible explanation for the absence of post-hydration tensile cracks in large monolithic RMC structures built to date, it is not yet possible to predict whether even larger structures, or structures exposed to more extreme temperature drops, may be susceptible to post-hydration cracking. Further research to quantify the absolute range of coefficients of thermal expansion of RMC, to be expected in practice, is needed to address this question. Moreover, the question of the use of joints to relieve thermal stresses in large RMC structures, such as dams may warrant investigation.

3) The material specifications and design and construction guidelines proposed in this thesis need to be tested, debated and substantiated by practitioners before they can be collated into comprehensive codes of practice which engineers can confidently use.

9.5 FINAL WORD

We do not understand the mechanical behaviour and properties of RMC as well as we understand those of conventional concrete, and we probably never will, due to the difficulties in testing it and the limited interest expressed by those who are able to fund its research. However, RMC could become a strategic material on our planet since indications are that throughout the First-World and Third-World, unemployment and poverty will increase in the foreseeable future. RMC is the type of construction material with which we could build some essential physical infrastructure and contribute to alleviating these two problems.

CHAPTER 9 REFERENCES

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9.3) Chemaly, A.G. *The Construction and Design of Masonry Arch and Gravity Dams. A Report Based on a Visit to Zimbabwe and Several Selected Papers*, Department of Water Affairs and Forestry, Feb. 1995.

SELECTED LIST OF IMPORTANT PUBLICATIONS ON RMC

The following list of publications are, in the authors opinion, the major contributions in the field of RMC engineering.

1) Chemaly, A.G. *The Construction and Design of Masonry Arch and Gravity Dams.* A report based on a visit to Zimbabwe and several selected papers, Pretoria: The Department of Water Affairs and Forestry, 1995.

2) De Beer, Hand-made arch dam, *International Water Power and Dam Construction,* Jan. 1997, pp. 21-23.

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APPENDIX 1

THE PILOT STUDY

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Journal

of The South African Institution of Civil Engineers

TECHNICAL PAPERS / TEGNIESE VERHANDELINGS

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The behaviour of continuous flight auger piles in the quarternary sediments underlying Durban 1

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Technical analysis of the employment creation potential of a National Public Works Programme 18

Joernaal

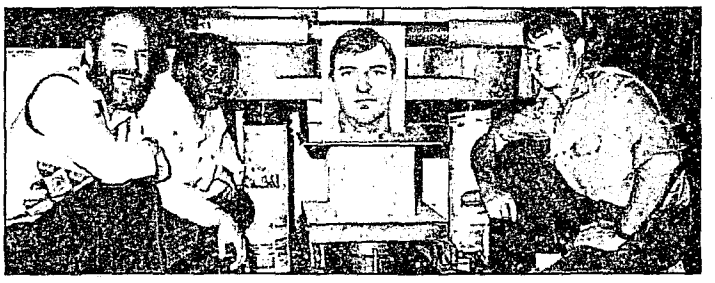
van Die Suid-Afrikaanse Instituut van Siviele Ingenieurs

Appendix 2: Statistical data on road network in Johannesburg

Year	*Total (miles)*	*Tarred (miles)*	*% tarred*
1920	699,63	190,98	27,3
1921	696,23	197,73	28,4
1922	701,41	197,73	28,2
1923	707,56	197,73	27,9
1924	709,27	197,73	27,9
1925	711,25	228,64	32,1
1926	713,31	228,64	32,0
1927	712,14	242,50	34,0
1928	721,41	270,89	37,6
1929	722,23	292,55	40,5
1930	728,13	313,77	43,1
1931-32	na	na	na
1933	759,68	383,09	50,4
1934	759,78	399,09	52,5
1935	762,92	420,22	55,1
1936	780,36	441,65	56,6
1937	781,99	467,45	59,8
1938	804,31	503,07	62,5
1939	845,12	551,19	65,2
1940	848,83	582,08	68,6
Increase 1920 to 1940			
	149,2	391,1	
% increase			
	21	205	

Note: These data relate to the area within the municipal area only. Some development occurred outside designated townships, but within the boundary of the municipality.

Authors of 'Structural aspects of stone masonry arch bridges' (page 13)



Rod Rankin *(right) obtained his BSc (Bldg) and MSc (Civ Eng) at the University of the Witwatersrand (Wits) in 1988 and 1991 respectively. He served in the South African Navy in 1990 and then he worked on an oil rig during the Mosgas project in 1991. He is now a member of the academic staff at Wits University where he is engaged in research towards his PhD, which includes quantative analysis of the properties of uncut stone masonry for use in rural structures to be built with labour-intensive methods as a means of employment creation.*

Geoff Krige *(left) graduated from the University of the Witwatersrand (Wits) with a BSc in Civil Engineering in 1973. He then joined Dorbyl Structural Engineering, working on the design of a wide range of steel construction projects. In 1978 he received an MSc degree in Structural Steel Design from Imperial College, London University, and in 1983 he obtained a PhD degree from Wits. In 1984 he joined the staff of the Department of Civil and Environmental Engineering at Wits as a Senior Lecturer in Structural Engineering. He is currently an Associate Professor and Head of Structural Engineering.*

Dellelegne Teshome *(rear) has a BSc in Civil Engineering (1979) and a MSc in Structural Engineering (1982) from Addis Ababa University and a DEng in Structural Mechanics (1987) from Tokyo University. He worked at Addis Ababa University as a lecturer, assistant professor and associate professor from 1979 to 1984, and is currently a senior lecturer at the University of Durban-Westville.*

Louis Grobler *(inset) graduated in Civil Engineering from the University of Pretoria in 1989. He then joined African Consulting Engineers Inc at their Pietersburg office, where he has worked since. He has been involved in a number of labour-intensive construction projects as a design engineer as well as project manager. He is presently doing an MSc in Construction Management at the University of the Witwatersrand. His involvement in the construction of some 30 small bridges using labour-intensive methods is the basis of his research.*

R G D Rankine, G J Krige, D Teshome, L J Grobler

Structural aspects of labour-intensively constructed, uncut stone masonry arch bridges

Synopsis

A labour-intensive technique for constructing arch bridges using uncut stones and mortar was recently used very effectively. However, at present little is known about the strength and behaviour of the material or the structural form. This paper considers other work that has been done to define the information required to quantify the structural performance. A pilot study is described, in which large size cube tests were used to measure the compressive strength of the stone-mortar matrix. This indicated that the composite strength is lower than the mortar strength, and also that the orientation of the stones is important. A method of measuring the live-load induced stresses in these structures is described and the measured stresses are discussed. Finally, an outline is sketched of further research being undertaken.

Samevatting

Onlangs is 'n brugboutegniek wat staatmaak op arbeidsintensiewe gebruik van ruwe klip en mortel om boogbrûe op te rig, suksesvol toegepas. Daar is egter te min inligting oor die sterkte of gedrag van óf die materiaal óf die ontwerp beskikbaar. Hierdie referaat ondersoek wat reeds gedoen is om die vereiste inligting te defineer wat betrekking sou hê op die strukturele gedrag van so 'n brug. 'n Voorlopige studie waarin groot betonblokke gebruik is om die druksterkte van 'n klip/mortel-matriks te bepaal, word beskryf. Hieruit word afgelei dat die saamgestelde sterkte laer is as dié van die mortel en dat die oriëntasie van die klip belangrik is. 'n Tegniek waardeur die stres, agv toegevoegde las, gemeet kan word, word beskryf en die gemete stres word bespreek. Verder word toekomstige navorsing in hierdie verband beskryf.

Authors' resumes appear on previous page.

Historical background

Many impoverished rural communities throughout Southern Africa experience periods of isolation when floods wash away parts of their road infrastructure, or just make road use impossible during the rainy seasons (Grobler et al, 1995).

In 1992 the former Lebowa Government's Roads and Bridges Department called for tenders for the installation of bridges consisting of structural corrugated steel sheet void formers, as well as simply supported concrete deck structures, as part of its road management system. The method of construction using steel void formers was not particularly labour-intensive, as the linings were placed in position by crane and then back-filled by earth moving machinery. After awarding the contract, the need to use more labour and a supply shortage of these linings, caused by a strike in the steel industry, prompted the engineers to explore alternative construction techniques in order to avoid delaying the project. An innovative labour-intensive technique using uncut stone masonry was proposed and successfully implemented (Grobler et al, 1995; Engineering News, 1993).

Considering the broad guidelines of the Reconstruction and Development Programme (RDP, 1994) and the National Public Works Programme (NPWP, 1994), these structures offer significant benefits, which include the following:

1. Most of the money spent on each bridge (more than 60 per cent) (Grobler, 1995) is invested in the local community through wages, hiring of local subcontractors and local purchasing of material. In addition, a cost saving can be achieved (Engineering News, 1993).
2. At present several more of these structures are under construction, providing jobs for hundreds of local people who would otherwise be unemployed and destitute.
3. This building technique minimizes energy input by using abundant local material and is therefore likely to be sustainable.
4. Although skilled professional advice is required, there is limited dependence on externally sourced materials and labour skills, thus empowering developing communities to provide their own requirements for transportation infrastructure.
5. The design philosophy of a rigid mass structure aims to achieve very low stress levels, thereby increasing the structural tolerance to geometric and material variations, so as to accommodate local skills.

An extensive literature search and a great deal of questioning of relevant authorities seems to indicate that this bridge building method has a close relative in stone arch causeways that were built in Rhodesia during the war years, 1973 to 1980. The design considerations and methods of constructing the Rhodesian arch causeways have been reported by Mainwaring and Petzer (1988), Mainwaring and Hasluck (1985) and Wooton and Stephens (1987). These structures were also built manually with uncut, hand-picked stone and mortar, but with three principle differences worthy of mention: (a) the maximum spans of their bridges were significantly smaller, not exceeding 2 m to 3 m; (b) the structures contained no reinforcing steel; and (c) where elevated approach structures were necessary, these were built of masonry with many openings so as to minimize disturbance to the flow of water.

A significant hurdle to the more general use of these bridges is the fact that little is currently known of the strength of the material or of the specific structural behaviour. It is the purpose of this paper, and the work described in it, to study these aspects of this type of bridge structure.

Factors influencing the structural design

Before embarking on a careful study of the structural behaviour and

material properties of stone masonry bridges, it is first necessary to assess the various factors that will influence the structural design. The loading on the bridges will be determined primarily by the traffic conditions and side pressures from flood waters, with possible foundation settlement as an additional consideration. The size and layout of foundations must be considered. The major influence on material strength will result from the construction methods employed. These are all considered with reference to previously published work.

Flooding and foundation conditions

Since many of the lessons learned by the Rhodesians (Mainwaring and Petzer, 1988; Mainwaring and Hasluck, 1985; Wooton and Stephens, 1987) were brought about by destruction of their initial attempts during floods, and since many of their successful causeways are still standing today, it is worth quoting their practical recommendations:

1. Ideally, submersible structures should be founded on bedrock. Founding them on unstable silty material is dangerous, owing to the tendency for river bed material to become liquid up to a depth in excess of 2 m (Mainwaring and Hasluck, 1985).
2. The construction of inclined buttresses on the upstream side of each pier was found to be effective in deflecting floating debris over the top of the bridge deck, thereby preventing the waterway in the structure from becoming blocked (Mainwaring and Hasluck, 1985; Wooton and Stephens, 1987).
3. Approaches should be made either in solid masonry, which must be founded on rock, or on 2 m deep raft foundations, or alternatively may be made in cut. Fill approach embankments should never be permitted or they may be washed out by the first major flood (Mainwaring and Hasluck, 1985; Wooton and Stephens, 1987).
4. Avoid using a skew crossing, since this tends to channel the full force of the river towards one of the river banks, and the chance of this approach being washed away is considerably increased. Rivers have been known to cut a completely new channel around a bridge structure, and remain permanently realigned, necessitating the construction of a new bridge (Mainwaring and Hasluck, 1985).

Often it is not possible, or indeed economic, to found structures on bedrock, and if liquefaction of the river bed can be prevented by the construction of a stone floor interconnected with upstream and downstream cut-off walls, then there may be justification for the use of bases above bedrock level. The current structures are thus founded on bedrock where possible, but in the absence of bedrock, bases are built under each pier from stone and mortar. These are arranged in such a way as to minimize the danger of undermining by the scouring action of the water during floods, and also to provide the required bearing area, so as not to exceed the bearing strength of the river bed, as is shown in Fig 1.

To guard against scouring of the foundations, where these structures are not founded on rock, a grouted stone floor is laid on top of the river bed from one abutment to the other, terminating at cut-off walls on the leading and trailing edges. The upstream cut-off wall built along the leading edge is at least 500 mm deep, while the downstream cut-off wall should be built to a depth below which liquefaction of the river bed material is deemed unlikely.

The structures are preferably built during the dry months when most of the rivers contain little or no water. This reduces the risk of flooding during construction and also permits excavation in the river beds, which is executed by hand.

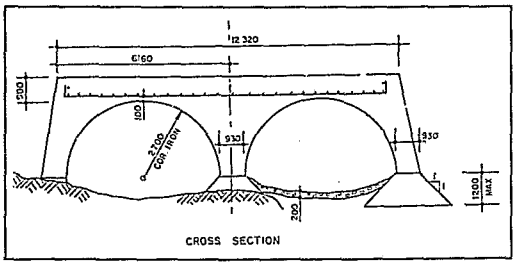


Fig 1: A cross-section of a typical stone arch bridge. The abutment on the right-hand side is founded on a base because suitable bedrock could not be found.

Flooding of this type of bridge will cause significant lateral loads on the structure, as demonstrated for example by the collapse of an arch bridge after very heavy local rain reported by Konishi(1987). The lateral thrust on the bridge from flood waters must thus also be considered as an important structural loading condition. TMH7(1981) gives specific guidance regarding load factors and combinations of this load with other loads. Flood loading is defined as a transient load so that it must be considered in combination with self weight, but not with heavy traffic loads. Information about the actual values of flood water pressures must be sought from other sources, such as Konishi (1987), or from an assessment of the water depth such as that given by the Department of Transport guidelines (CSIR, 1993a).

The construction method used

Grobler (Grobler et al, 1995; Grobler, 1994) has described the bridge construction, but some of the main procedures are summarized here for completeness. Once the formations are in place, sheets of corrugated iron are pre-cranked to the shape of the intrados and positioned over the foundations to provide temporary support to the arch span. In the case of large radii, these sheets are propped with gum poles to help bear the weight of the masonry until it becomes self-supporting. In some areas, alternative centring using low grade timber or 50 mm saplings supported by temporary masonry supports has been successfully utilised. Such centring costs only 25 per cent of the corrugated iron alternative, and all the money is spent locally, but a much rougher finish results, which may not always be acceptable.

Materials used: Workers contracted by local truck owners gather stones (typically weathered dolerite, decomposed granites and quartz vein, at a rate of at least 1 m³/labourer/day) in the fields nearby and load them onto trucks. Payment to the contractor takes place after measurement of the volume of rock delivered. The truck owner in turn pays his stone collectors.

Sand from the river bed nearby is used to make a cement mortar. Cement is bought from local dealers who are prepared to deliver small quantities, thus reducing storage and theft problems on site. In the dry season, water is transported in 200 ℓ drums, also by subcontractors. The cement:sand ratio varies from 1:4 for most of the construction to 1:3 for the outer faces exposed to flood scour action. The 1:4 mix is achieved by combining one pocket of cement with 2,5 wheelbarrows of sand, and the 1:3 mix uses one pocket of cement to 1,8 wheelbarrows of sand.

The mortar is hand mixed by the workers. The mix is prepared in such a way as to achieve a consistency that is just fluid enough to fill the interstices between the underlying stones. This minimum quantity of water does, however, appear to lead to the water content being slightly higher than that which would achieve optimum strength. The only materials imported from afar are reinforcing steel and some corrugated iron, which constitute approximately five per cent of the total cost of the completed structure.

Construction procedure: The manual assembly of the bridge begins by placing individual stones upon a thick bed of mortar. The stones are not cut, dressed or cleaned in any way whatsoever, but any stones with adhering soil are rejected. They are simply placed dry on the bed of mortar. Stones that are flat, or long and slender, tend to be placed horizontally rather than vertically, because gravity makes this practice more convenient. Some effort is made to place the stones in such a way as to minimize the size of interstices between them, but at a rough estimate 30 per cent of the volume of the structure is made up of mortar.

This procedure is followed, horizontal layer upon horizontal layer, until a solid structure results. The exposed masonry work on the sides of the bridge is built and cleaned very carefully at a rate that started at about 0,2 m³/labourer/day, but quickly increased to 0,3 m³/labourer/day, while the inner fill masonry, which is not visible, is built at a faster rate, which started at about 0,3 m³ to 0,5 m³/labourer/day, but increased to 0,6 m³/ labourer/day.

As each layer hardens it becomes somewhat self-supporting, so that at no time does the formwork support the entire volume above. This is evidenced by the photograph of a bridge under construction in Fig 2. It is not deemed necessary or economical to cure this masonry, as only a proportion of grout area is exposed to evaporation for a short duration (until the next layer is placed, which is usually overnight and which seldom exceeds 65 hours over weekends).

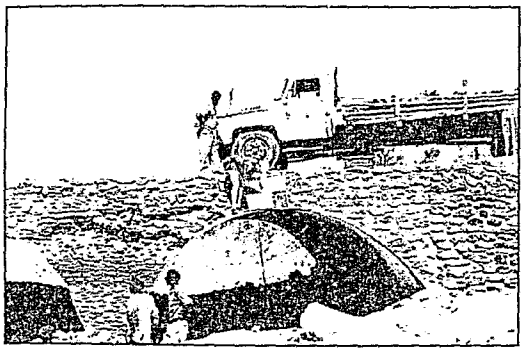


Fig 2: The layers arising from construction of the bridge are clearly visible in this photograph

As a precaution, a grid of high tensile reinforcing steel has been placed horizontally in both directions, 100 mm above the crown of the arch in the initial structures. This steel is placed in a thick bed of cement mortar, of a 1:3 cement:sand mix, without any coarse aggregate whatsoever. Another bed of mortar (or preferably concrete) is used to even out the deck of the bridge to form a trafficable surface.

Where approach embankments are necessary, these are made with hand compacted (or preferably mechanically compacted) earth fill and protected from erosion by a grouted stone pitching.

Traffic loading

An assessment of appropriate standards for rural roads (CSIR, 1993a) has considered the traffic that is likely to use the rural roads for which these stone masonry bridges are used. The recommendation is made that NA and NB24 vehicle loads, as defined by TMH7(1981), should be used. The maximum vertical loads that the bridges are required to withstand are thus 16 wheel loads of 280 kN each, consisting of four axles in two pairs. Each pair of axles is separated by 2 m, and each axle has four wheels spaced 1 m apart across the bridge.

Requirements for improved knowledge

Knowledge required

The geometric layout of these structures is designed in accordance with Department of Transport guidelines (CSIR, 1993a, 1993b), but owing to the present limited knowledge of the material characteristics and internal stresses, the structural design is done using simplifying assumptions for analysis and empirical material strength data. In the structural analysis of the bridges that have been constructed in the Northern Province, the consulting engineers modelled the bridges as equivalent frames (Pienaar, 1994) and the design philosophy aimed at achieving a high factor of safety by placing an upper limit of 1,0 MPa on internal stresses. The material used in these structures differs from traditional stone masonry arch bridges in that the stone used is not dressed in any way whatsoever, but is placed randomly in a mortar matrix. There is no published information regarding the strength and behaviour of this type of stone and mortar matrix.

If the potential of these bridges is to be further exploited, it is vital that knowledge of the stone/mortar construction material should be enhanced. Four specific areas have been identified:

1. Determination of the strength of the material as well as gaining an understanding of its behaviour under load. This is necessary so as to make recommendations regarding the most appropriate stone type, shape and size, as well as the optimum orientation of individual stones in critical regions.
2. Exploring alternative shapes of arch openings so as to increase the possible spans and load carrying capacities while reducing the material content and hence cost. Consideration of tolerances to geometric inaccuracies, such as the exact arch shape and road level, and the effect of formwork deformation will be important for defining labour skills and construction procedures required.
3. The analysis of stresses in critical regions of different arch shapes will aid the specification of material strength and any reinforcing steel that may be required. The strength implications of any cracking that may occur are also important.
4. Definition of a design procedure and construction guidelines that are consistent with the goal of job creation and cost minimization.

Pilot research study

A pilot study has been completed, in which the in situ stresses and deflections were measured in a bridge and large cubes of the stone masonry material were tested.

In situ measurements: In order to gain an initial understanding of the magnitude and nature of the stresses and strains within the arch structure, two calibrated stress measuring devices (hereafter referred to as stress cells) were built into an arch bridge during its construction. These stress cells were made by fixing electronic strain gauges onto concrete prisms in such a way as to preclude the influences of temperature fluctuations and induced bending strains. The first stress cell was placed horizontally as close as possible to the intrados at mid-span and the second stress cell was placed halfway between the first stress cell and the foundation, inclined at 45 (see Fig 3). Both stress cells were placed parallel to the longitudinal centreline of the bridge, which was the anticipated line of thrust of the arch.

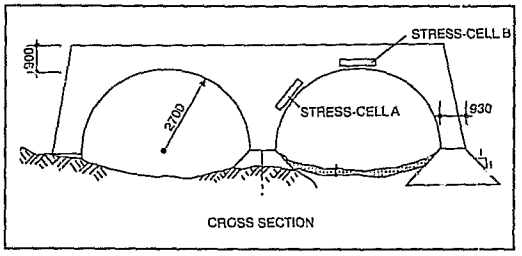


Fig 3: A cross-section of the stone arch bridge used in the pilot study showing the position of the recording devices

At a stage when the bridge became trafficable, a lorry loaded with a payload of rock was driven over the structure so that the stresses in each stress cell at various axle positions could be recorded (see Fig 4). The axle load applied to the structure was measured with the aid of a portable weighbridge, capable of weighing one wheel at a time. At an axle load of 100 kN (ie 5 t under each wheel), the stresses induced by live loads (excluding self-weight stress) recorded by the stress cells was less than 0,1 MPa. Fig 5 shows the stresses induced by live load and the corresponding axle positions. It was observed that it was of little consequence whether one of the wheels was directly over a stress cell, or whether the centre of the axle was directly over a stress cell. These measurements of live load behaviour are currently being used to check the validity of a finite element analysis computer simulation, which will be used to compute more complex load conditions and to analyse different geometrical arch shapes.

Unfortunately, with the very low stresses occurring, it was not pos-

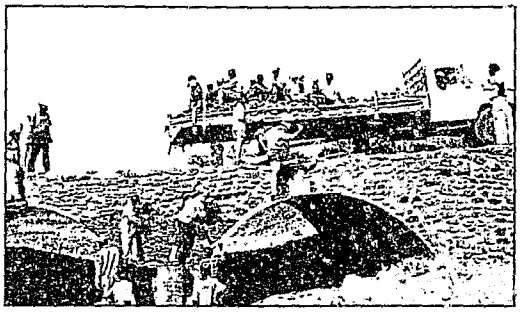


Fig 4: Live load induced strain test

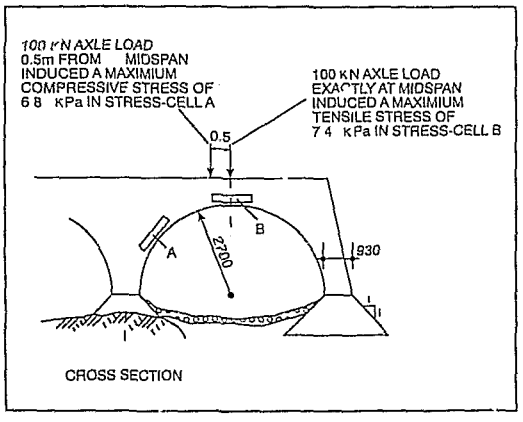


Fig 5: Recorded induced stresses in a 2,7 m radius arch with corresponding axle positions

sible to obtain meaningful measurements of the stresses due to self-weight. The preliminary finite element results indicate a self-weight stress of 0,6 MPa. Once the finite element model has been fully calibrated to the live load measurements, it will be possible to confirm this level of stress.

Deflections of the arch were measured with a highly sensitive plunger type dial gauge mounted mid-span against the crown of the intrados upon a tripod. The 100 kN axle load resulted in a deflection of 0,040 mm, which indicates the very stiff nature of these structures.

Material strength: The strength of the stone-mortar matrix used as building material is difficult to predict theoretically and it is also difficult to measure, since SABS 863 (1976) makes no provision for inclusion of aggregates larger than 75 mm. Numerous authorities (Walker and Bloem, 1960; Walker at al, 1959; Higginson, 1963) have reported the general trend that for a given cement:water ratio, compressive strength decreases as the maximum size of coarse aggregate is increased. However, no numerical information is available to deduce the quantitative effect of this phenomenon for the particular building material under consideration.

In order to establish, as closely as possible, the strength of this material above the crown of the arch, some very large cubes were fabricated on site using the same materials and the same labour force as were used to build the bridge structure. A total of three of these cubes, of sizes 300 mm, 400 mm and 500 mm, were cast to represent typical crown thicknesses. No cubes were made larger than 500 mm because their size and weight would have precluded handling, transportation and crushing with the available resources. Cube moulds were made from block-board and oiled with linseed oil. After standing in the open for 48 hours, the large cubes were transported to the University of the Witwatersrand in their moulds. In addition, three standard cubes 100 mm in size were made of the 1:4 mortar. For these tests, the cubes were cured in a water bath for 28 days, and the measured strengths were 15,73 MPa, 14,85 MPa and 14,21 MPa. This indicated that the strength of the typical mortar used averaged 14,93 MPa. An analysis of the proportions of sand and cement in a sample of the mortar showed approximately 1 part cement to 6,4 parts sand.

The large stone masonry cubes were crushed on a 500 t Amsler press, housed in the Mechanical Engineering Laboratory at the University of the Witwatersrand. Load was applied perpendicular to the plane of casting, so as to simulate the compressive stresses generated horizontally over the crown of the arch.

The large cube data and test results are reported in the accompanying table. All three cubes failed at a stress below the mortar strength, with a wide scatter of results. The larger two cubes failed at a low stress of less than two thirds of the mortar strength. Failure of the specimens appeared to be initiated by bond failure at the mortar stone interface, which resulted in a wedging or cleaving action by the long slender stones that lay parallel to the axis of applied load (see Fig 6). This orientation of the stones (with their longest axes parallel to the compressive thrust) was deliberately chosen, since it closely approximates the stressed condition above the crown of the arch where the compressive thrust forces act horizontally, in line with the longitudinal axes of the stones which are currently placed horizontally.

Results of compressive tests on stone-mortar cubes

Cube size (mm)	Mass (kg)	Density (kg/m³)	Failure load (kN)	Failure stress (MPa)
300	59	2 222	1 260	14,0
400	149	2 328	1 435	9,0
500	-	-	2 406	9,6

Continuation of research

This pilot study has established a practical procedure for obtaining material strength and measuring internal stresses in these structures, but was very limited in the number of tests conducted. A continuing research programme is envisaged, in which further similar tests and a more detailed investigation of other structural factors will be included. The main immediate aims are as follows:

1. A much more comprehensive testing programme on large cubes. At present a further 15 cubes, 500 mm in size, are in preparation, to explore the orientation of stones and the effects of stone size, and to gain a better understanding of the likely variability of the material strength.
2. Further in situ tests are planned, to improve our database of information on the actual field behaviour of full size bridges.
3. The finite element modelling will be continued to study a range of different arch shapes, larger spans and the effects of imperfections and cracks that might develop.

An interesting parallel study is to investigate other structures for which this uncut stone masonry material has applications. For example, Shaw (1994) discusses its use in the construction of dams.



Fig 6: Large-scale compression test on a 500 mm stone mortar cube

Conclusion

This labour-intensive method of bridge construction enhances the goals of the National Public Works Programme and in particular the Reconstruction and Development Programme more than any existing small bridge construction technique.

Results of a limited pilot study to measure stresses and strains in a completed structure and determine the strength of the stone-mortar material indicate good potential for profitable research. The groundwork for investigating different geometrical arch shapes so as to increase the possible spans and load carrying capacities while reducing the material content and hence cost, is well under way. A need for further research into the material properties, including strength sensitivity as a result of substituting different geological materials, has also been established.

Provided the bridge location is in close proximity to a source of suitable stone, there are thousands of communities throughout Southern Africa that could benefit from the building of these structures.

Acknowledgements

The co-operation of African Consulting Engineers Inc and of the Department of Public Works of the Northern Province in providing the bridge structures for testing is gratefully acknowledged. Financial support for the pilot study was provided by the Department of Building at the University of the Witwatersrand. The portable weighbridge device was borrowed free of charge from Mikros Systems. The large-scale testing was made possible by the willing assistance of Dr Chandler, Mark Oelofse, Tokie Muller and Gerhard Harmse, all of the School of Mechanical Engineering at the University of the Witwatersrand.

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Discussion on papers

Written discussion on the technical papers in this issue will be accepted until 31 October 1995. This, together with authors' replies, will be published in the First Quarter 1996 (March) issue of the *Journal of the South African Institution of Civil Engineers,* or the issue thereafter. For the convenience of overseas contributors only, the closing date for discussion will be extended to 30 November 1995. Discussion must be sent to the Directorate of SAICE.

Such written discussion must be submitted in duplicate, should be in the first person present tense and should be typed double spacing. It should be as short as possible and should not normally exceed 600 words in length. It should also conform to the requirements laid down in the 'Notes on the preparation of papers' as published on the inside back cover of this issue of the *Journal.*

Whenever reference is made to the above papers this publication should be referred to as the *Journal of the South African Institution of Civil Engineers* and the volume and date given thus: J SA Inst Civ Eng, Vol 37, No 3, Third Quarter 1995.

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Structural aspects of labour-intensively constructed, uncut stone masonry arch bridges

The original paper is by R G D Rankine, G J Krige, D Teshome and L J Grobler

A labour-intensive technique for constructing arch bridges using uncut stones and mortar was recently used very effectively. However, at present little is known about the strength and behaviour of the material or the structural form. This paper considered other work that had been done to define the information required to quantify the structural performance. A pilot study was described, in which large size cube tests were used to measure the compressive strength of the stone-mortar matrix. This indicated that the composite strength is lower than the mortar strength, and also that the orientation of the stones is important. A method of measuring the live-load induced stresses in these structures was described and the measured stresses were discussed. Finally, an outline was sketched of further research being undertaken.

This technical paper was originally published in *Journal* 37 (3), 1995.

Rod Rankine is a lecturer in the Department of Building at the University of the Witwatersrand.

Geoff Krige is an associate professor in the Department of Civil and Environmental Engineering at the University of the Witwatersrand.

Dellelegne Teshome is a senior lecturer at the University of Durban-Westville.

Louis Grobler is with African Consulting Engineers in Pietersburg.

Ian Glauber

I read this paper with great interest and applaud the efforts being made in this direction.

Below a certain size of coarse aggregate in the mix, the criterion for failure, other things being equal, is the strength of the mortar, determined in simplified terms by the cement/water ratio.

However, when the size of coarse aggregate is as described, the limit of strength, other things being equal, will be determined by the bond strength of the components.

By virtue of:

1. The cost of cement
2. The presumed lack of abundance of water, and
3. The absence of a need for workability

there is the probability that the minimum possible quantity of water will be used in the mix. When the dry stones are placed on the bed of mortar, there is the inevitable absorption of moisture from the mortar into whatever porosity exists in the stone. The mortar immediately contiguous with the surface of stone could thus be dehydrated and the cement at this interface could readily be under-hydrated for the development of the optimum strength in bond that is the determinant of the strength of the structure.

I suggest, therefore, that consideration should be given to requiring the procedure to include the submerging of each stone in water immediately prior to it being placed on the mortar bed. This would prevent the water that has been provided for the hydration of the cement being drawn out to fill any pores and cracks in the stone.

Parallel tests on 500 mm cubes using this procedure compared with the stones placed dry could show interesting results.

Authors' reply

In response to Mr Glauber's comment on the parameters governing the strength of rubble masonry, we agree that mortar strength and bond strength are both very significant factors contributing to the overall strength of the composite. However, it must be stressed that the 'chain is as strong as its weakest link' philosophy does not apply here and therefore ultimate failure is unlikely to be governed by the mortar strength or the bond strength alone.

Stones with large flat faces oriented perpendicular to the axis of applied load appear to provide bilateral constraint to the dilating matrix and therefore contribute strength, whereas stones placed with their longest axis parallel to the axis of applied load tended to cleave the matrix. Where stones come into intimate contact with one another (as is typical in rubble masonry), extremely high local stresses develop at the stone-to-stone interface and often result in premature failure at very small average stresses. It is interesting to note that this phenomenon has been well recognized for centuries. Ken Follet in his novel *Pillars of the earth* (a meticulous account of the construction of medieval stone cathedrals) explains the necessity of providing a bed of mortar between stones in order to prevent these high stone-to-stone contact stresses from developing and possibly fracturing a stone. Ultimately, it is our intention to write a comprehensive technical paper dealing specifically with the parameters that govern strength of rubble masonry. In the mean time we recommend the following references from rock mechanics sources for immediate explanations of the mechanics governing similar composites with stiff inclusions:

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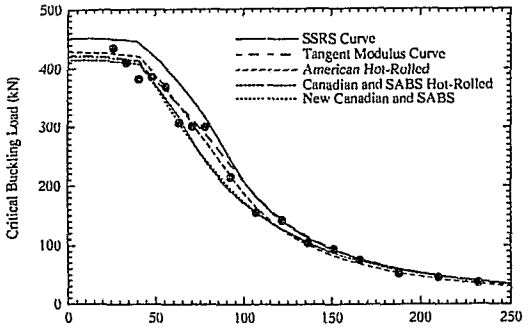

Fig 3: Critical buckling loads for angle columns

Conclusion

Type 3CR12 steel shows gradual yielding behaviour with a high proportional limit. The experimental results therefore agree with the theoretical predictions proposed by the South African, Canadian and American design specifications. The theoretical predictions, when the SSRC approach is used, overestimate the strength of Type 3CR12 steel.

It is concluded that although acceptable prediction of compact column behaviour may be obtained through the use of the tangent modulus formula, the equations used by the American (AISC, 1986), Canadian (CSA, nd) and South African (SABS, 1993b) hot-rolled design specifications should be investigated further. Research by Van der Merwe (1987) and Van den Berg (1988) concluded that the tangent modulus approach should be used to predict the strength of stainless steel and Type 3CR12 steel structural members.

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Discussion continued from page 22

In response to Mr Glauber's suggestion of wetting each stone prior to placement on the bed of mortar, I am in partial agreement but for a different reason. His recommendation is valid for masonry units, such as clay bricks, which have a tremendous propensity to consume large quantities of water, thereby depleting the thin mortar bed of its water necessary for hydration. I would agree fully with this precaution if porous stones (such as sandstones, shales or very weathered dolerites) were to be used for rubble masonry. However, to date engineers have tended to avoid such porous rocks owing to their inferior mechanical properties and have favoured stronger and less thirsty igneous material.

Wetting stones has another benefit. Any loose dust or other contaminants on the surface of the stones are removed. The larger the aggregate particles, the lower the surface area to the volume ratio and the greater the bond stress at the stone/mortar interface. Hence the importance of cleanliness is proportional to stone size. The Maritsane Dam currently under construction in Mpumalanga is being built from freshly quarried granite. The engineer specified that all stockpiles of rock be high-pressure blasted with water, immediately prior to bonding, to ensure cleanliness and good bond. It is interesting to note that on a hot day, these stones dry completely during the time that they are transported by wheelbarrow from the wash site to the structure.

Discussion on papers

Written discussion on the technical papers in this issue of the *Journal* will be accepted until 31 July 1996. This, together with the authors' replies, will be published in the Fourth Quarter 1996 (December) issue of the *Journal*, or the issue thereafter. For the convenience of overseas contributors only, the closing date for discussion will be extended to 31 August 1996. Discussion must be sent to the Directorate of SAICE.

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APPENDIX 2

DEVELOPMENT OF A COMPRESSIVE STRENGTH TEST FOR RMC

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R G D Rankine

Development towards a proposed compressive strength test for rubble rock masonry

Synopsis

Knowledge of the strength and failure characteristics of rubble rock masonry has been identified as a prerequisite to the widespread adoption of its use in RDP projects. Traditional 100 mm and 150 mm cube tests cannot accommodate even the smallest stones used in this composite and are therefore not appropriate. Lack of an existing large-scale testing procedure necessitates innovation of a reference test. Evolutionary research work towards a reference testing procedure for quantifying the unconfined compressive strength of rubble rock masonry is described. A 500 mm specimen cube size was found to be the most realistic compromise between being representative of this material and manageable. This specimen size is proposed for adoption as a reference test for making meaningful comparisons between series of specimens. Some important parameters found to influence the strength are identified and practical steps to control them are discussed.

Samevatting

Kennis van die sterkte en falingeienskappe van ruwerotsbeton is 'n voorvereiste vir die meer algemene gebruik daarvan in HOP-projekte. Die gebruiklike 100-mm- of 150-mm-kubusse is te klein, aangesien die ruwe aggregaat dikwels van groter afmetings is en sulke kubusse dus nie verteenwoordigend sou wees nie. Die gebrek aan 'n aanvaarde tegniek vir die toets van groter monsters het vereis dat sodanige toetse ontwikkel moes word om 'n gestandaardiseerde verwysingstoets daar te stel. Hierdie verhandeling beskryf die ondersoeke wat gelei het tot 'n tegniek om die sterkte-eienskappe van groot kubusse te bepaal. Daar word dan ook voorgestel dat kubusse van 500 mm aanvaar word as die standaardgrootte vir die karakterisering van die eienskappe van hierdie tipe beton. Belangrike parameters wat die sterkte van sodanige beton beïnvloed word ook uitgewys en voorsorgmaatreëls om hulle te beheer geïdentifiseer.



Roderick Rankine *obtained his BSc (Bldg) and MSc (Civ Eng) from the University of the Witwatersrand (Wits) in 1988 and 1991 respectively. He served in the South African Navy in 1990 and then worked on an oil rig on the Moegas project in 1991. He is now on the academic staff at Wits, where he is engaged in research towards his PhD, which includes quantitative analysis of the properties of rubble masonry for use in structures to be built with labour-intensive methods as a means of employment creation.*

Introduction

The use of a building material (which is described more comprehensively in Rankine et al, 1995) consisting of uncut stones bound together with cementitious mortar has aroused a great deal of recent interest as a means of enhancing employment opportunities (Rankine, 1995). This material, dubbed bastard concrete (Schuyler, 1888), uncut stone masonry (Rankine and Krige, 1996), masonry concrete, cyclopean concrete (Shaw, 1994) or rubble rock masonry (De Beer, 1995), has already been successfully used for the construction of many small dams and bridges (De Beer, 1995, 1996; Grobler et al, 1995; Labour-intensive me[illegible]1993; Le Roy, 1996; McAllister, 1996; Rankine and Krige, 1996; Shaw[illegible], 1996).

Until now, all design has had to be empirical, for want of reliable data concerning the strength and failure characteristics of the material, and engineers have had to limit allowable stresses to very conservative values (typically around 1 MPa to 2 MPa). At least one authority (Krige, 1995) has recognized the importance of defining the material's strength and mechanical properties as being 'prerequisite to structural engineers widely adopting this initiative'. Practising engineers who were using rubble masonry at the start of this investigation expressed initial curiosity regarding the material's compressive behaviour as it is usually used in arch structures where the critical load case is often compressive.

Search for an existing standard test

A comprehensive literature search (SABS Method 863, 1976; Walker and Bloem, 1960; Higginson, 1963; Compressive tests, 1967; Rajendran, 1965; Krishna Raju and Basavarajaiah, 1979) was conducted to explore the possibility of adopting some precedent as a basis for comparisons. Walker and Bloem (1960) and the United States Bureau of Reclamation (Higginson, 1963) used large-scale cylinder tests (up to 48 inches [1 200 mm] long by 24 inches [600 mm] in diameter) in an effort to explore the effect of maximum size of aggregate (up to six inches [150 mm]) upon the compressive strength of concrete used in dams, such as Hoover Dam. They concluded that for a given cement:water ratio, strength is inversely proportionate to maximum aggregate size.

The South African Bureau of Standards (Compressive tests, 1967) used 16 inch [400 mm] concrete cubes in 1967 to evaluate the strength of concrete used in the construction of the Naute Dam in Namibia, which contains aggregate of up to four inches [100 mm]. Rajendran (1965) and Krishna Raju and Basavarajaiah (1979) investigated the effect of cube size (up to 255 mm) upon the compressive strength of concrete and concluded a general trend indicating that the cube strength is inversely proportionate to size and that this phenomenon is more pronounced in higher strengths (30–40 MPa) of concrete.

Unfortunately, none of these sources gave adequate details of their testing procedures and the tolerances required to ensure acceptable consistency. Hence, work was undertaken at the University of the Witwatersrand to develop an appropriate compression test.

Evaluation of the suitability of cylindrical test specimens

Despite the availability of large-diameter thick-walled polyethylene pipes as moulds, three factors resulted in rejection of a cylindrical test shape:

1. Any bleedwater, drying shrinkage and projecting stones cause unevenness of the top casting surface, and it is upon this surface that axial test load must be applied. If load is applied to such an uneven surface, very large local stresses develop around the high spots, resulting in premature failure. The only way to overcome this problem is either to grind the faces flat with very sophisticated machinery or to cap the faces. Both solutions necessitate considerable expertise, skill and expense.
2. Despite the excellent releasing properties of polyolefines, it is very doubtful that cast specimens could be released from one-piece moulds without a taper. This means that moulds would either have to be cut open to release the specimens (which is expensive as it precludes reuse) or that moulds would have to be purpose-made in two halves (which would be very expensive).
3. The availability of a constant supply of a tube with a particular diameter in the future cannot be guaranteed.

Cubic test specimens

Cubic specimens on the other hand have four flat faces. This allows the user to select the best of two pairs of faces through which to transfer axial load. Provided the mould faces in contact with the platens are plane and true, they are automatically ready for testing as soon as the specimen is released from the mould.

Cubic specimens also make optimal use of the square platen area available on the press, thereby capitalizing on the potential for bigger, more representative specimens. It was therefore decided that cubic test specimens were more appropriate than cylindrical specimens. What remained uncertain was their optimum size.

Development of an appropriate test for rubble rock masonry

The following limiting parameters were identified as important to the success and future acceptance of the testing procedure:

1. The cost of obtaining test specimens should be as low as possible, so as not to discourage the generation of new data.
2. The procedure for specimen preparation on site should be simple enough to be undertaken with minimal supervision by the same unskilled labour force as is deployed on actual structures.
3. The test specimens should be large enough to be representative of the composite material, yet light enough to lift manually and transport safely on a one ton vehicle.
4. The test specimens should be able to be accommodated between the square platens of a large hydraulic press.
5. The axially stressed area of the test specimens should be limited by the maximum load capacity of the press to guarantee destruction of all likely material.

Determination of an optimum specimen size

Initially, cubes of 300 mm, 400 mm and 500 mm were chosen as a means of exploring different possible sizes (Rankine, 1995; Rankine et al, 1995). The 300 mm size proved too small to accommodate any but the very smallest stones and the 400 mm size permitted one, or at the most two, of the larger stones. The need for large aggregate accommodation thus favoured the largest possible cube size. The 500 mm size was subsequently adopted for the following reasons:

1. This size of specimen appears to border on the threshold of what it is possible to lift manually. Six labourers are able to share the load of approximately 300 kg, each lifting 50 kg (see Fig 1). Heavier specimens would undoubtedly require mechanical lifting.
2. A minimum of three specimens are needed to ensure a representative statistical average. Three 500 mm rubble masonry cubes have a combined weight of about 900 kg, which makes optimal use of a typical light delivery vehicle with a 1 000 kg load capacity.
3. The crown of many typical low level rubble masonry arch bridges is of the order of 500 mm deep and a 500 mm cube is therefore not unrepresentative in mimicking this load condition.

Fabrication of cube shuttering

There is little doubt that machined cast iron is probably the ultimate material for fabrication of accurate and durable cube moulds. However, the fabrication costs of a 500 mm cast iron cube mould would most certainly exceed most budgets and the weight of such an article would definitely preclude manual lifting of the mould and its contents.

The immediate challenge was to develop a technique to fabricate moulds of adequate tolerances using available materials and skills at a minimum of cost.

The idea of using pressed plate steel formwork panels was rejected because they are not readily available in the desired size and because their surfaces are not flat. Surprisingly, good timber boarding was found to be far flatter and stiffer; probably as a result of its thickness. Twenty millimetre thick board proved ideal, as the sides can be butt-jointed with wood screws without additional bracing to form a stiff box.



Fig 1: A 500 mm rubble masonry cube specimen weighs approximately 300 kg and can be manually lifted by six labourers (a 400 mm and a 300 cube can be seen on the right)

Suitable materials

The two most important requirements of the material are that the sheets must be flat (truly planar) and that the glue used for their lamination must be waterproof. Materials sold as 'shutter-ply', 'shutter-board' and some 'block-boards' have proved adequate. Materials sold as 'block-board' may vary considerably in quality. If the material can be delaminated by hand or by exposure to water, it should be rejected. Waterproof chipboard, unless extremely thick, is unsuitable as its bi-directional fibre orientation may provide insufficient stiffness to resist the bulging of the sides induced by hydrostatic pressure during casting.

Mould construction

Three 500 mm cube moulds can be made from two boards (2 400 mm by 1 200 mm by 20 mm), as shown in Fig 2. The boards should be cut on a table and/or radial arm saw, exercising utmost care to ensure that cuts are

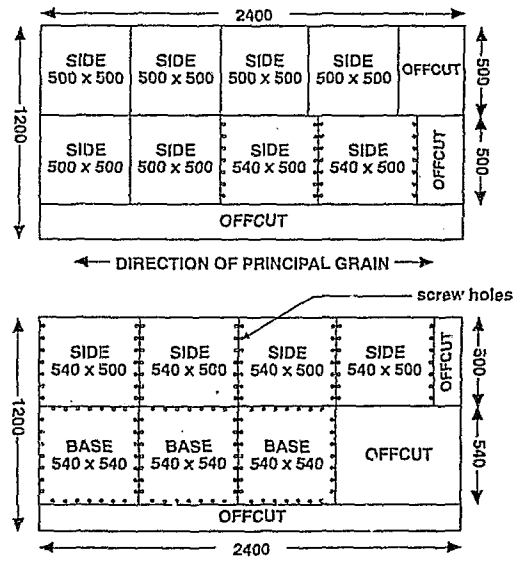

Fig 2: Cutting plan of two boards (2 400 mm by 1 200 mm by 20 mm) for fabrication of three 500 mm cube moulds

straight, parallel and perpendicular. Consideration should be given to the direction of the principal grain to derive its maximum stiffness along the exposed edge of the top of the mould. By exercising a reasonable degree of care and correct shop procedures, a tolerance within ±1 mm can be consistently achieved. The most critical tolerance is the flatness of the two mould faces that form the sides of the cube that will be in contact with the press platens.

Procedure for casting

- All surfaces of the moulds should be coated with an undercoat of raw

linseed oil and the inside faces coated with an additional top coat of shutter oil release agent.

- The moulds must be placed on top of a 50 mm to 100 mm blinding layer of loose sand to minimize distortion as a result of being supported by uneven ground.
- Moulds must be filled with material by the same labour-force employed to build actual structures, using exactly the same method of placing.
- Stones must not be allowed to project more than 20 mm beyond the top of the mould.
- Once filled with material, the moulds must be covered with plastic sheeting to prevent loss of moisture.
- After 72 hours, the cubes may be carefully moved and transported, provided they remain in their moulds and are covered with plastic sheets.

Description of an appropriate test facility

The requirements of a suitable press are well documented (BS 1881, 1986; BS 1881, 1983; Dewar et al, 1971; Foote, sa; Sigvaldason, 1964; Cole, 1967; Newman and Sigvaldason, 1965) and should be thoroughly understood by anyone undertaking compression tests and interpreting such data.

The smallest suitable press should have a capacity of no less than 500 t, as all of this force is required for crushing specimens with a maximum strength of 20 MPa. To date, all the material tested (typical of that used in rubble masonry projects) has yielded at significantly less than 20 MPa.

A four-column press is preferable to a two-column machine (see Fig 3) since it is stiffer as the columns do not bend if one side of the cube yields before the other. It is conceivable that at near maximum load such bending might even plastically deform the columns of a two-column press if the specimen reaction is not symmetrical about the vertical axis.

One platen must be ball-seated and able to rotate in two dimensions. This arrangement facilitates a degree of tolerance to geometric inaccuracies in the cast specimens, particularly to opposite faces not being perfectly parallel. Unlike the ball-seating in most conventional small concrete cube presses (which typically facilitate early movement of the top platen), larger presses often have their ball-seating on their lower platens and rigidly fixed top platens (see Fig 4). This makes little difference other than slight awkwardness in loading.

A mechanical coupling (a standard part of the rig employed) from the lower platen to a drum plotter via a length of trace fishline was used to record deflections (see Fig 4). However, the relaxation of the hardboard packing coupled with the inherent memory and instability of trace fishline made the load-deflection graphs questionable. In addition, there is always a noticeable 'jump' as the reaction against the upper platen equals the weight of the head of the press and displaces the backlash in the machine threads against the columns. A system that uses electronic strain measuring devices embedded into the cubes, the sensitivity of which can be manipulated mechanically (Rankine, 1996), will in future be used to determine the material's elastic modulus and Poisson's ratio.

Platen/cube interface material

Traditionally, packing between the cube faces and the machine platens with any material whatsoever has been prohibited in all standard cube tests (Dewar et al, 1971). Contrary to this conventional wisdom, packing is recommended between the rubble masonry cube faces and the machine platens for two reasons:

- To prevent possible damage to the platens of the press as a result of the very high point stress concentrations anticipated where large rocks make contact with the cube faces. These stresses are bound to be higher than those encountered in small concrete cubes, since the number of contact points per unit area of platen face is substantially reduced as an exponential function of aggregate size. This reduced contact area is also much more unyielding as it is backed by a few massive stiff stone inclusions rather than by a soft mortar matrix.
- Although the timber boarding produced surprisingly plane cube faces, it is not as stable as cast iron. Erntroy (1964) has shown how a platen with a convex departure from planeness of 0,1 per cent can cause a 10 per cent reduction in measured cube strength, the 'point-load' acting in a somewhat analogous manner to the indirect tensile splitting of cylinders. It is therefore speculated that departure from planeness of the rubble masonry cube face would also negatively alter the strength. Very great variation in stress across the cube with high points having to deflect excessively before bearing on the hollow regions can occur.

Initially, 6 mm 'Masonite' hardboard was chosen to protect the faces

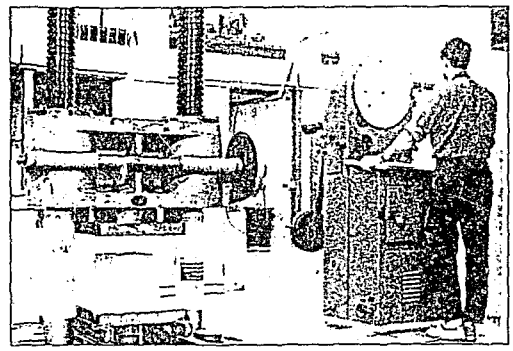

Fig 3: The 500 t Amsler press used to test rubble rock masonry cube specimens. The cube on the lower platen is 300 mm long.

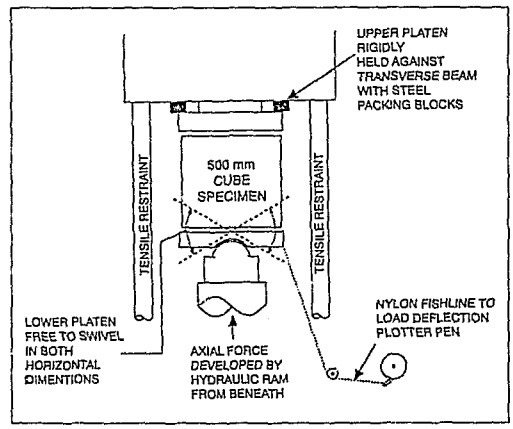

Fig 4: Diagrammatic representation of the loading arrangement on the 500 t press used to test the rubble masonry cubes

and to provide some cushioning to equalize stress variations across the cube:platen interfaces (consequent upon any geometric deviation from planeness). On its own, the hardboard was observed to have three downfalls:

- The coefficient of friction between the cube face and the hardboard is far greater than that between the cube face and the steel platen. This phenomenon is clearly illustrated in Fig 5, where the dilation of a 500 mm cube of mortar is greatly constrained by a sheet of hardboard against its upper face, whereas its lower face in contact with the steel platen is relatively unrestrained. The classical symmetrical hourglass shape is very skewed. It might be expected that as the surface texture of the hardboard roughened with time this phenomenon would become even more pronounced.
- As the specimen was manoeuvred onto the rig, the hardboard tended to slide sideways off the lower platen (see Fig 6).
- Sudden dilation, following the fracture of a large stone, would sometimes result in tensile tearing and puncturing of the hardboard.

These problems were overcome by sandwiching the hardboard between the platen face and a sheet of 2 mm mild steel plate. The problem of the packing material sliding upon manipulation of the cube into the rig was solved by bending one edge of the lower mild steel plate downward at 90° to form a lip hard up against the platen as shown diagrammatically in Fig 6.

The 6 mm hardboard was later substituted with thicker lower-density packing material in an effort to enhance the cushioning effect. During failure, the sharp corners of the stones penetrated and dimpled the mild

steel plate. This is undesirable as it increases the constraining effect of the platens. Six millimetre hardboard is therefore proposed as a standard. The cushioning effect of 6 mm hardboard appears to be adequate.

Future users of this test are cautioned against the temptation to u[illegible] form of plated, galvanized or painted steel since these surfaces ine[illegible] have a lower coefficient of friction and therefore provide less restra[illegible] the dilating specimens than pure mild steel. Similarly, the practice of [illegible]ing the plates to prevent rust should also be condemned. Use only plain untreated mild steel.

The importance of concentricity

Entroy (1964) has observed that displacements of cube specimens of as little as five to six per cent from the true vertical axis of a press can produce a decrease in strength of as much as 10 per cent. Indeed, Rosenman (5a) has discovered the phenomenon to be even more pronounced in large rubble masonry cubes, recording a decrease in strength of as much as 36 per cent where 400 mm cube specimens were centred onto 500 mm packing plates by eye rather than by measurement. This increased sensitivity might well be explained in terms of the size effect principle, but it is more likely due to the relative size of the ball-seating. Larger presses often have a smaller ball-seating relative to the platen size than do small concrete cube crushers. This permits greater freedom of rotation of the 'mobile' platen and hence less symmetrical loading.

Speed of loading

In tests performed to date load was applied at the rate of about 120 kN per minute. This provided sufficient time to monitor deformations and cracking as well as for photography. (Visible deformation and cracking were frequently evident before maximum load). At this rate of loading, an average specimen can be tested to destruction within 30 minutes.

Test limitations

The objective of developing this testing procedure was to identify factors that can influence measured cube strength and propose ways to limit them so as to provide a basis whereby one series of results can be compared with another in an effort to enhance knowledge about rubble masonry. Several investigations (Barclay, 1964; Cole, 1964, 1966) have indicated that considerable variations in standard concrete cube strength can be obtained when similar concrete specimens are tested in different machines at different laboratories. Calibration error, stiffness of the loading frame and the characteristic of the ball-seating locking under load have been identified as the principal causes of such variations between different machines. Consequently, wherever possible, meaningful comparisons should be made only between specimens tested on one particular machine.

By exercising reasonable care in fabricating moulds and casting cubes with graded aggregates, results from a series of twelve 500 mm cubes (four sets of three batches) with coefficients of variation of between 1,03 and 7,03 per cent were obtained.

Future research

Once data has been gathered from large-scale strength tests performed on rubble masonry specimens containing a variety of different variables (such as mortar strengths, geological materials, stone sizes and surface conditions), these might be compared with the strength of 100 mm mortar cubes. From this analysis it may be possible to correlate the composite strength of rubble masonry with the mortar cube strength alone within defined parameters. Such knowledge may afford engineers a convenient reference for controlling ongoing quality once the composite's potential strength has been established.

Recently, several practising dam engineers have expressed the view that the tensile capability of rubble masonry should not be ignored as a limiting property and that it would be appropriate to develop a splitting test that is more directly governed by the composite's tensile strength. The vulnerability of masonry dams to progressive tensile failure has been revealed by case studies of incidents in India (Murthy et al, 1979), Australia (Nicol et al, 1970) and elsewhere (ICOLD, 1981). It is interesting to note that during construction of the Sukkur Barrage in India in 1925 the number of specimens tested for tensile strength comfortably exceeded the number tested for compressive strength. Among the reasons for this may have been a concern that the masonry should be ductile, perhaps to resist differential foundation settlement.

Conclusion

Improved knowledge of the strength and mechanical properties of rubble rock masonry has been acknowledged as a prerequisite to the widespread adoption of this material in RDP projects. Lack of an appropriate existing testing procedure as a basis for making comparisons has necessitated innovation. A simple standardized testing procedure has evolved, using timber board shuttering for a 500 mm cubic specimen. Simple specimen preparation instructions have been provided for the benefit of researchers with very limited resources. Some critical parameters that may influence the test results are identified and practical steps to limit them suggested. Further research is proposed to discover the material's elastic modulus and Poisson's ratio, as well as its relation to mortar strength.

To next page

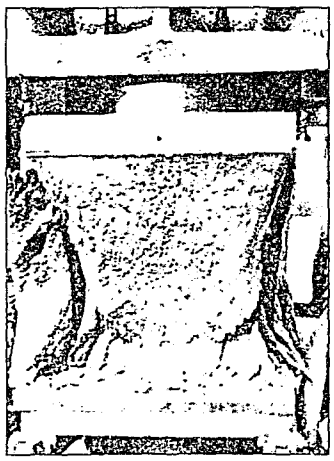

Fig 5: The distorted hourglass failure pattern of this 500 mm cube of mortar (containing no rock) was caused by different platen constraints. The lower surface of the cube in contact with steel has experienced far less severe dilatory constraint than the top surface, which is in contact with hardboard. Hence it is crucial that platen interfaces in future testing be consistent with a chosen standard.

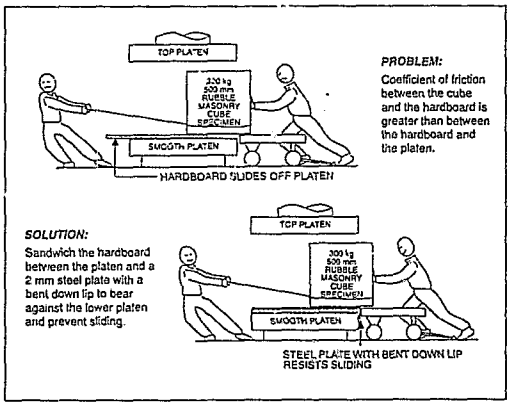

Fig 6: Diagrammatic representation of the solution to the platen interface problem

Acknowledgements

Financial support for the research was provided by the Kenneth Birch Fund, Prof R McCutcheon and the Department of Transport. Dr Chandler and Mr Tokkie Muller of the Department of Mechanical Engineering at the University of the Witwatersrand made available to this endeavour their 500 t Amsler press. Mr Jacob Kroonstuiwer of the CSIR Mine Hoist Testing Laboratory at Cottesloe made available the use of the 1 000 t press. Mr Altus de Beer of de Beer and Whyte, Mr Walter Barker of the CSIR and the librarians of the Cement and Concrete Institute library all assisted me with the literature survey. Their help and generosity are greatly appreciated.

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APPENDIX 3

DEFORMATION PROPERTIES OF RUBBLE MASONRY CONCRETE

VOLUME 41 NUMBER 1 FIRST QUARTER 1999

Journal

The South African Institution of Civil Engineering

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Joernaal

Die Suid-Afrikaanse Instituut van Siviele Ingenieurswese

B Fine, R G D Rankine and R T McCutcheon

Deformation mechanics of rubble masonry concrete

Synopsis

The deformation mechanics of rubble masonry concrete (RMC) are explored experimentally and theoretically with reference to the behaviours of analogous concrete and natural conglomerate. The composite stiffness of large RMC cubic specimens, containing rock types exhibiting diametrically extreme values of stiffness, was determined dynamically and measured statically to gain a preliminary indication of the range of values likely to be encountered. A broad spectrum of moduli of deformation and Poisson's ratios were recorded. An idealized finite element model is used to gauge the effects of spherical inclusion size and inclusion suspension within the matrix as well as the effect of orientation of elongated particles with respect to the principal load axis. It is concluded that in addition to the parameters currently known to govern the stiffness of conventional concrete, RMC stiffness is probably also affected by the extent of inter-particle contact, shape and orientation of elongated inclusions with respect to the principal stress axis and the maintenance of bond at the transition interface.

Samevatting

Die vervormingsmeganika van messelwerkpuinbeton (MPB) word eksperimenteel en teoreties ondersoek met verwysing na die gelyksoortige gedrag van beton en natuurlike konglomerate. Die algehele saamgestelde styfheid van groot MPB kubiese monsters wat rotstipes met ekstreme diametriese styfheidswaardes bevat, is dinamies bepaal en staties gemeet om 'n voorlopige aanduiding van die reeks waardes te kry wat verwag kon word. 'n Breë spektrum van vervormingsmodulusse en Poisson-verhoudings is gevind. 'n Geïdealiseerde eindige-elementmodel is gebruik om die effek van sferiese insluitingsgrootte en insluitingssuspensie binne-in die matriks sowel as die effek van orientering van langwerpige deeltjies tot die hoofbelastingas te bepaal. Daar is tot die slotsom gekom dat benewens die parameters wat ons huidiglik weet die styfheid van konvensionele beton beinvloed, word MPB se styfheid waarskynlik ook beinvloed deur die mate van interpartikelkontak, vorm en orientering van langwerpige insluitings relatief tot die hoofbelastingas en die instandhouding van die binding by die oorgangsfase.

Introduction

Finite element models are increasingly regarded as the best analytical tools for ambitious arch designs. They are extremely input-sensitive and depend upon accurate knowledge of material stiffness for predicting structural response to loading and thermally induced strains, particularly in the case of large monolithic rubble masonry concrete (RMC) dams. The current practice of guessing the stiffness of RMC, or at best assuming an equivalent concrete modulus, for want of a reliable estimate undoubtedly compromises the fidelity of their output.

Although it may be argued that the Zimbabweans have successfully built many large wide-valley RMC arch dams empirically (Chemaly, 1997) and without any acquired knowledge of this material's elastic modulus (Chemaly, 1997; Shaw, 1998), there is cause for concern that the adoption of this practice elsewhere may result in unsafe structures. Their success may be attributable to a combination of favourable factors such as their almost exclusive use of granitic aggregates (which are not exceptionally stiff and do not exhibit extreme coefficients of thermal expansion) combined with their temperate climate. Their precedent is therefore of limited value to engineers who are required to make use of other geologies in climates that experience greater temperature fluctuations, such as South Africa's. Shaw (1998) has illustrated this by testing the response sensitivity of an idealized wide-valley arch dam to subtle manipulations of these parameters using a finite element model. His model predicts that maximum stresses (compressive and tensile) may be subject to an increase of as much as an order of magnitude, upon a realistic 10 °C temperature drop, as the material's elastic modulus is increased from 15 GPa to 40 Gpa, assuming coefficients of thermal expansion typical of ordinary concretes.

Alexander (nd) has reviewed several methods of estimating the elastic modulus of ordinary concretes, based upon a relationship between elastic modulus and compressive strength (SABS 0100, 1980; BS 8110, 1985; American Concrete Institue, 1982; Euro-International Committee for Concrete, 1978, 1990) as well as micro-rheological two-phase mathematical models (Hanson, 1965; Hirsch, 1962; Counto, 1964; Hashin, 1962; Hobbs, 1971) (shown in Fig 1). Recent research has shown that concrete strength per se makes a relatively minor contribution to the composite's elastic modulus

Rod Rankine *completed his BSc (Bldg) at the University of the Witwatersrand in 1988. He subsequently carried out an investigation on a new type of corrosion-resistant reinforcing steel for Middelburg Steel and Alloys (now Columbus), for which he was awarded his MSc (Eng). In 1991 he worked for R J Southey on the Actinia oilrig during the Mosgas project. In 1992 he joined the academic staff at Wits, where he developed an interest in technical aspects of labour-intensive construction, particularly appropriate design and construction guidelines for rubble masonry structures. In January 1998 he was appointed a lecturer at the School of Concrete Technology at the Cement and Concrete Institute*

Prof Robert McCutcheon *obtained his BSc (Eng) in 1963 and a Graduate Diploma in Engineering in 1964 from the University of the Witwatersrand. Between 1965 and 1971 he worked as an engineer and chairman of the South African Voluntary Service (SAVS). During the SAVS work he developed an interest in community development in relation to housing and the provision of low-cost infrastructure. From 1975 to 1987 he worked in Iran, Botswana and East Africa. Since then he has been a Professor of Civil Engineering at Wits. In 1994 he was appointed Head of the Department of Civil Engineering. He is the author of a Green Paper on Job Creation in Transport and Public Works (Western Cape).*





Benjamin Fine *registered at the University of the Witwatersrand for a BSc in Town and Regional Planning in 1991. In 1992 he changed direction and obtained an N4 in light current electronics from the Johannesburg Technical College. In 1993 he registered at Wits University for a BSc Building Science, which he completed in 1997. Research into the deformation characteristics of rubble masonry concrete was the topic of his final-year discourse.*



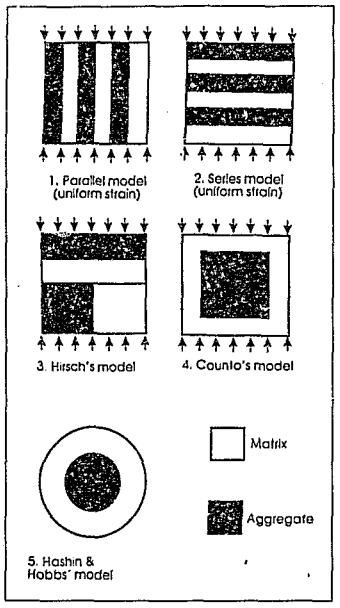


Fig 1: Existing theoretical two-phase models for predicting the elastic modulus of concrete (Alexander, nd)

and that the major contribution comes from the stiffness of the aggregate and its volume concentration (Kaplan, 1959; Teychenne, 1978). Thus, the mathematical two-phase models would appear to simulate the true behaviour of ordinary concrete more realistically. However, these two-phase concrete models are blind to the consequences of aggregate size and shape, factors that may be significant in RMC.

Beyond the immediate ambit of concrete literature, geophysicists Lindquist and Goodman (1994), engaged in a study of rock mechanics, have shown by a series of triaxial tests on physical model melanges (French word for 'mixture' of relatively large, competent blocks within a matrix of finer and weaker texture) that the modulus of deformation may be greatly affected by the orientation of inclusions with respect to the axial loading direction (see Fig 2). Although their model melanges were intended to simulate the chaotic fabric of confined rock, such as boulder conglomerate found in nature, their findings may be analogous to the similarly chaotic man-made RMC melange. Unfortunately, their six inch (150 mm) diameter specimens were too small to permit exploration of the increased dependence upon bond as the interfacial transition surface diminishes in proportion to inclusion size. Moreover, all literature appears completely silent on the question of a relationship between stiffness and nominal aggregate particle size.

Rankine (nd), Rosenman (1996) and Roxburgh (nd) have attempted to measure the stiffness of 500 mm and 700 mm RMC cubes during tests where the primary objective was to determine compressive strengths and failure characteristics. These efforts included the use of a load-displacement drum recorder, connected via a 'fish-line' to the mobile platen of a hydraulic press (as shown in Fig 3) and the use of pairs of displacement dial gauges mounted symmetrically between platens on opposite sides of cubic specimens. Both these methods produced questionable results for numerous reasons, some of which are listed below:

1. The low aspect ratio of the cubic specimens, coupled with the significant bilateral and, to a lesser extent, trilateral frictional constraints offered by the platens, restricts specimen dilation, producing artificially stiff results.
2. The recorded lower platen displacement (measured with the apparatus illustrated in Fig 3) is obscured by several extraneous phenomena such as the 'bedding-in phase' of the specimen surfaces, yielding of the packing material, elastic deformation of the press and rotation of the moving platen if the specimen yields asymmetrically. Furthermore, at a stage when the reaction against the top platen equals the weight of upper transverse, backlash in the threaded reaction columns (at the moment when the threaded columns react in tension) causes an abrupt displacement (as shown in the load displacement plot in Fig 4).
3. The fish-line connecting the drum recorder to the moving platen is subject to creep, particularly where nylon is used and maximum displacement amplification is selected. (Maximum displacement amplification subjects the line to slightly greater stress.)
4. The maximum displacement amplification in drum recorders is determined by the ratio of pulley to drum diameters and is limited to a ratio of about 1:4, by which stage the maintenance of static friction between the drum pulley and the fish-line becomes questionable and significant line strain occurs before the static friction between pen and paper is overcome. Discrimination between the true gradient and even the most dominant extraneous phenomena is impossible at such low amplification.

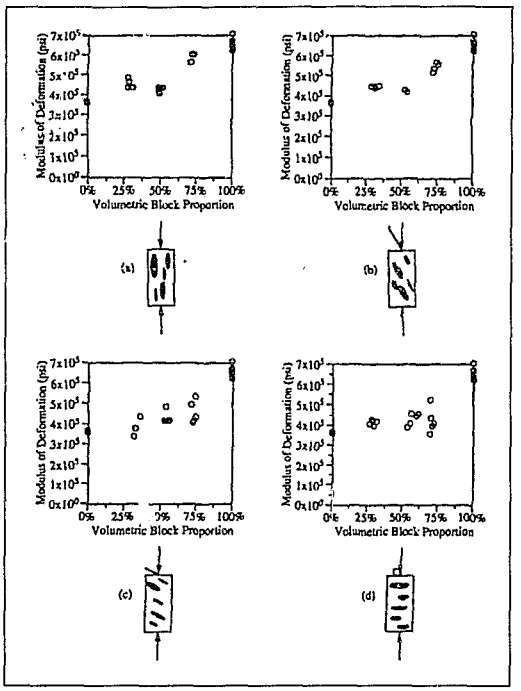


Fig 2: The effect of inclusion orientation (a) 0° (b) 30° (c) 60° (d) 90° with respect to axial load and volumetric block proportion (inclusion volume concentration) on the modulus of deformation of Lindquist and Goodman's (1994) triaxially loaded melanges

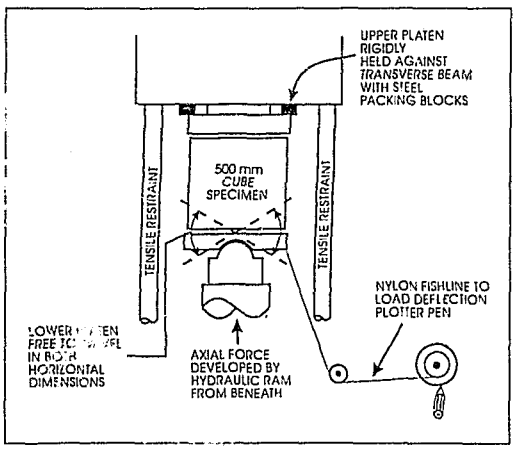


Fig 3: 'Fish-line' and drum displacement recording apparatus used by Rankine (nd), Rosenmann (1996) and Roxborourgh (nd) in various preliminary attempts to measure stiffness

Experimentation

Most reported values of concrete elastic modulus have been derived from static tests (Alexander, nd), although dynamic methods, such as the ultrasonic p se velocity (UPV) method, which exploit the relationship between medium stiffness and the transmission velocity of mechanical vibration, also exist. The static modulus is typically derived at a stress of one third (BS 1881, 1983) to 40 per cent (ASTM, 1987) of the specimen's strength where load is applied slowly over a period of minutes, whereas the dynamic modulus is derived at extremely low stress where the load is applied rapidly over a period of microseconds. Consequently, the dynamic method measures the initial tangent modulus and the static method measures the 'secant' or 'chord' modulus.

Both methods have their advantages and shortcomings. The static method instils confidence for its simplicity of direct measurement under stress similar to the working stresses encountered by structures. However, because the static modulus is derived at a relatively high stress over a relatively long period, it includes an extraneous non-elastic (irrecoverable) component. In practice this hysteresis may be reduced, but never totally eliminated, by subjecting the specimen to several load cycles before the strain that determines the modulus is recorded. Because the dynamic modulus is determined so quickly and at such low levels of stress, its non-elastic component is negligible. However, it remains unknown whether pulse velocity is independent of variations in specimen size and shape (Whitehurst, 1966) and the presence of interfacial transition boundaries may cause interference, particularly at discontinuities created by bleed-water and load induced cracks. Nevertheless, the relationship between statically and dynamically determined values of RMC stiffness warrants exploration, since the latter may obviate the need for large unwieldy cube specimens in future.

Test specimens

Existing standard test methods (BS 1881, 1983; ASTM, 1987; RILEM, 1975), used to determine the static modulus of elasticity in concrete, prohibit test specimens with an aspect ratio (length to diameter) of less than two, since the gauged length of shorter specimens may be made artificially stiffer by the frictional confinement of the platens. The encroachment of this phenomenon, away from the loaded faces, is proportional to the coefficient of friction at the platen/specimen interface and may be significant, as evidenced by the classic hourglass fracture of concrete test cubes. Nevertheless, cubic specimens were desired to exploit limited resources, as they permit comparable measurements in more than one dimension, facilitating two sets of readings per specimen. Furthermore, the cubic shape also permits a representative cross-sectional girth without making the specimens unmanageably heavy. Three 500 mm cubic specimens were manufactured according to a method proposed by Rankine (1997). In an effort to minimize the frictional platen constraint during testing, a double membrane of 250 μm slippery polyethylene sheeting, sandwiching a layer of petroleum jelly, was placed between the loaded cube faces and 6 mm hardboard packing above and below the specimens (see Fig 7). A thin quadrant lip was bonded to the circumference of the lower hardboard packing as a precaution against possible lateral displacement of the specimen (a potential consequence of the reduced friction).

Two of the cubes were cast with different geological inclusions and the third cube contained pure mortar as a control with which to relate size effect phenomena during comparisons with nine 100 mm cube and prism specimens. Olifantsfontein dolomite and Leach and Brown indurated siltstone were deliberately chosen as aggregates, following work by Alexander and Davis (1992) that showed these geologies to produce concretes with diametrically extreme values of elastic modulus over a wide range of strengths (see Fig 5). Analogously, it is anticipated that the values of RMC stiffness obtained from this study might similarly define the boundaries of a significant range of other geologies likely to be encountered and that such data may even be interpolated to estimate the composite stiffn... where aggregates of intermediate stiffness are used. Moreover, the elongated shape of the siltstone fragments (a consequence of the slaty cleavage of this sedimentary rock) permits exploration of the effect of their orientation with respect to axial loading direction. A range of rock sizes up to about 20 kg (typical of the sizes used in RMC construction) were hand packed to maximize the volume concentration of stone. The elongated axes of the siltstone fragments were aligned parallel with one pair of mould faces to simulate the current practice (noted by Rankine et al, 1993) of placing flat stones horizontally. A one part 30 per cent fly ash cement to four parts river sand (by volume) mortar with a 30 mm slump and 21,7 MPa 28-day, 100 mm cube strength was used for the matrix. The material properties are presented in Table 1.

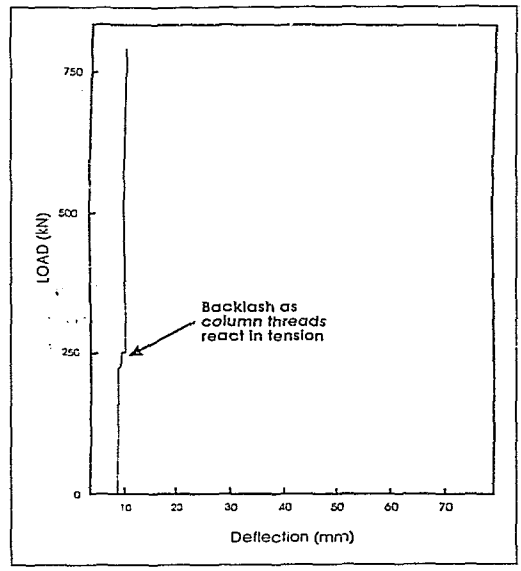


Fig 4: Typical load displacement plot obtained from the apparatus shown in Fig 3. The arrow points to the sudden extraneous displacement caused by 'backlash' in the machine threads of the upper transverse at the moment when the threaded columns begin to react in tension.

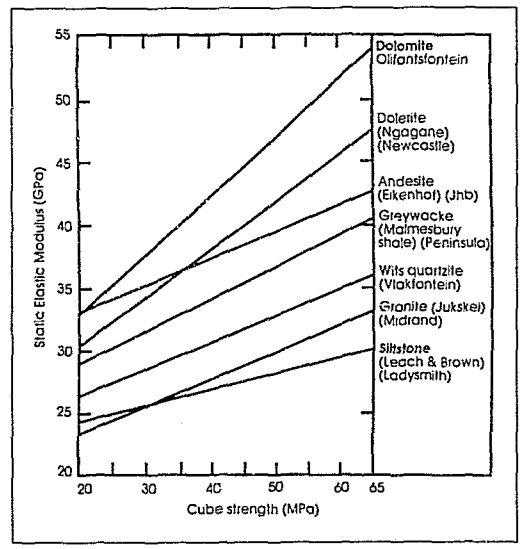


Fig 5: The relationship between the statically determined modulus of elasticity and cube strength of a range of common aggregates recorded by Alexander and Davis (1992)

Apparatus

Most reported values of concrete elastic modulus have been obtained with static apparatus outlined in BS 1881: Part 121 (1983), ASTM method C469-87a (1987) and RILEM CPC 8 (1975). These standards make provision for the use of electronic resistance strain gauges bonded onto the specimen surfaces and the use of a displacement gauge compressometer. The ASTM method restricts the minimum gauge length to not less than three times the maximum nominal aggregate size to reflect the homoge-

Table 1: Specimen ingredient properties (Davis and Alexander, 1994) and proportions

	Dolomite	*Siltstone*	*Mortar*
Elastic modulus	109-118 GPa	24-27 GPa	18 GPa
Relative density	2,86	2,55 - 2,66	2,01 - 2,16
Vol fraction in cube	43%	23%	Remainder
Water absorption	0,21%	3,07%	-
Uniaxial compressive strength (MPa)	212 - 396 (40 mm cylinder)	125 - 226 (40 mm cylinder)	21,7 (100 mm @ 28 days)

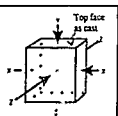
neous behaviour of the composite with maximum fidelity. This precaution reduces the probability of bias arising from locally dominant material concentrations. For this reason, a very long gauge length is particularly crucial for RMC specimens, longer than commercially available bondable electronic gauges and preferably almost as long as the length of the 500 mm specimens.

The elimination of the bonded electronic gauge option prompted the development of an original compressometer (shown in Fig 6), with two square yokes monitoring a gauge length of 80 per cent of the cube, based on the ASTM design and an independent micrometer for measuring dilation between targets fixed centrally to diametrically opposing vertical faces midway between the loaded surfaces. Several practical difficulties in reading the micrometer manifested during first testing. The restricted space between the platens made access to the targets extremely awkward, the size of the micrometer necessitated an extra pair of hands to support its distal end and the aluminium frame lacked sufficient stiffness to consistently record a constant distance with acceptable accuracy. Consequently, the micrometer was abandoned and the compressometer was modified by the addition of an integrated extensometer illustrated in Fig 7.

An ultrasonic pulse velocity recorder (shown in Fig 8) was used to record the dynamic modulus of each cubic specimen. Because of the material's anisotropy, 49 readings were taken over a square uniformly distributed grid (in each dimension) to obtain an acceptable average with the aid of two templates, one of which is shown in the photograph.

The results are summarised in Table 2. The ultrasonic testing was conducted prior to the static testing to preclude the possibility of interference arising from material discontinuities as a consequence of load-induced cracking. Thereafter, 'static' load was applied using the 1 000 short ton (907 metric tons) press at the CSIR Mine Hoisting Laboratory at Cottesloe, Johannesburg. Load was applied first to the designated XX axis (the anticipated strongest direction in the case of the siltstone: perpendicular to the longitudial orientation of the elongated specimens). [The siltstone was first loaded along its strongest axis to increase the probability of it remaining intact to facilitate the second static test.] The specimens were then rotated 90° and reloaded orthogonally to their first loading along their ZZ axes. The results are summarised in Figs 9A and 9B. Static strains were recorded from virgin loading because there was no means of predicting each specimen's ultimate compressive strength. Furthermore, unlike conventional concrete, the relationship between load cycle frequency and hysteresis remains unexplored.

Table 2: Initial tangent moduli measured dynamically along different axes with the ultrasonic pulse velocity apparatus shown in Fig 8

	Mean dynamic elastic modulus (GPa) $\bar{x}$	Standard deviation from the mean (GPa) σ	Coefficient of variation (standard deviation from the mean expressed as a percentage of the mean) $\sigma/\bar{x}$
Dolomitic cube			
XX	65.3	9.6	14.7%
YY	55.5	6.6	11.8%
Siltstone cube			
XX	40.4	3.6	8.8%
YY	36.4	3.1	8.4%
ZZ	29.0	1.8	6.1%
Mortar cube			
XX	24.6	0.5	2.2%
YY	24.4	0.6	2.3%

Idealised theoretical modelling

The effects of spherical inclusion size and their suspension within the matrix

Two series of six idealised RMC melange models, containing uniform cubically packed spherical inclusions, were analysed using a two-dimensional finite element model ('Prokon' Groenkloof SA & London UK Second Beta Plane Stress Strain Analysis) to explore the relationship between inclusion size, composite stiffness and maximum internal stresses. In the first series, the inclusions make intimate contact with their neighbours and the platen surfaces, whereas the inclusions of the second series are separated from one another as well as the platen surfaces by a volume of

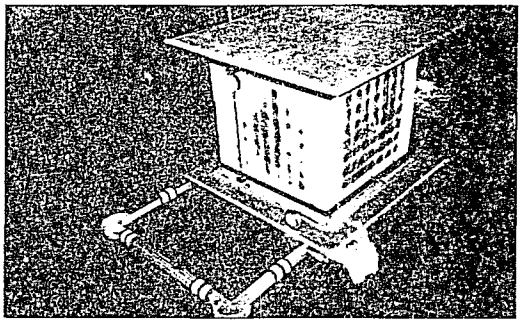


Fig 6: Prototype compressometer attached to a 500 mm RMC cube specimen and micrometer (foreground) built to measure moduli of deformation and dilation respectively

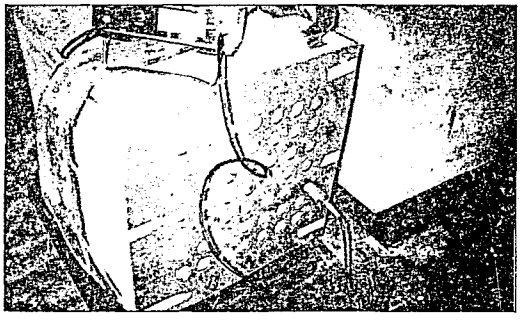


Fig 7: Improved integrated compressometer/extensometer to measure both axial strain and dilation. The gauges measure twice the actual strain. The double polyethylene slip membrane can be seen between the upper face and the packing.

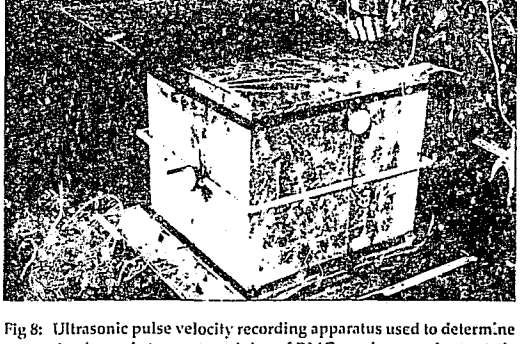


Fig 8: Ultrasonic pulse velocity recording apparatus used to determine the dynamic tangent modulus of RMC specimens prior to static testing

mortar. The inclusion volume fraction remains constant throughout each of the six cases and is 0,524 and 0,382 respectively. A 10 GPa mortar matrix stiffness and a 100 GPa inclusion stiffness (a tenfold composite phase stiffness ratio) were simulated, bordering the highest variance anticipated in practice. A Poisson's ratio of 0,1 is assumed and stiffness is recorded at a stress of 3 MPa. The model assumes perfect bond with the aggregate and absolute platen constraint. Furthermore, uniform strain of the load surfaces is assured by a pair of 'infinitely stiff' platens. The results of the theoretical analysis are presented in Table 3 and typical compressive stress distributions are presented in Fig 10.

The effects of particle orientation with respect to principal stress trajectory

The material properties, volume fractions and geometry of the siltstone specimen were simulated using the same theoretical model and assumptions described previously. The model was 'test loaded' both parallel and perpendicular to the longitudinal axes of the elongated inclusions. A schematic representation of the model and its predicted stiffness in each axis is presented in Fig 11.

Discussion

The preliminary findings of this study confirm that a significant spectrum of composite material stiffness and dilation potential confronts the designer of RMC structures. The vast differences in measured intra-specimen values along alternate axes of the RMC cubes is indicative that this is a highly aelotropic material that is sensitive to the shape and orientation of the rock inclusions, factors that to date have not concerned concrete technologists.

The statically measured deformation moduli (secant moduli) were, without exception, lower than those determined dynamically (initial tangent moduli). [The term 'elastic modulus' is deliberately avoided since 'elastic' implies fully recoverable deformation. During virgin loading the specimen undergoes both recoverable (elastic) and non-recoverable (non-elastic) deformations.] This is to be anticipated in brittle disordered materials such as concrete, which strain-soften upon fracture (see Fig 12). The consistency with which the dynamic apparatus seems

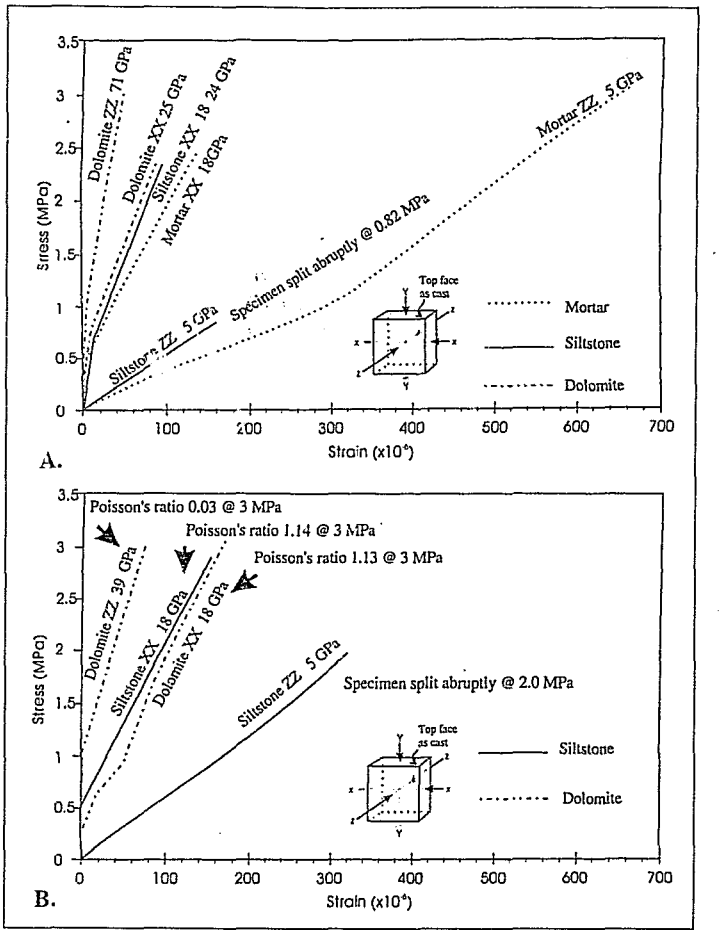


Fig 9: Graphical comparison of the stress-strain relationship of the test specimens upon testing with the prototype device shown in Fig 6 (A) and upon subsequent re-testing with the improved device shown in Fig 7 (B)

Table 3a: Relationship between contiguous idealized spherical inclusion size, composite stiffness and maximum internal stresses derived theoretically using a finite element model. The values apply to Series 1 where the inclusions are contiguous.

SERIES 1 Inclusions in contiguous array Vol fraction stone 0.524						
Inclusion diameter mm	1000	500	333	250	125	63
Particle volume l	524	65	19	8	1	0.1
No of particles/m³	1	8	27	64	512	4096
Specific surface m²/m³	3.14	6.28	9.42	12.57	25.13	51.07
Composite stiffness GPa	27	29	30	33	41	43
Max comp stress (MPa)	66	38	32	27	18	7
Max tens stress (MPa)	19	10	7	5	4	3

Table 3b: Relationship between idealised suspended (within the matrix) spherical inclusion size, composite stiffness and maximum internal stresses derived theoretically using a finite element model. The values apply to Series 2 where the inclusions are separated by a bed of mortar.

SERIES 2 Inclusions separated by a bed of mortar Vol. fraction stone 0.382						
Inclusion diameter mm	900	450	300	225	113	56
Particle volume (-)	382	48	14	6	0.7	0.09
No of particles/m³	1	8	27	64	512	4096
Specific surface m²/m³	2.54	5.09	7.63	10.18	20.36	40.72
Composite stiffness GPa	32	32	32	32	32	32
Max comp stress (MPa)	(-6)	(-6)	(-6)	(-6)	(-6)	(-6)
Max tens stress (MPa)	(-1)	(-1)	(-1)	(-1)	(-1)	(-1)

capable of predicting the composite's initial tangent modulus (as evidenced by the coefficients of variation in Table 2) appears to be related to homogeneity and perhaps also to the ratio of inclusion to matrix stiffness. The pure mortar specimen, which was the most homogeneous in the group, showed the best consistency, followed by the specimen containing siltstone. The superior consistency of ultrasonic pulse data from the RMC specimen containing siltstone might be ascribed to the similarity in stiffness between the siltstone and its mortar matrix. As the stiffness of the rock approaches that of the matrix, the transmission velocity of mechanical vibration becomes less affected by the disordered inclusions (ie the ultrasonic pulse is less able to distinguish between the two phases). Thus it is hypothesized that dynamic methods of elastic measurement may be better suited to RMC containing softer rather than harder inclusions.

An examination of Figs 9A and 9B reveals a marked contrast in specimen stiffness between XX loading and subsequent ZZ reloading in every case. The 'virgin' XX loading curve of the pure mortar specimen in Fig 9A shows slight strain-softening up to the final stress of 2,5 MPa, at which loading was terminated when fine vertical cracks became visible. Its subsequent reloading curve steepens under increasing stress, suggesting that the material stiffened after the 'virgin' cracks (now perpendicular to the load) closed. The much lower subsequent modulus (upon reloading) is probably the result of significant strain absorption by the cracks (now perpendicular to the ZZ reloading axis) during closure. In contrast, the absence of visible cracks upon 'virgin' XX loading in the specimen containing siltstone and the linearity of its ZZ reload curve call for an alternative explanation for the low stiffness recorded along its ZZ axis.

Theoretically, an increase in modulus ought to accompany a rotation of elongated inclusions from horizontal to vertical. Simple models such as the one proposed by Hirsch (1962) for concrete and the finite element model illustrated in Fig 11 show that when the stiff bands are vertically inclined they attract more of the axial stress. The result is a stiffer composite. Moreover, Lindquist and Goodman's (1994) physical triaxial tests (see Fig 2) further support this theory. The counter-intuitive departure of the siltstone specimen from this theory might be explained by the mechanical action of the stones (elongated stones with their longitudinal axes parallel to the principal stress trajectory of the composite wedge and cleave the matrix apart, a phenomenon illustrated in Fig 13a and described more fully elsewhere (Rankine, nd)) in fracturing the composite at very low levels of axial stress. Indeed, the specimen abruptly split almost completely

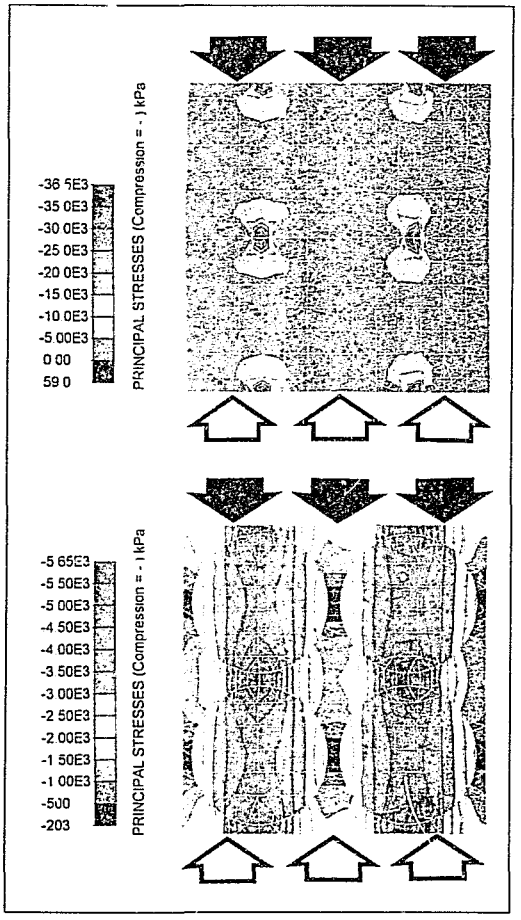


Fig 10: Typical compressive stress distributions, in the case of spherical inclusions touching one another (A) and in the case of spherical particles floating in a bed of mortar (B)

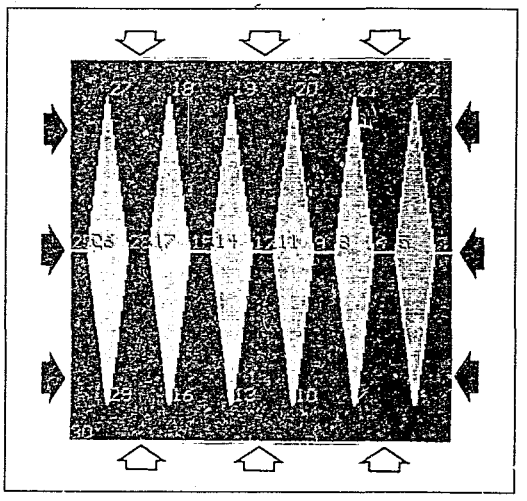


Fig 11: Idealized theoretical two-dimensional model simulating the siltstone specimen's theoretical behaviour under deformation assuming perfect retention of interfacial bond. The model predicts stiffness perpendicular and parallel to the particle longitudinal axes of 14 GPa and 15 GPa respectively.

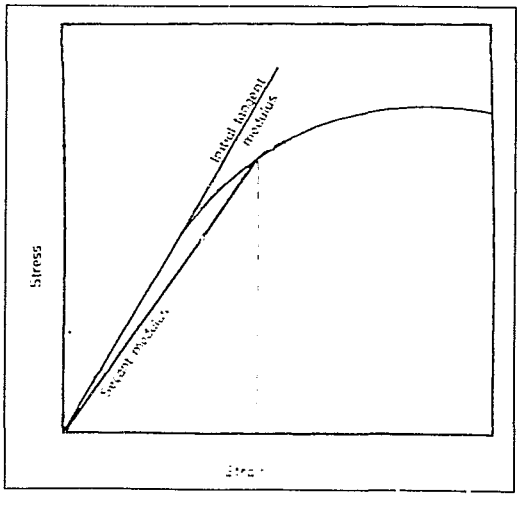


Fig 12: Two forms of modulus in brittle disordered materials that strain-soften upon fracture (Alexander, nd)

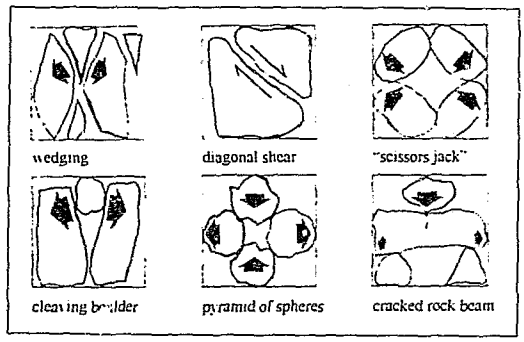


Fig 13: Proposed mechanisms that may result in high values of dilation

in half at only 0,82 MPa when it was first compressed along its ZZ axis. It was then carefully prised apart for examination. Fracture appeared to have initiated at the interface of the flat stone surfaces parallel to the principal stress. The specimen was then carefully bonded together with an epoxy adhesive before being reloaded. Its subsequent behaviour (see Fig 9B) was similar. Again, when reloaded along its ZZ axis, it split abruptly parallel to the previous fracture (now repaired) at a slightly higher stress of 2 MPa, by which time its rate of dilation, measured by the extensometer (shown in Fig 7), exceeded its axial deflection by such an order that recording proved impossible.

The behaviour of the specimen containing dolomite is almost perplexing. Intuitively, its chunky spherical shape might have been expected to yield a composite with similar properties in both the XX and ZZ axes. However, the stiffness measured in the ZZ axis proved more than double that measured in the XX axis in both tests. Furthermore, the dilations measured perpendicular to those same axes differed by nearly 37 times. Since the stiffness measured during ZZ reloading exceeded the XX 'virgin' stiffness by a large margin, this discrepancy cannot be attributed to strain weakening and/or fracture closure. Examination of the origin of the ZZ gradient in Figs 9A and 9B shows an extremely high initial tangent modulus (measured statically), perhaps as a result of a very favourable distribution of rock in that axis. Unfortunately, the absence of dynamically recorded data about its ZZ axis precludes a comparison with a dynamically predicted modulus in this axis.

It is worth noting that without exception, measured dilation was greatest in the plane orthogonal to the axis of least stiffness. Several proposed mechanisms that might be responsible for this phenomenon are illustrated in Fig 13.

Examination of the theoretical output presented in Table 3 reveals a marked contrast in behavioural response between the composite containing idealised spherical inclusions in intimate contact with one another and the composite containing inclusions separated by a bed of soft mortar matrix. While the stiffness of the former appears to be inversely proportional to inclusion size, the stiffness of the latter appears unaffected. Thus, it would appear that a bed of mortar effectively neutralises the size effects of perfectly bonded spherical inclusions. Furthermore, maximum internal stresses around points of contact appear to be inversely proportional to the number of inter-particle contacts per cross-sectional area, orthogonal to the principal stress axis. Theoretically, the contact area between touching spheres or spheres and plane surfaces is infinitesimal, regardless of their size. This gives rise to substantial stresses under even small inter-particle forces. An increase in the number of inclusion contacts per unit area in the plane orthogonal to the principal stress axis results in load sharing, among a greater number of stiff columns of contiguous spheres, and the consequent lowering of contact stresses. This is analogous to the model shown in Fig 14, where the columnar supports (the columns of contiguous spheres that attract stress) are represented by springs. Since the contact area between spheres is assumed to be infinitesimal regardless of the sphere dimensions, the stiffness of all springs is equal. Therefore, the greater the number of springs per unit of stressed area, the lower the stress in each spring. Since strain is proportional to stress, more springs deflect less (the composite experiences an overall stiffening).

In practice, the RMC melange departs from this theoretical model in the following respects:

1. The ir[illegible]unlikely to remain perfect under high stress, partic[illegible]irface diminishes as the inclusions become bigger.
2. Despite t[illegible]ate attempts to minimize the mortar content f[illegible]ne stones making intimate contact), the proportion [illegible]ontacts capable of increasing composite stiffness is in all pr[illegible]y less when inclusions are not regular and true.
3. RMC typically adopts a twisted orthorhombic packing arrangement, which is theoretically optimal (Rankine, 1988). Although the inefficient cubical packing arrangement of the model reproduces the volume fraction of aggregate reportedly achieved in practice, this is probably a coincidence brought about by the absence of sharp protrusions that would otherwise prohibit regular close contact between inclusions in reality. A more complex three-dimensional model capable of simulating the twisted orthorhombic packing arrangement would be a logical progression for profitable future research.

Nevertheless, the model illustrates the principle whereby load is attracted to the stiffest parts of the composite and how the enhanced distribution of this load reduces the composite strain. It also illustrates the advantage of a mortar bed in eliminating high contact stresses where very large inclusions are used and/or the wisdom of limiting the maximum size of boulders.

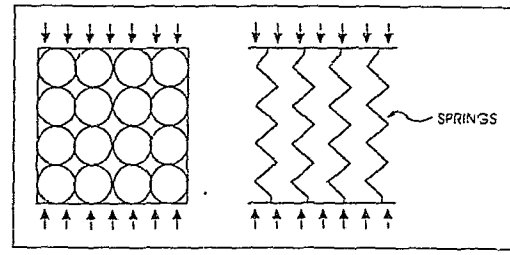


Fig 14: Analogous spring model, where the columns of spheres are represented as equivalent springs

Conclusion

The careful selection of rock types, exhibiting extreme values of stiffness, and the radical manufacture of one of the cubic test specimens used in this preliminary study yielded a wide range of statically measured deformation moduli (between 5 GPa and 71 GPa) and Poisson's ratios (between 0,03 and more than one). Dynamically determined initial tangent moduli were consistently greater than the statically measured secant moduli. A hypothesis is proposed whereby the dynamic determination of initial tangent moduli may be better suited to RMC containing softer rock types. A theoretical finite element model was used to illustrate how the presence of a thin layer of soft mortar, separating very large stones, may be effective in greatly reducing maximum stress levels within the composite. The model also shows that stresses developed at points of rock contact are inversely proportional to the number of inter-particle contacts disposed to distribute load. In addition to the exponents currently recognized to govern the stiffness of concrete (namely the stiffness of the individual ingredients, their volume fractions, maturity and the presence of voids), RMC stiffness appears sensitive to the number of inter-particle contacts disposed to distribute load within the principally stressed area, the orientation of inclusions and the maintenance of bond at the rock/matrix interface.

Acknowledgements

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APPENDIX 4

ESTIMATING THE HUMAN ENERGY COST TO MIX MORTAR MANUALLY

CONCRETE BETON

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P OBox/Posbus 168
Halfway House 1685
Tel: (011) 315-0300/0314
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TECHNICAL PAPER

ESTIMATING THE HUMAN ENERGY COST TO MIX MORTAR MANUALLY

Roderick G.D. Rankine, Edith M. Peters and Robert T. McCutcheon



R.G.D.Rankine

Rod Rankine matriculated at St Stithians in 1984 and completed his B.Sc. (Bldg.) at Wits in 1988. He subsequently conducted an investigation into the use of 3CR12 alloy as a new corrosion resisting reinforcing steel for Middelburg Steel and Alloys (now Columbus) for which he was awarded his M.Sc.(Eng.) In 1990, he served in the SA Navy and in 1991 he worked for R.J. Southey on the Actinia oilrig during the final stages of the Mosgas project. From 1992 - 1998, he lectured on the academic staff at Wits where he became a senior lecturer and developed a research interest in formulating design and construction guidelines for labour-intensively built rubble masonry concrete structures. Recently he joined the Cement and Concrete Institute's School of Concrete Technology.

SYNOPSIS

A new method of estimating the human energy cost to mix mortar on the ground by shovel as well as by a simple manually powered drum mixer is described. The heart-rate response of a human subject engaged in sub-maximal upper-body activity simulating the mixing of concrete is shown to correlate closely with both oxygen consumption and mechanical work output. The test results confirm that shovel mixing is an arduous, inefficient and potentially hazardous activity. The manually powered drum mixer was found to be approximately twice as productive as shovel mixing on the ground, possessing vast potential for improved mechanical efficiency. The high cost of human energy is compared with the cost of energy derived from diesel fuel and the daily food requirement of mixer labourers is quantified in terms of the equivalent energy contained in loaves of bread. On this basis, the minimum wage of R7 per day, proclaimed by the National Public Works Program is questioned.

1.0 INTRODUCTION

Rankine and McCutcheon [1] have identified large discrepancies in reported productivity amongst labourers engaged in the construction of rubble masonry structures. Production rates of as little as 0.2 to 0.3 m^3 of placed rubble masonry per labourer per day have been reported [2,3] in cases where mortar has been mixed on the ground by shovel, whereas rates as high as 0.8 to 1.9 m^3 per labourer per day are reported [4,5,6] where mortar was mixed mechanically. Hence, it seemed probable that the cause of the low productivity might be the result of exceptional energy taxation of the human body by shovel-mixing. A literature review revealed limited existing knowledge of the energy cost and productivity of mixing concrete [7] and mortar [8] by hand. De Beer [7] determined the human energy requirement to shovel mix concrete by measuring the oxygen consumption of a subject with a Max Planck portable spirometer. Knowledge of a subject's oxygen consumption provides an ideal means for accurate quantification of human metabolic activity since 20.22 kJ of energy is liberated for every litre of oxygen consumed at Standard Temperature and pressure [9]. Direct measurement of oxygen consumption can be achieved by open circuit spirometry. The volume of expired air and its carbon dioxide and oxygen content can be recorded using portable spirometers or very costly computerised metabolic analysers (metabolic carts). Such methods are not yet practical for measuring the energy requirements to mix mortar since the former are extremely costly and the validity of their data is reliant upon the accuracy of "auto-calibrated" gas analysers within the confines of their small carry cases; while the metabolic carts are cumbersome and heavy and only located in laboratory settings. Conventional metabolic research has therefore been confined to task simulations and data collection in the laboratory.

Numerous work physiologists [10,11,12] have identified a correlation between heart-rate and oxygen consumption in humans at sub-maximal workloads. By recording the relationship between heart-rate and oxygen consumption at known sub-maximal workloads, a heart-rate monitor may theoretically be used to predict metabolic response to another similar sub-maximal physical activity which taxes the same mass of muscle. If the mechanical power output can be measured, for example by an ergometer, then the energy conversion efficiency of a system can also be estimated. Modern heart-rate monitors are relatively inexpensive and well suited to complicated physical activities such as concrete mixing because, unlike spirometers, they are inexpensive , light-weight and do not bind the subject with tubes or wires, thereby restricting the range of movement. Heart-rate monitors were successfully used by Lambert et al [13] (1994), to estimate the energy expenditure of sugarcane cutters working in the field, and by Engebos [14] (1997), in a study parallel to this one, to estimate the energy expenditure of trench diggers.

2.0. OBJECTIVE

The objective of the experiment was to develop a technique to estimate the human energy cost of mixing mortar, of a consistence typically used in rubble masonry concrete, by two different manual mixing methods and to determine whether either or both of these manual methods might be responsible for overtaxing manual labourers to the detriment of viable productivity.

2.1. *Description of the two mixing methods evaluated*

1 A procedure described by Addis [15] for hand mixing by shovel on the ground "requiring less labour" was adopted. A shovel was used to spread the sand in a roughly circular layer some 75 to 100 mm thick on a flat concrete slab mixing platform. The cement was then spread evenly over the sand and mixed until the two blended uniformly in



E.M. Peters

Edith Peters is currently lecturing in Exercise Physiology in the Department of Physiology of the Medical School of the University of Natal. Edith completed her MSc (med) degree in Sport Science in the Department of Physiology of UCT Medical School in 1984 and is presently completing her PhD in Sports Nutrition at the University of Cape Town. She has published widely in the fields of exercise physiology and sports nutrition and was responsible for co-ordinating the Sport Science Courses and directing the Sport Science Laboratory in the Division of Physical Education at the University of the Witwatersrand between 1989 and 1997.

colour. The materials were then shovelled into a flat heap with a saucer-shaped depression in the centre, into which about half* the water was poured. The wet mixing was then carried out by shovelling material from the edges of the heap into the centre, turning over each shovelful as it was dumped. The remainder of the water was then added slowly as the material was turned over, by shovelling from the centre to the side, and back to the centre of the heap. Mixing terminated when no dry ingredients remained and the colour and consistency of the mix appeared uniform.

2 A simple low cost 90/60 ℓ manually powered drum mixer manufactured by "New Dawn Engineering" [16] in Manzini, Swaziland was used. The mixer (see Fig 1) consists of an inclined drum with four revolving handles mounted perpendicular to its axis of rotation and opposed 90° apart. The ingredients are homogenised by three sets of seven internal chain "fingers" radiating from a common ring at the centroid of the drum and terminating at its circumference, which "comb" the mix as it falls under gravity. The drum is rotated by continually pulling and/or pushing on each of the four revolving handles. The mixer was charged in accordance with the manufacturers instructions [16] and slowly turned 44 "handles" (11 revolutions) at a speed of approximately seven revolutions per minute.

3.0 EXPERIMENTAL

3.1 Fieldwork

Two trial batches, followed by three experimental batches of mortar, were mixed using each of the two mixing methods; yielding six test batches in total. The volume of material mixed in every batch was set at 60 ℓ of wet output to make optimal use of the 90/60 ℓ capacity of the drum mixer. This output was achieved by trial and error and was made up of the following ingredients:-

Cement (CEMII/A-V 32.5)	14 dm^3
Decomposed Granite Sand (FM 3.02)	70 dm^3
Lime	4 dm^3
Water	16 ℓ

Lime was added to control bleeding in order to facilitate an average slump of 50 mm (typical of the 30 - 70 mm slump used for rubble masonry construction [2]. For objective comparison, time and energy recordings commenced after the ingredients had been measured into volume gauges and terminated immediately after the mixed mortar had been placed into a wheelbarrow. Data were recorded with a heart-rate monitor (Vantage XL, Polar USA, Stamford, Connecticut, USA) consisting of an unobtrusive recorder transmitter, attached by an elastic strap around the subject's abdomen just beneath the pectoral muscles. The heart rate

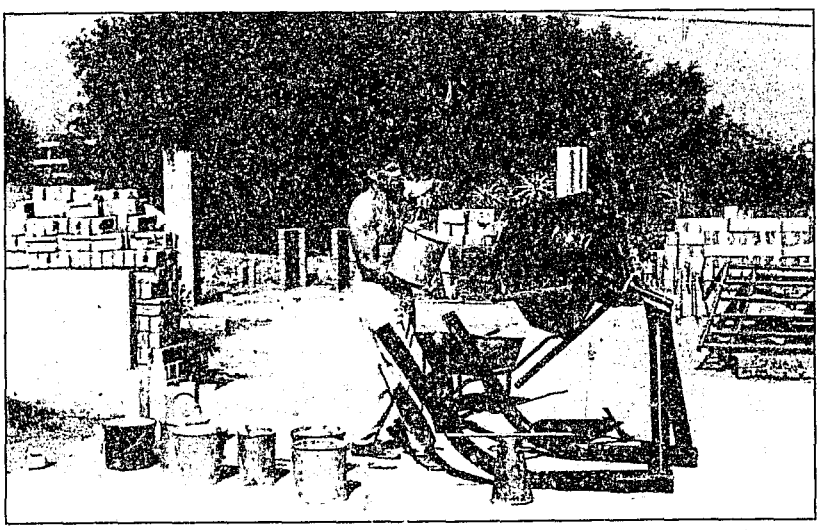

Fig 1:Manually powered 90/60 drum mixer being charged with ingredients during a physiological evaluation test

** The addition of all the water at one time may result in the washing away of the cement and fines from the heap as runnels of water escape from it.*



R.T. McCutcheon

Professor Robert McCutcheon obtained his B.Sc. (Eng) in 1963 and a Graduate Diploma in Engineering in 1964 from the University of the Witwatersrand. Between 1965 and 1971 he worked as an engineer and chairman of the South African Voluntary Service (SAVS). During the SAVS work he developed an interest in community development in relation to housing and the provision of low-cost infrastructure. From 1975 to 1987 he worked in Iran, Botswana and East Africa. Since then he has been a Professor of Civil Engineering at Wits. In 1994, he was appointed Head of Department of Civil Engineering. He is author of a Green Paper on Job Creation in Transport and Public Works, Western Cape.

signal was transmitted to a receiver attached to the subject's wrist which was programmed to store heart rate readings every five seconds. These data were subsequently transmitted via an interface (Polar Computer Interface, Polar USA) onto a computer. Typical response curves are presented in Fig 2 and a summary of notable data is presented in Table 1.

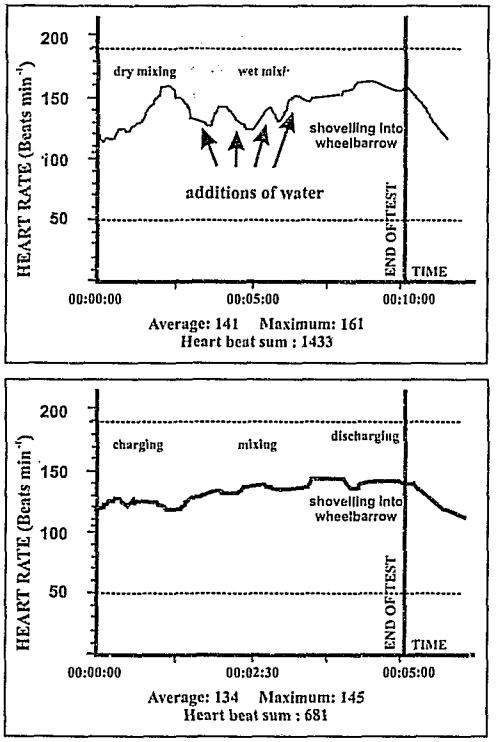

Fig 2: Typical heart rate response curves to manual mixing or mortar. A.Manual mixing on the ground with a shovel. B.Mixing with a manually powered drum mixer. The dotted lines indicate the subject's maximal heart rate (MHR) (predicted using the formulaa 220 minus age in years)[17] and lower heart rate threshold respectively.

	Shovel Mixing			Manual Drum Mixing		
	Mix Time min:sec	HR max (beats.min $^{-1}$)	HR Average (beats.min $^{-1}$)	Mix Time min:sec	HR max (beats.min $^{-1}$)	HR Average (beats.min $^{-1}$)
1st Test	09:20	156	138	05:00	142	132
2nd Test	09:30	158	140	05:05	145	134
3rd Test	10:10	161	141	05:05	159	137
Mean $\bar{x}$	09:40	158.3	139.7	05:03	148.7	134.3
Std dev σ	00:26	2.5	1.5	00:03	9.1	2.5
Coeff $\sigma/\bar{x}$	4.6%	1.6%	1.1%	0.1%	6.1%	1.9%

Table 1: Mixing times, maximum and average heart rates (HR, mean values $\bar{x}$, standard deviations from the mean values σ and co-effcients of variation expressed as percentages of the mean values $\sigma/\bar{x}$ for both methods of mixing.



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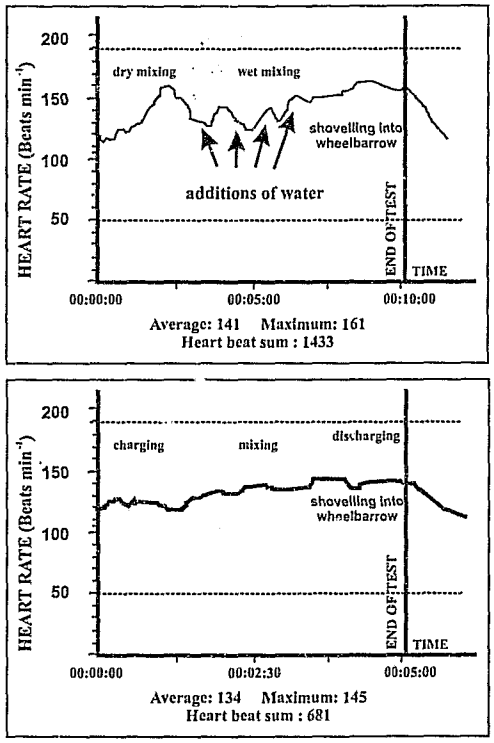

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Two sets of three cube specimens were tested after 28 days, by the Cement and Concrete Institute, to ascertain the coefficient of variation of cube strength as an indicator of the extent of homogeneity achieved by each mixing method (see Table 2).

RECORDED CUBE STRENGTHS AT 28 DAYS (MPa)	SHOVEL MIXING	MANUAL DRUM MIXING
	6.2 6.9 7.8	6.4 6.4 6.9
Mean $\bar{x}$	7.0	6.6
Std deviation from mean σ	0.79	0.28
Coefficient of variation $\sigma/\bar{x}$	11.3%	4.2%

Table 2 : Cube strength statistics as a preliminary indicator of homogeneity of mixing

An 80 kg, 30 year old male (non-smoker) in good health and finess was used as the subject for all tests to maintain a consistent basis for comparisons. Alternation of both mixing techniques from one batch to the next precluded the possibility of fatigue from prejudicing the results of either method of mixing as the subject tired toward the end of a day of testing.

3.2. *Calibration*

Work physiologists [10,11,12] have shown a strong correlation between heart-rate and oxygen consumption of subjects at sub-maximal workloads using treadmill, bicycle and snow ski ergometers which utilise the body's largest and most powerful muscle groups found in the legs. However, there appears to be a dearth of information on the correlation between oxygen consumption and upper-body activity which utilises the smaller muscle groups involved during the mixing of mortar. To estimate the energy expenditure of the subject during sub-maximal manual mortar mixing, heart-rate was correlated to oxygen consumption. Since the efficiency of metabolic conversion is largely dependent upon the mass of muscle taxed, it is desirable to simulate the activity in question as closely as possible whilst performing such calibrations.

Consequently, an arm crank ergometer (Monark 881 Sweden) with its axis mounted 1.28 m above floor level, was used to exercise upper body muscles at a cadence of 75 RPM to mimic the activity of mixing concrete as closely as practically possible in the Sport Science Laboratory at the Division of Physical Education at the University of Witwatersrand. The subject was connected to a stationary on-line metabolic analyser (Oxygon 4, Mijnhardt, Bunnik, The Netherlands) and oxygen consumption (VO_2) and carbon dioxide production (VCO^2) were measured continuously via an open-circuit technique. The subject was fitted with a two-way Hans Rudolf valve (Model 7910, Hans Rudolf Inc., Kansas, USA) which was connected to the metabolic analyser by means of 30 mm tubing.

Time (Min)	Power Output (Watts)	Heart Rate ($b.min^{-1}$)	VO_2 ($l.min^{-1}$)	Power Input (Watts)	Efficiency Output/Input (%)	Energy Expenditure in 6 hrs (MJ)	Daily energy requirement to work at this rate (inc. basal metabolic requirement for 18 hrs) (MJ)
1-3	0	64	0.26	87.62	0	1.89	7.57
3-4	7.5	109	0.60	202.2	3.7	4.37	10.05
4-5	15	110	0.78	262.9	5.7	5.68	11.35
5-6	30	123	0.99	333.6	9.0	7.21	12.89
6-7	45	129	1.13	380.8	11.8	8.23	13.91
7-8	60	140	1.39	468.4	12.8	10.12	15.80
8-9	75	151	1.51	508.9	14.7	10.99	16.67
9-10	90	157	1.75	589.8	15.3	12.74	18.42
10-11	105	162	2.01	677.4	15.5	14.63	20.30
11-12	120	169	2.18	734.7	16.3	15.87	21.55
12-13	135	176	2.38	802.1	16.8	17.32	23.00
13-14	150	179	2.62	882.9	17.0	19.07	24.75
14-15	165	182	2.81	947.0	17.4	20.45	26.13

Table 3: Heart rate/VO_2 relationship during sub-maximal arm ergometry to simulate manual mixing of concrete

The subject's basal oxygen consumption was measured for three minutes prior to commencement of the test. Thereafter, following a warm-up at 7,5 Watts, the workload was increased at the end of every minute for twelve minutes until the subject began to fatigue. The arm ergometer calibration is presented in Table 3 and the heart-rate VO_2 (oxygen consumption) relationship is presented in Fig 3.

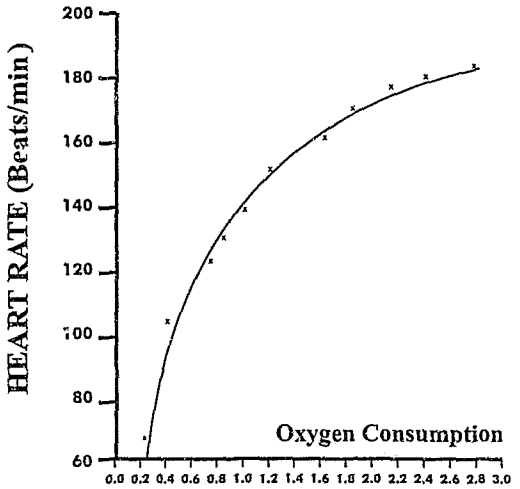
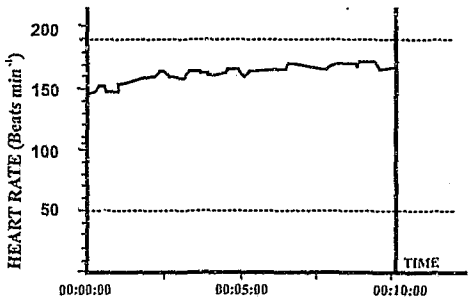


Fig 3: Logarithmic sub-maximal heart-rate (/min) $V0_2$ relationship for the test subject during arm ergometry to simulate the manual mixing of mortar



Average: 153 Maximum: 164 Heart beat sum: 1538

Fig 4 : Heart-rate response to continuous drum mixing at a constant handle effort of 180 Newtons

3.3. *Validation of simulation and efficiency*

The ability of the arm ergometer to simulate the activity of mortar mixing was validated by comparing heart-rate response during arm ergometry (Table 3) with that in response to continuous drum mixing (Fig 4). The Mechanical work output to turn the drum mixer at a specific speed is simply the product of average handle effort required to turn the fully charged drum, and the circumference travelled by the centre of the handles in a given time. The average handle effort, measured with a spring balance, was 180 Newtons and the handle circumference was 5.34m. Thus, at a speed of 7 revolutions per minute, 6.728kJ of mechanical work was done per minute; a power output of 112 Watts. During a long period of sustained mixing activity at this rate (with the drum mixer loaded to capacity), the maximum heart rate response was recorded as 164 beats per minute. This compares very favourably to the heart rate response of 165 beats per minute (interpolated from Table 3) recorded during arm ergometery.

4.0 DISCUSSION

Interpolation of power input (Table 3), to correspond with the average heart-rate responses (derived from Table 1), can be used to reveal the metabolic power and energy requirements to yield 60 / of mortar (about 140 kg) for each mixing method considered. Shovel mixing of mortar at 466 Watts (interpolated) consumed

466 W X 580 s = 270.3 kJ per batch Eqn 1

whilst manual powered drum mixing at 423 Watts (interpolated) consumed

423W X 303 s = 128.2 kJ per batch Eqn 2

Thus, the human energy cost of a cubic metre of shovel mixed mortar is

270.3 kJ X (1000 ÷ 60) = 4.505 MJ/m³ Eqn 3

This is more than twice the energy cost of the drum mixed mortar which is

128.2 kJ X (1000 ÷ 60) = 2.136 MJ/m³. Eqn 4

Thus, it follows that healthy male labourers, mixing mortar at maximum capacity, may achieve a daily output of just over 2 m³/labourer/day, shovel-mixing, and just under 5 m³/labourer/day, drum mixing. However, the reader is reminded that this output is strictly relevant to the task of mixing alone, and does not include tasks of batching and transporting which are commonly included in the job descriptions of mixer labourers. Although the energy consumed by these two activities will substantially reduce the above

estimates, it is extremely project specific and difficult to quantify and therefore beyond the scope of this investigation. However, Rankine [8] (1865) provides a rule of thumb estimate which held for 40 years in an era when manual mixing was commonplace:- *"The labour of mixing mortar by shovel may be estimated at about ¾ of a day's work of a man per cubic yard"*. This is equal to one cubic metre per labourer per day. Thus, it would appear that the tasks of batching and transporting, associated with mixing, are probably as onerous as shovel mixing itself.

The values derived from Eqns 3 & 4 are both significantly lower than those derived from the investigation conducted by de Beer [7] which determined the energy required to mix concrete by shovel on the ground as being 8.178 MJ/m^3 at a metabolic rate of 514 Watts. Thus, it would appear that shovel mixing of concrete consumes about 80% more energy than shovel-mixing of mortar. The additional activity of folding stone into the mix increases both the resistance to shovel penetration and effort required to turn the heap. There is no doubt that mixing concrete and/or mortar by shovel is an extremely tiring and inefficient job. This finding is confirmed by Grandjean [16] who reports the efficiency of "shoveling in a stooped posture" to be 3% and rates it as the most inefficient activity out of a list of 16 common physical tasks. Scholz and Sieber have shown that working in a stooped posture can increase work energy expenditure by as much as 25% and heart-rate by 20% [18]. In addition to being extremely inefficient, working in a stooped posture, particularly if the knees are not bent, imposes very high stress on the discs between the lumbar vertebrae and may eventually lead to disc degeneration; a common cause of premature disability.

Whilst the manufacturer of the drum mixer under investigation ought to be commended for successfully producing an affordable, low maintenance and easily constructable device in harmony with the philosophy of employment creation, this initiative should be viewed as a point of departure. The mixing of mortar could be defined as the uniform redistribution of ingredients within a given volume. Thus, the energy requirement to achieve homogenisation need not be significant provided the task is tackled intelligently to avoid excessive random multiple handling. The chain "fingers" of the device are a crude alternative to blades. The chains offer great resistance to the sliding of the mixture against the inside of the drum and homogenisation occurs rather haphazardly by "raking" the sliding conglomerate rather than by folding together the ingredients. A blade configuration, optimally deployed, to deliberately lift and fold the ingredients whilst offering the least possible resistance would be an obvious progression to increase the mechanical efficiency of this machine. A further improvement in efficiency ought to result from the incorporation of at least one extra handle since the present arrangement demands a long range of effort between handle changes, necessitating an awkward snatch between pulling and pushing at about mid-stroke, which tends to stall the momentum and wrench the elbow and shoulder joints.

4.1 Energy requirement in perspective

Column 6 of Table 3 presents the food energy requirement of the test subject to perform 6 hours of work per day at each of the work output levels simulated during arm ergometry. By interpolation of heart-rate response between 129 and 140 b.min^{-1}, it follows that about 9.14 MJ of energy would be required to perform 6 hours mortar mixing with the manually powered drum mixer and about 10.07 MJ would be required to mix mortar by shovel. Several work physiologists [19,20,21] share the opinion that this energy requirement borders the threshold of the maximum sustainable daily work energy available to healthy males of about 10.5 MJ per work day. The quantity of food needed to provide this much energy might be more readily appreciated when reduced to equivalent loaves of bread.

An 800g loaf of brown bread has an energy content of 8.056 MJ [22]. Thus, six hours of mixing mortar by either of the two methods consumes more energy than that contained in a loaf of bread. However, unlike machines, humans do not only consume energy whilst they are working. Essential life functions like breathing consume energy all the time. The power required to sustain life, known as the basal metabolic rate (BMR), was in the case of this test subject, 87.6 Watts (see top of column 4 in Table 3). The right hand column of Table 3 shows the total daily energy requirement of the test subject; the sum of the energy requirement to mix concrete for six hours per day, plus the basal metabolic requirement to rest for the remaining 18 hours. Thus, by interpolation, it follows that to perform 6 hours work mixing mortar by manually powered drum mixer would demand replenishment of 14.83 MJ/day and the same time of shovel mixing would demand 15.78 MJ/Day. Thus, the shovel mixer needs to replenish himself with the equivalent amount of energy found in two loaves of bread just to sustain his existence for the sole purpose of mixing mortar.

In reality, labourers need even more energy than this to meet other demands placed on their metabolic systems such as fetching water and firewood, bathing, cooking and walking to work. This need has been termed "leisure requirement" which in the case of a typical adult male, is estimated to consume about 2.5MJ/day [19,20,21]. Therefore, a healthy productive male mortar mixer probably requires closer to three loaves of bread per day to replenish himself. Thus, the remuneration of R7 per labourer per day, recommended by the Government's National Public Works Program's Target Focus Group [23], is unlikely to sustain a mixer labourer on a staple diet of bread, let alone support his or her dependants! (Assuming

brown bread is available at a cost of between two and three Rand per loaf).

Another way to gain appreciation of the cost of human energy is to compare it with diesel fuel. A litre of diesel contains 37 to 39 MJ [24] of energy and currently costs about two Rand. Thus, the cost of diesel is about 5,3 cents per MJ. The energy cost of a healthy male labourer selling his maximum sustainable work output of 10.5 MJ/day [19,20,21] for the minimum remuneration of R7 is therefore

$$R7 \div 10.5\ MJ = 66\ cents/MJ \qquad \text{Eqn 5}$$

Thus, currently, the cost of human energy is at least 12 times the cost of diesel. This should not be interpreted by the protagonists of automation to imply that labour intensive methods are not competitive compared to conventional alternatives, since, unlike most existing construction machinery, labourers are versatile and capable of using intelligence towards greater effectiveness. Furthermore, human labourers are not subject to the many economic inefficiencies of machinery such as capital acquisition, insurance and depreciation costs. Antithetically, an appreciation of the need to develop technologies which conserve and optimally utilise human energy needs to be fostered amongst policy and decision makers.

5.0 HEALTH AND NUTRITIONAL STATUS

Unfortunately, the assumption of a healthy labour-force is, at present, most unrealistic. Vorster et al [25] have exhaustively reviewed the literature on the nutritional status an anthropometry of South Africans between 1975 and 1996 and concluded that "the health and nutritional status (malnourished as opposed to undernourished) of millions of adults is far from optimal; particularly amongst the destitute". Disease and malnourishment deplete the body of energy and capacity to work; often synergistically. Unhealthy people are less efficient and productive doing arduous work, (even if they are over nourished) because their bodies require more energy to rectify their conditions. Consequently, it makes sound economic sense to invest in the health care and promotion of sound nutritional habits amongst the labour force. Mainwaring [26] recalls his experience in Zimbabwe where unhealthy recruits to the labour force had to be treated for malaria and fed a balanced diet for a month before they built up the necessary fitness and stamina to perform the arduous manual work demanded of them, ' ' Beer [4] reports similar experience in South Afri during the building of Maritsane Dam. In his case, two weeks of feeding the labour force on a balanced diet proved sufficient time.

6. THE HOMOGENEITY OF MIX AS A FUNCTION OF THE DISTRIBUTION OF 28 DAY CUBE STRENGTH RESULTS

Unfortunately, only six cube moulds of a constant 150mm size were available to this initiative. More specimens might have indisputably confirmed the degree of homogeneity achieved by each mixing method. Nevertheless, Table 2 indicates a superior consistence throughout the mix achieved by the drum mixer. The coefficient of strength variation of the three shovel mixed cube specimens is nearly three times higher than that of the drum mixed cube specimens, despite the unsophisticated method of homogenization. The mean strength of the shovel mixed cubes is marginally higher than the drum mixed specimens; probably as a consequence of the absorption of some moisture from the mix by the porous concrete mixing slab.

7.0 CONCLUSION

A good correlation was found to exist between heart-rate and oxygen consumption during sub-maximal upper body activity on an arm ergometer simulating the action of mixing mortar. The accuracy of the arm ergometer's ability to mimic manual mortar mixing was validated by comparison of the energy conversion efficiencies during arm ergometry with calculated work done in turning the manually powered drum mixer at the same heart rate. Human energy was shown to cost (currently) at least 12 times as much as diesel and the current recommended remuneration of R7 per day for labourers was demonstrated to be entirely consumed just to sustain the replenishment of this energy on a staple diet of bread alone. Consequently, the need to efficiently utilise human energy is emphasised. Shovel mixing of mortar was found to be an extremely arduous, inefficient and potentially hazardous task, requring about 4.5MJ/m^3. Mixing mortar with the manually powered drum mixer was shown to be less taxing and more than twice as productive, requiring 2.1MJ/m^3. A limited statistical analysis of cube strength found greater homogeneity in a batch of drum mixed cubes compared with a batch of shovel mixed cubes. Consequently, recommendation is made to avoid shovel mixing of mortar and concrete and to encourage the technical refinement of manually powered mixers to make better use of limited human energy.

Acknowledgements:

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ESTIMATION OF THE HUMAN ENERGY COST TO MIX MORTAR MANUALLY

Roderick. G.D. Rankine, Edith M. Peters & Robert T. McCutcheon

Synopsis

A new method of estimating the human energy cost to mix mortar on the ground by shovel as well as by a simple manually powered drum mixer is described. The heart-rate response of a human subject engaged in sub-maximal upper-body activity simulating the mixing of concrete is shown to correlate closely with both oxygen consumption and mechanical work output. The test results confirm that shovel mixing is an arduous, inefficient and potentially hazardous activity. The manually powered drum mixer was found to be approximately twice as productive as shovel mixing on the ground, possessing vast potential for improved mechanical efficiency. The high cost of human energy is compared with the cost of energy derived from diesel fuel and the daily food requirement of mixer labourers is quantified in terms of the equivalent energy contained in loaves of bread. On this basis, the minimum wage of R7 per day, proclaimed by the National Public Works Program, is questioned.

1.0 INTRODUCTION

Rankine and McCutcheon[1] have identified large discrepancies in reported productivity amongst labourers engaged in the construction of rubble masonry structures. Production rates of as little as 0.2 to 0.3 m^3 of placed rubble masonry per labourer per day have been reported[2,3] in cases where mortar has been mixed on the ground by shovel, whereas rates as high as 0.8 to 1.9 m^3 per labourer per day are reported[4,5,6] where mortar was mixed mechanically. Hence, it seemed probable that the cause of the low productivity might be the result of exceptional energy taxation of the human body by shovel-mixing. A literature review revealed limited existing knowledge of the energy cost and productivity of mixing concrete[7] and mortar[8] by hand. de Beer[7] determined the human energy requirement to shovel-mix concrete by measuring the oxygen consumption of a subject with a Max Planck portable spirometer. Knowledge of a subject's oxygen consumption provides an ideal means for accurate quantification of human metabolic activity since 20.22 kJ of energy is liberated for every litre of oxygen consumed at Standard Temperature and Pressure[9]. Direct measurement of oxygen consumption can be achieved by open circuit spirometry. The volume of expired air and its carbon dioxide and oxygen content can be recorded using portable spirometers or very costly computerised metabolic analysers (metabolic carts). Such methods are not yet practical for measuring the energy requirements to mix mortar since the former are extremely costly and the validity of their data is reliant upon the accuracy of "auto-calibrated" gas analysers within the confines of their small carry cases; while the metabolic carts are cumbersome and heavy and only located in laboratory settings. Conventional metabolic research has therefore been confined to task simulations and data collection in the laboratory.

Numerous work physiologists[10,11,12] have identified a correlation between heart-rate and oxygen consumption in humans at sub-maximal workloads. By recording the relationship between heart-rate and oxygen consumption at known sub-maximal workloads, a heart-rate monitor may theoretically be used to predict metabolic response to another similar sub-maximal physical activity which taxes the same mass of muscle. If the mechanical power output can be measured, for example by an ergometer, then the energy conversion efficiency of a system can also be estimated. Modern heart-rate monitors are relatively inexpensive and well suited to complicated physical activities such as concrete mixing because, unlike spirometers, they are inexpensive, light-weight and do not bind the subject with tubes or wires, thereby restricting the range of movement. Heart-rate monitors were successfully used by Lambert et al[13] (1994), to estimate the energy expenditure of sugarcane cutters working in the field, and by Engebos[14] (1997), in a study parallel to this one, to estimate the energy expenditure of trench diggers.

2.0 OBJECTIVE

The objective of the experiment was to develop a technique to estimate the human energy cost of mixing mortar, of a consistence typically used in rubble masonry concrete, by two different manual mixing methods and to determine whether either or both of these manual methods might be responsible for overtaxing manual labourers to the detriment of viable productivity.

2.1 Description of the two mixing methods evaluated

1) A procedure described by Addis[19] for hand mixing by shovel on the ground " requiring least labour" was adopted. A shovel was used to spread the sand in a roughly circular layer some 75 to 100 mm thick on a flat concrete slab mixing platform. The cement was then spread evenly over the sand and mixed until the two blended uniformly in colour. The materials were then shovelled into a flat heap with a saucer shaped depression in the centre, into which about half* the water was poured. The wet mixing was then carried out by shovelling material from the edges of the heap into the centre, turning over each shovelful as it was dumped. The remainder of the water was then added slowly as the material was turned over, by shovelling from the centre to the side, and back to the centre of the heap. Mixing terminated when no dry ingredients remained and the colour and consistency of the mix appeared uniform.

2) A simple low cost 90/60 ℓ manually powered drum mixer manufactured by "New Dawn Engineering"[16] in Manzini, Swaziland was used. The mixer (see Fig 1) consists of an inclined drum with four revolving handles mounted perpendicular to its axis of rotation and opposed 90^0 apart. The ingredients are homogenised by three sets of seven internal chain "fingers" radiating from a common ring at the centroid of the drum and terminating at its circumference, which "comb" the mix as it falls under gravity. The drum is rotated by continually pulling and/or pushing on each of the four revolving handles. The mixer was charged in accordance with the manufacturers instructions[16] and slowly turned 44 "handles" (11 revolutions) at a speed of approximately seven revolutions per minute.

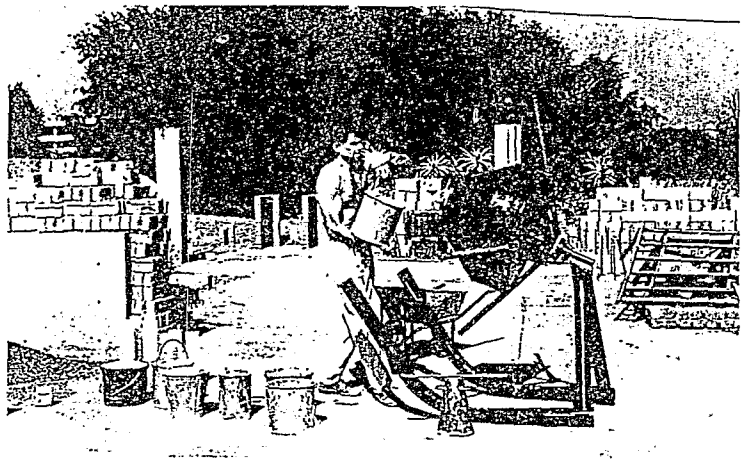

Fig 1 Manually powered 90/60 drum mixer being charged with ingredients during a physiological evaluation test.

* The addition of all the water at one time may result in the washing away of the cement and fines from the heap as runnels of water escape from it.

3.0 EXPERIMENTAL

3.1 Fieldwork

Two trial batches, followed by three experimental batches of mortar, were mixed using each of the two mixing methods; yielding six test batches in total. The volume of material mixed in every batch was set at 60 ℓ of wet output to make optimal use of the 90/60 ℓ capacity of the drum mixer. This output was achieved by trial and error and was made up of the following ingredients:-

Cement (CEMII/A-V 32.5)	14 dm^3
Decomposed Granite Sand (FM 3.02)	70 dm^3
Lime	4 dm^3
Water	16 ℓ

Lime was added to control bleeding in order to facilitate an average slump of 50 mm (typical of the 30-70 mm slump used for rubble masonry construction[2]). For objective comparison, time and energy recordings commenced after the ingredients had been measured into volume gauges and terminated immediately after the mixed mortar had been placed into a wheelbarrow. Data were recorded with a heart-rate monitor (Vantage XL, Polar USA, Stamford, Connecticut, USA) consisting of an unobtrusive recorder transmitter, attached by an elastic strap around the subject's abdomen just beneath the pectoral muscles. The heart rate signal was transmitted to a receiver attached to the subject's wrist which was programmed to store heart rate readings every five seconds. These data were subsequently transmitted via an interface (Polar Computer Interface, Polar USA) onto a computer. Typical response curves are presented in Fig 2 and a summary of notable data is presented in Table 1.

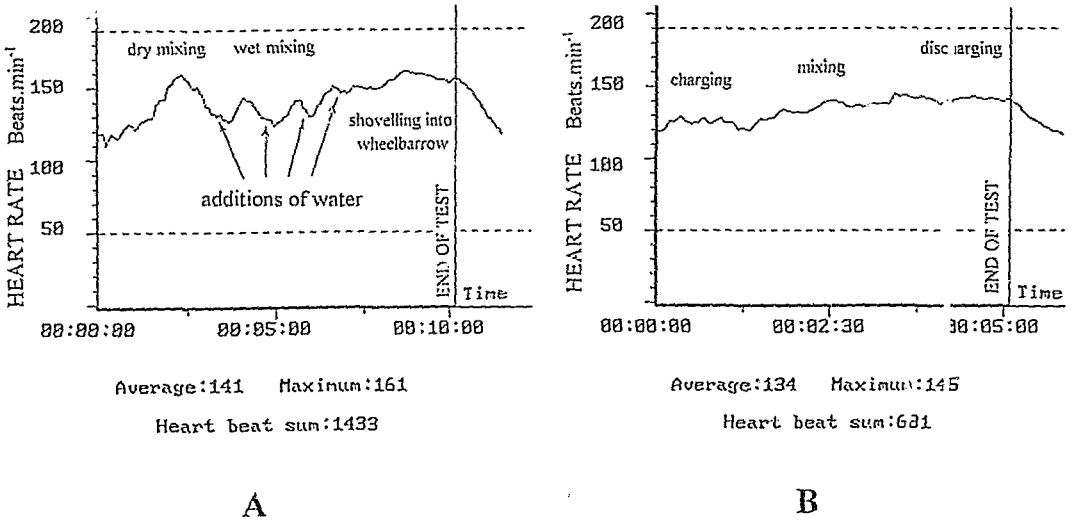

Fig 2 Typical heart rate response curves to manual mixing of mortar. A. Manual mixing on the ground with a shovel. B. Mixing with a manually powered drum mixer. The dotted lines indicate the subject's maximal heart rate(MHR) (predicted using the formula 220 minus age in years)[17] and lower heart rate threshold respectively

	Shovel mixing			Manual drum mixing		
	Mix Time min:sec	HR Max (beats.min^{-1})	HR Average (beats.min^{-1})	Time min:sec	HR Max (beats.min^{-1})	HR Average (beats.min^{-1})
1st Test	09:20	156	138	05:00	142	132
2nd Test	09:30	158	140	05:05	145	134
3rd Test	10:10	161	141	05:05	159	137
Mean ×	09:40	158.3	139.7	05:03	148.7	134.3
Std dev σ	00:26	2.5	1.5	00:03	9.1	2.5
Coeff σ/×	4.6%	1.6%	1.1%	0.1%	6.1%	1.9%

Table 1 Mixing times, maximum and average heart rates (HR), mean values ×, standard deviations from the mean values σ and coefficients of variation expressed as percentages of the mean values σ/× for both methods of mixing.

Two sets of three cube specimens were tested after 28 days, by the Cement and Concrete Institute, to ascertain the coefficient of variation of cube strength as an indicator of the extent of homogeneity achieved by each mixing method (see Table 2).

	SHOVEL MIXING	MANUAL DRUM MIXING
RECORDED CUBE STRENGTHS AT 28 DAYS (MPa)	6.2 6.9 7.8	6.4 6.4 6.9
Mean ×	7.0	6.6
Std deviation from the mean σ	0.79	0.28
Coefficient of variation σ/×	11.3%	4.2%

Table 2 Cube strength statistics as a preliminary indicator of homogeneity of mixing.

An 80 kg, 30-year old male (non-smoker) in good health and fitness was used as the subject for all tests to maintain a consistent basis for comparisons. Alternation of both mixing techniques from one batch to the next precluded the possibility of fatigue from prejudicing the results of either method of mixing as the subject tired toward the end of the day of testing.

3.2 *Calibration*

Work physiologists[10,11,12] have shown a strong correlation between heart-rate and oxygen consumption of subjects at sub-maximal workloads using treadmill, bicycle and snow ski ergometers which utilise the body's largest and most powerful muscle groups found in the legs. However, there appears to be a dearth of information on the correlation between oxygen consumption and upper-body activity which utilises the smaller muscle groups involved during the mixing of mortar. To estimate the energy expenditure of the subject during sub-maximal manual mortar mixing, heart-rate was correlated to oxygen consumption. Since the efficiency of metabolic conversion is largely dependent upon the mass of muscle taxed, it is desirable to simulate the activity in question as closely as possible whilst performing such a calibration.

Consequently, an arm crank ergometer (Monark 881 Sweden) with its axis mounted 1.28 m above floor level, was used to exercise upper body muscles at a cadence of 75 RPM to mimic the activity of mixing concrete as closely as practically possible in the Sport Science Laboratory at the Division of Physical Education at the University of the Witwatersrand. The subject was connected to a stationary on-line metabolic analyser (Oxygon 4, Mijnhardt, Bunnik, The Netherlands) and oxygen consumption (VO_2) and carbon dioxide production (VCO^2) were measured continuously via an open-circuit technique (see Fig 3). The subject was fitted with a two way Hans Rudolf valve (Model 7910. Hans Rudolf Inc., Kansas, USA) which was connected to the metabolic analyser by means of 30 mm tubing.

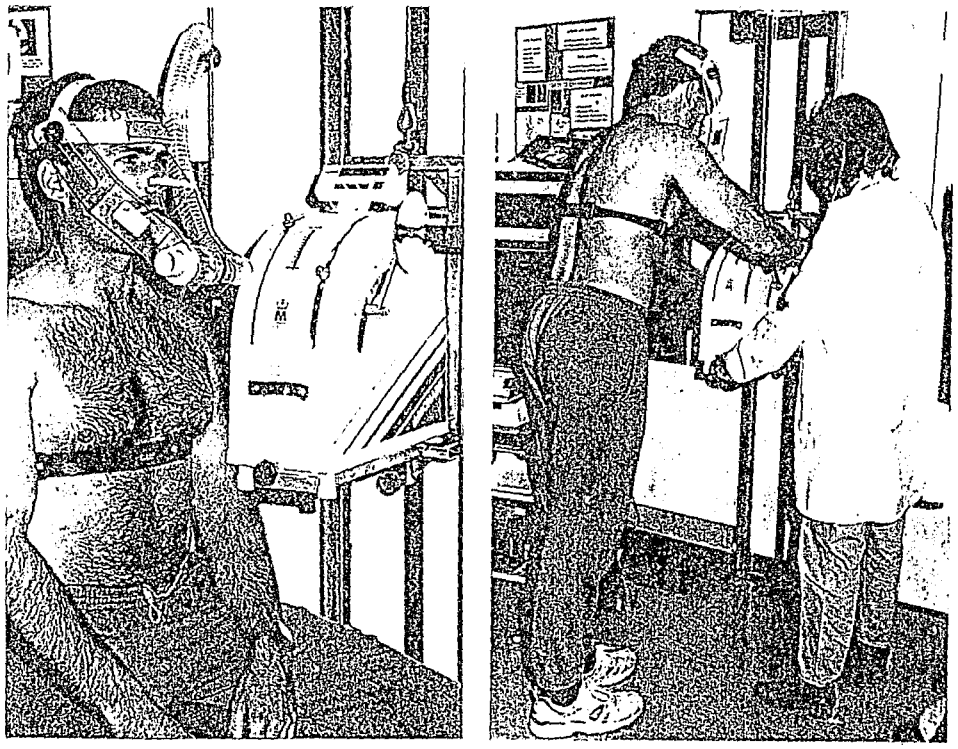

Fig 3 Arm ergometer, heart-rate monitor (strapped around the abdomen below the pectoral muscles) and respiratory gas collection apparatus used to establish the correlation of the heart-rate of the subject with oxygen consumption in response to upper-body sub-maximal work output (left). Subject engaged in arm cranking ergometry in a slightly stooped standing position (right).

The subject's basal oxygen consumption was measured for three minutes prior to commencement of the test. Thereafter, following a warm-up at 7.5 Watts, the workload was increased at the end of every minute for twelve minutes until the subject began to fatigue. The arm ergometer calibration is presented in Table 3 and the heart-rate:VO_2 (oxygen consumption) relationship is presented in Fig 4.

Time (Min)	Power Output (Watts)	Heart Rate ($b.min^{-1}$)	VO_2 ($\ell.min^{-1}$)	Power Input (Watts)	Efficiency Output/Input (%)	Energy Expenditure in 6 hrs (MJ)	Daily energy requirement to work at that rate (incl basal metabolic requirement for 18 hrs) (MJ)
1-3	0	64	0.26	87.62	0	1.89	7.57
3-4	7.5	109	0.60	202.2	3.7	4.37	10.05
4-5	15	110	0.78	262.9	5.7	5.68	11.35
5-6	30	123	0.99	333.6	9.0	7.21	12.89
6-7	45	129	1.13	380.8	11.8	8.23	13.91
7-8	60	140	1.39	468.4	12.8	10.12	15.80
8-9	75	151	1.51	508.9	14.7	10.99	16.67
9-10	90	157	1.75	589.8	15.3	12.74	18.42
10-11	105	162	2.01	677.4	15.5	14.63	20.30
11-12	120	169	2.18	734.7	16.3	15.87	21.55
12-13	135	176	2.38	802.1	16.8	17.32	23.00
13-14	150	179	2.62	882.9	17.0	19.07	24.75
14-15	165	182	2.81	947.0	17.4	20.45	26.13

Table 3 Heart rate/VO_2 relationship during sub-maximal arm ergometry to simulate manual mixing of concrete.

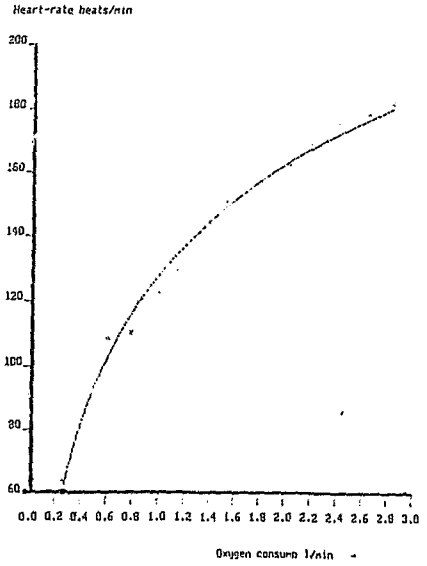

Fig 4 Logarithmic sub-maximal heart-rate:VO_2 relationship for the test subject during arm ergometry to simulate the manual mixing of mortar.

3.3 *Validation of simulation and efficiency*

The ability of the arm ergometer to simulate the activity of mortar mixing was validated by comparing heart-rate response during arm ergometry (Table 3) with that in response to continuous drum mixing (Fig 5). The mechanical work output to turn the drum mixer at a specific speed is simply the product of average handle effort required to turn the fully charged drum, and the circumference travelled by the centre of the handles in a given time. The average handle effort, measured with a spring balance (see Fig 6), was 180 Newtons and the handle circumference was 5.34m. Thus, at a speed of 7 revolutions per minute, 6.728 kJ of mechanical work was done per minute; a power output of 112 Watts. During a long period of sustained mixing activity at this rate (with the drum mixer loaded to capacity), the maximum heart rate response was recorded as 164 beats per minute. This compares very favourably to the heart rate response of 165 beats per minute (interpolated from table 3) recorded during arm ergometery.

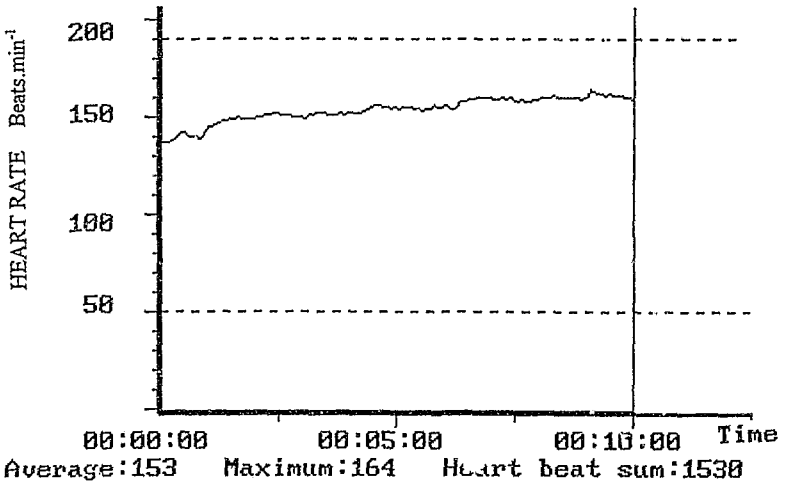

Fig 5 Heart-rate response to continuous drum mixing at a constant handle effort of 180 Newtons.

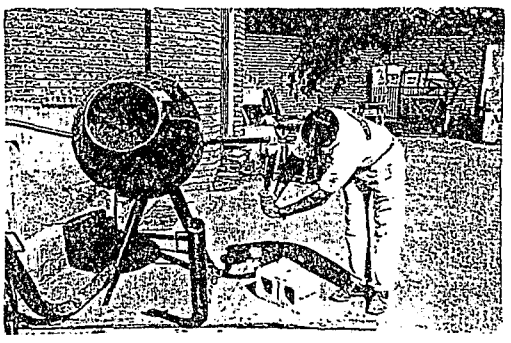

Fig 6 Spring balance used to quantify effort required to revolve mixer.

4.0 DISCUSSION

Interpolation of power input (Table 3), to correspond with the average heart-rate responses (derived from Table 1), can be used to reveal the metabolic power and energy requirements to yield 60 ℓ of mortar (about 140 kg) for each mixing method considered. Shovel mixing of mortar at 466 Watts (interpolated) consumed

$$466\ W\ X\ 580\ s\ =\ 270.3\ kJ\ per\ batch \qquad Eqn\ 1$$

whilst manual powered drum mixing at 423 Watts (interpolated) consumed

$$423\ W\ X\ 303\ s\ =\ 128.2\ kJ\ per\ batch. \qquad Eqn\ 2$$

Thus, the human energy cost of a cubic metre of shovel mixed mortar is

$$270.3\ kJ\ X\ (1000 \div 60)\ =\ 4.505\ MJ/m^3. \qquad Eqn\ 3$$

This is more than twice, the energy cost of the drum mixed mortar which is

$$128.2\ kJ\ X\ (1000 \div 60)\ =\ 2.136\ MJ/m^3. \qquad Eqn\ 4$$

Thus, it follows that healthy male labourers, mixing mortar at maximum capacity, may achieve a daily output of just over 2 m^3/labourer/day, shovel-mixing, and just under 5 m^3/labourer/day, drum mixing. However, the reader is reminded that this output is strictly relevant to the task of mixing alone, and does not include tasks of batching and transporting which are commonly included in the job descriptions of mixer labourers. Although the energy consumed by these two activities will substantially reduce the above estimates, it is extremely project specific and difficult to quantify and therefore beyond the scope of this investigation. However, Rankine[8] (1865) provides a rule of thumb estimate which held for 40 years in an era when manual mixing was commonplace:- *"The labour of mixing mortar by shovel may be estimated at about 3/4 of a day's work of a man per cubic yard".* This is equal to one cubic metre per labourer per day. Thus, it would appear that the tasks of batching and transporting, associated with mixing, are probably as onerous as shovel mixing itself.

The values derived from Eqns 3 & 4 are both significantly lower than those derived from the investigation conducted by de Beer[7] which determined the energy required to mix concrete by shovel on the ground as being 8.178 MJ/m^3 at a metabolic rate of 514 Watts. Thus, it would appear that shovel-mixing of concrete consumes about 80% more energy than shovel-mixing of mortar. The additional activity of folding stone into the mix increases both the resistance to shovel penetration and effort required to turn the heap. There is no doubt that mixing concrete and/or mortar by shovel is an extremely tiring and inefficient job. This finding is confirmed by Grandjean[18] who reports the efficiency of "shovelling in a stooped posture" to be 3% and rates it as the most inefficient activity out of a list of 16 common physical tasks. Scholz and Sieber have shown that working in a stooped posture can increase work energy expenditure by as much as 25% and heart-rate by 20%[18]. In addition to being extremely inefficient, working in a stooped posture, *particularly if the knees are not bent, imposes very high stress on the discs* between the lumbar vertebrae and may eventually lead to disc degeneration; a common cause of premature disability. Such posture is common amongst South African labourers and is shown in Fig 7.



Fig 7 Stooped posture with "straight knees" whilst mixing mortar is a common cause of back distress. Such posture is common amongst the South African labour-force.

Whilst the manufacturer of the drum mixer under investigation ought to be commended for successfully producing an affordable, low maintenance and easily constructable device in harmony with the philosophy of employment creation, this initiative should be viewed as a point of departure. The mixing of mortar could be defined as the uniform redistribution of ingredients within a given volume. Thus, the energy requirement to achieve homogenisation need not be significant provided the task is tackled intelligently to avoid excessive random multiple handling. The chain "fingers" of the device are a crude alternative to blades. The chains offer great resistance to the sliding of the mixture against the inside of the drum and homogenisation occurs rather haphazardly by "raking" the sliding conglomerate rather than by folding together the ingredients. A blade configuration, optimally deployed, to deliberately lift and fold the ingredients whilst offering the least possible resistance would be an obvious progression to increase the mechanical efficiency of this machine. A further improvement in efficiency ought to result from the incorporation of at least one extra handle since the present arrangement demands a long range of effort between handle changes, necessitating an awkward snatch between pulling and pushing at about mid-stroke, which tends to stall the momentum and wrench the elbow and shoulder joints (see Fig 8).

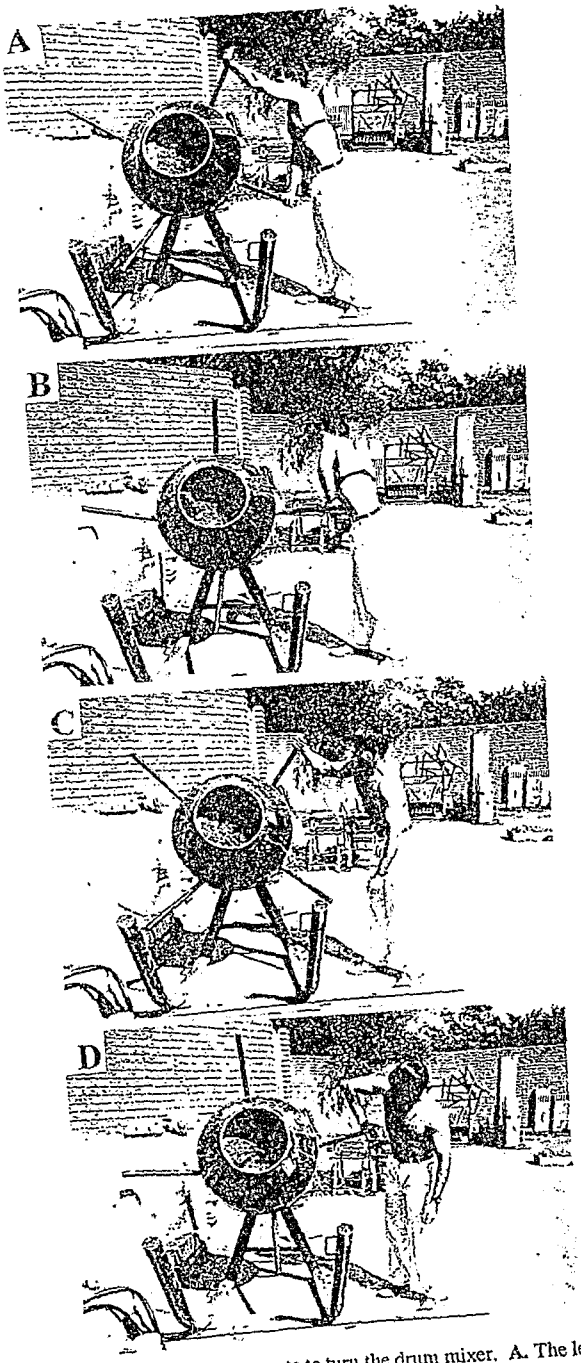

Fig 8 Photographic series of upper-body movements to turn the drum mixer. A. The left hand grasps the handle pulling it downwards and towards the abdomen using the latisimus dorsi muscles. B. The range of this movement ends before the drum is rotated a full 90°, (before the next handle can be reached with the right hand), necessitating a reverse-snatch movement of the left elbow mid-stroke to enable the left triceps to complete the stroke. This causes a temporary loss of momentum and might subject the shoulder and elbow to distress. C. Right hand pull-down. D. right elbow reverse-snatch.

4.1 Energy requirement in perspective

Column 6 of Table 3 presents the food energy requirement of the test subject to perform 6 hours of work per day at each of the work output levels simulated during arm ergometry. By interpolation of heart-rate response between 129 and 140 b min^{-1}, it follows that about 9.14 MJ of energy would be required to perform 6 hours mortar mixing with the manually powered drum mixer and about 10.07 MJ would be required to mix mortar by shovel. Several work physiologis s[19,20,21] share the opinion that this energy requirement borders the threshold of the maximum sustainable daily work energy available to healthy males of about 10.5 MJ per work day. The quantity of food needed to provide this much energy might be more readily appreciated when reduced to equivalent loaves of bread.

An 800 g loaf of brown bread has an energy content of 8.056 MJ[22]. Thus, six hours of mixing mortar by either of the two methods consumes more energy than that contained in a loaf of bread. However, unlike machines, humans do not only consume energy whilst they are working. Essential life functions like breathing consume energy all the time. The power required to sustain life, known as the basal metabolic rate (BMR), was in the case of this test subject, 87.6 Watts (see top of column 4 in Table 3). The right hand column of Table 3 shows the total daily energy requirement of the test subject: the sum of the energy requirement to mix concrete for six hours per day, plus the basal metabolic requirement to rest for the remaining 18 hours. Thus, by interpolation, it follows that to perform 6 hours work mixing mortar by manually powered drum mixer would demand replenishment of 14.83MJ/day and the same time of shovel mixing would demand 15.78 MJ/day. Thus, the shovel mixer needs to replenish himself with the equivalent amount of energy found in two loaves of bread just to sustain his existence for the sole purpose of mixing mortar.

In reality labourers need even more energy than this to meet other demands placed on their metabolic systems such as fetching water and firewood, bathing, cooking and walking to work. This need has been termed "leisure requirement" which in the case of a typical adult male, is estimated to consume about 2.5 MJ/day[19,20,21]. Therefore, a healthy productive male mortar mixer probably requires closer to three loaves of bread per day to replenish himself. Thus, the remuneration of R 7 per labourer per day, recommended by the Government's National Public Works Program's Target Focus Group[23], is unlikely to sustain a mixer labourer on a staple diet of bread, let alone support his or her dependants! (Assuming brown bread is available at a cost of between two and three Rand per loaf)

Another way to gain appreciation of the cost of human energy is to compare it with diesel fuel. A litre of diesel contains 37 to 39 MJ[24] of energy and currently costs about two Rand. Thus, the cost of diesel is about 5.3 cents per MJ. The energy cost of a healthy male labourer selling his maximum sustainable work output of 10.5 MJ/day[19,20,21] for the minimum remuneration of R7 is therefore

$$R7 \div 10.5\ MJ = 66\ cents/MJ \qquad Eqn\ 5$$

Thus currently, the cost of human energy is at least 12 times the cost of diesel. This should not be interpreted by the patrons of automation to imply that labour intensive methods are not competitive compared to conventional alternatives, since, unlike most existing construction machinery, labourers are versatile and capable of using intelligence towards greater effectiveness. Furthermore, human labourers are not subject to the many economic inefficiencies of machinery such as capital acquisition, insurance and depreciation costs. Antithetically, an appreciation of the need to develop technologies which conserve and optimally utilise human energy needs to be fostered amongst policy and decision makers.

5.0 HEALTH AND NUTRITIONAL STATUS

Unfortunately, the assumption of a healthy labour-force is, at present, most unrealistic. Vorster et al[25] have exhaustively reviewed the literature on the nutritional status and anthropometry of South Africans between 1975

and 1996, and concluded that "the health and nutritional status (malnourished as opposed to undernourished) of millions of adults is far from optimal; particularly amongst the destitute". Disease and malnourishment deplete the body of energy and capacity to work; often synergistically. Unhealthy people are less efficient and productive doing arduous work, (even if they are over-nourished), because their bodies require more energy to rectify their conditions. Consequently, it makes sound economic sense to invest in the healthcare and promotion of sound nutritional habits amongst the labour-force. Mainwaring[26] recalls his experience in Zimbabwe where unhealthy recruits to the labour-force had to be treated for malaria and fed a balanced diet for a month before they built up the necessary fitness and stamina to perform the arduous manual work demanded of them. de Beer[4] reports similar experience in South Africa during the building of the Maritsane Dam. In his case two weeks of feeding the labour-force on a balanced diet proved sufficient time.

6.0 HOMOGENEITY OF MIX AS A FUNCTION OF THE DISTRIBUTION OF 28 DAY CUBE STRENGTH RESULTS

Unfortunately, only six cube moulds of a constant 150 mm size were available to this initiative. More specimens might have indisputably confirmed the degree of homogeneity achieved by each mixing method. Nevertheless, Table 2 indicates a superior consistency throughout the mix achieved by the drum mixer. The coefficient of strength variation of the three shovel-mixed cube specimens is nearly three times higher than that of the drum mixed cube specimens, despite the unsophisticated method of homogenisation. The mean strength of the shovel mixed cubes is marginally higher than the drum mixed specimens; probably as a consequence of the absorption of some moisture from the mix by the porous concrete mixing slab.

7.0 CONCLUSION

A good correlation was found to exist between heart-rate and oxygen consumption during sub-maximal upper-body activity on an arm ergometer simulating the action of mixing mortar. The accuracy of the arm ergometer's ability to mimic manual mortar mixing was validated by comparison of the energy conversion efficiencies during arm ergometry with calculated work done in turning the manually powered drum mixer at the same heart-rate. Human energy was shown to cost (currently) at least twelve times as much as diesel and the current recommended remuneration of R 7 per day for labourers was demonstrated to be entirely consumed just to sustain the replenishment of this energy on a staple diet of bread alone. Consequently, the need to efficiently utilise human energy is emphasised. Shovel mixing of mortar was found to be an extremely arduous, inefficient and potentially hazardous task, requiring about 4.5 MJ/m^3. Mixing mortar with the manually powered drum mixer was shown to be less taxing and more than twice as productive, requiring 2.1 MJ/m^3. A limited statistical analysis of cube strength found greater homogeneity in a batch of drum mixed cubes compared with a batch of shovel mixed cubes. Consequently, recommendation is made to avoid shovel mixing of mortar and concrete and to encourage the technical refinement of manually powered mixers to make better use of limited human energy.

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APPENDIX 5

INVESTIGATING THE USE OF LIME AS AN ADMIXTURE TO IMPROVE THE PERFORMANCE OF RUBBLE MASONRY CONCRETE MORTARS; WITH PARTICULAR REFERENCE TO THE PROBLEM OF BLEEDING

CONCRETE BETON

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Investigating the Use of Lime as an Admixture to Improve the Performance of Rubble Masonry Concrete Mortars; with Particular Reference to Bleeding PART ONE

TECHNICAL PAPER

Roderick GD Rankine and Prof RT McCutcheon

SYNOPSIS

Bleeding and the entrapment of bleed-water beneath large horizontal rock surfaces have been identified as inherent problems in Rubble Masonry Concrete (RMC) construction, possibly precluding the attainment of full potential mechanical properties. While several measures to control bleeding of these mortars (by the addition of fine materials and/or the entrainment of air) have recently been implemented in practice, none would appear ideal. Hydrated lime has a large specific surface area and a longstanding history bearing testimony to its durability as a binder for RMC mortars. However, its propensity to arrest bleeding does not appear to have been quantified. This was investigated by means of a bleeding test which confirmed that small additions (of as little as 1% by mass of cement) of lime may significantly reduce bleeding. Furthermore, added lime appeared to enhance short and long-term compressive strengths, probably by reacting with pozzolans, and it is argued that several other mortar properties may also be enhanced by the addition of a small quantity of lime.

1 INTRODUCTION

A noteworthy observation, common to all compressive tests conducted on cubic RMC specimens to date[1-3], is that fracture appears to originate at the interface between mortar and rock as shown in Figure 1. These fracture surfaces typically first appear sub-parallel to the principal compressive stress at levels significantly lower than maximum. Post-mortem examination of many of these fracture surfaces has revealed planes of weakness caused by interruption of the interfacial bond between mortar and rock by the presence of bleed-water at the time of casting, particularly below large flat horizontal rock surfaces as shown in Figure 2. Because the initiation of failure under compression is mainly tensile, this results in strength anisotropy*. The problem is exacerbated by the fact that RMC is typically cast in horizontal layers and that stones have a natural tendency to lie horizontally where they are most stable; it is these large horizontal surfaces which trap lense-shaped flaws of bleed-water as it tries to migrate upwards. These horizontal planes of weakness have a pronounced effect on the cubic specimen compressive strength since cubes are loaded 90° relative to their casting direction; which simulates the thrust loading of arch bridge and arch dam structures.

Reducing the extent of bleeding seems a logical means of enhancing this interfacial bond strength. Moreover, increasing the interfacial bond strength, might correspondingly increase the level of stress at which interfacial cracking begins, possibly resulting in a consequent improvement in unconfined compressive strength. Such an improvement is likely to be more significant in RMC than in conventional concrete since the compressive strength of RMC appears to depend more critically on interfacial bond[4]. This is because the mortar matrix is likely to experience greater inherent dessication shrinkage stress and thermally induced tensile stress on account of the lack of coarse aggregate to dilute and restrain itself within the large interstitial volumes between the boulder inclusions. Furthermore, the contiguous boulder interaction limits the contraction on a macroscopic level and the lower density of interfacial zone volume (to absorb these strains) counters relaxation of the interstitial matrix.

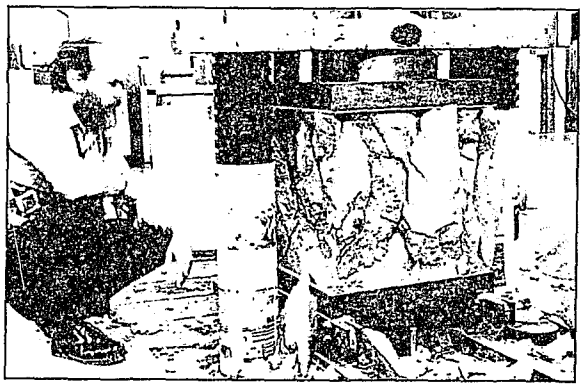

Fig 1 Typical fracture of a 500 mm RMC cube specimen initiated by an interfacial bond failure.

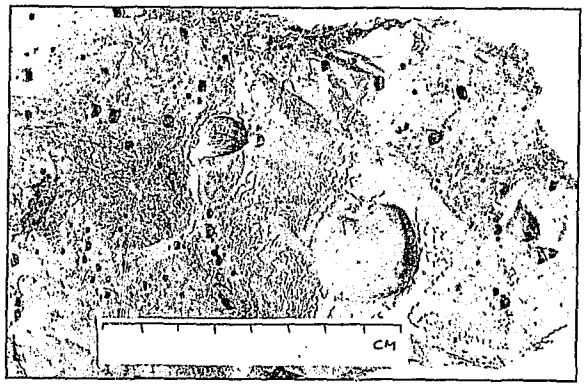

Fig 2 A large fragment of mortar which became prematurely detached from its adjacent rock surface as a result of bond interruption by bleed-water and air trapped against the horizontal undersurface of rock at the time of casting.

* *The material is weakest when a uniaxial compressive load acts parallel to these planes of weakness.*

BACKGROUND TO THE CURRENT USE OF RUBBLE MASONRY CONCRETE IN SOUTHERN AFRICA IN CONTEXT FOR THE BENEFIT OF FOREIGN READERS

Rubble Masonry Concrete (RMC) is a particulate composite construction material consisting of large irregular boulders which are manually placed into a mortar matrix. This method of construction is thousands of years old and was used in parts of many well known ancient structures including Hadrian's Wall, the Tower of Pisa and St Andrew's Castle. During the 19th and early 20th centuries RMC was used for the construction of many of the world's largest dams, particularly in America. Subsequently, it gave way to the faster placement of cyclopean masonry and masonry concrete (concrete containing plums) and eventually concrete as we know it today. Only a few developing countries, including India and Zimbabwe have continued to use RMC in an effort to conserve foreign reserves and provide economical infrastructure. More recently, labour-intensive RMC construction has been adopted in South Africa following incentives to relieve the problem of high unemployment. Since cement is the only material which need be purchased, a large proportion of the investment in RMC infrastructure is made available to be reinvested into destitute communities.



Typical labour-intensive casting of rubble masonry concrete (RMC). Here, a massive thrust block for the Maritsane Dam in South Africa is being built. The typical high proportion of mortar and the randomness of the rocks' shape, size and placement is evident in this photograph.

In addition to reducing the strength of the composite, bleed-water channels also increase permeability, which is undesirable in water retaining structures.

The extent of bleeding and the problems asso... with bleeding appear to be more severe in RMC than in conventional concretes or mortars, possibly as a result of combinations of the following factors:-

1) The relatively large volumes of mortar which occupy the interstices in RMC have a greater propensity to bleed than equivalent volumes of concrete which are diluted with coarse aggregate. (Coarse aggregate reduces the water content of a mix and it cannot bleed.)

2) The clean river sands, typically used for mortars in southern Africa, are often deficient in fines and consequently lack water retention capacity.

3) Water is added to the mortar mix until the desired consistency is achieved; a subjective judgement which is typically made at the visual discretion of unskilled labourers. Less effort is required to mix (especially when hand mixing) and use mortar with a high slump than is required for a low slump. Therefore, labourers have a tendency to add excessive quantities of water. Even a small excess of water does considerable harm. Powers[8] has shown that an increase in the water content of a mix of as little as 20% may increase the rate of bleeding by 2,5 times.

4) The contiguous interaction of large inclusions precludes them settling with the matrix (as may be possible in conventional concrete) as the bleed-water is displaced by gravity. Thus, discontinuities between the undersurfaces of boulders and the mortar matrix caused by plastic settlement of the latter are more likely.

5) The entrapment of migrating bleed-water, beneath the undersurfaces of rocks, becomes more pronounced as their size increases.

The most effective existing methods of reducing bleeding in conventional concretes appear to be by the addition of fine material to the mix and/or by deliberately entraining small air bubbles in the mortar[7]. Fine material has a large specific surface area (defined as the ratio of its total surface area to its total absolute volume) and it is this area which is able to retain water. Thus, the larger the specific surface area, the greater the water retention capacity. Therefore, the effectiveness of a fine material in reducing bleeding is proportional to its specific surface area. Entrained air bubbles are also effective in curbing bleeding by reducing the water requirement of a mortar[7] (a reduction made possible by their lubricating effect), dilating the capillary passages[13] (thereby blocking capillary flow of bleed-water) and by adding to the specific surface area of the mix in the same way as fine particles[7].

Past efforts to combat bleeding in RMC have utilised these measures. According to Chemaly[9], the Zimbabweans blend two parts river sand with one part of pit sand to combat the fines deficiency of their river sands. In South Africa, de Beer[5] has specified a 30% fly ash cement and a fine filler sand for blending with the river sand for the construction of Maritsane Dam, while Shaw[10], pioneered the use (for RMC in South Africa) of a

proprietary masonry cement, containing an air entraining agent and finely ground limestone, for constructing Bakubung Dam.

Although these measures all have some merit in reducing the extent of bleeding in large volumes of mortars, none appears ideal.

Pit sands generally contain a high proportion of clay and silt, the levels of which may often be excessive and variable[11]. Fine filler sands on the other hand often lack sub 150 µm particles. This was noted by Fine[5] who went to great trouble to obtain a well graded alluvial material, for the manufacture of an ideal mortar, by blending sands with different nominal size fractions from two Johannesburg rivers. Nevertheless, this blended material was still deficient in fines with only 3,3% of material passing the 150 µm sieve and a fineness modulus of 3,7. Despite using a 30% fly ash cement and a 1:4 nominal volume mix, this mixture bled so profusely, during the mixing, that it proved impossible to obtain a slump of greater than about 30 mm. Water could be seen draining out of the mortar while it was being hand mixed by shovel on the ground.

There is evidence that the use of fly ash cement and fine filler sand blending was not totally effective in limiting the ravages of bleeding in the mortar used to construct Maritsane Dam. Compressive tests on manufactured 500 mm RMC cube specimens revealed extensive planes of weakness below horizontal rock surfaces caused by entrapped bleed-water[12,3].

Although South African fly ash typically has a larger specific surface (400-600 m²/kg) than common cements (280-330 m²/kg) and a slightly smaller nominal spherical diameter (3,26-2,17 µm) than common cements (3,41-2,90 µm), its addition to the mix retards the stiffening or setting time, thereby causing the mortar to bleed for a longer period of time[12]. This has been found to partially offset the benefit of the finer particles in arresting bleeding[12].

Despite a decrease in the water:cement ratio afforded by the lubricating effect of the bubbles, their presence reduces compressive strength (by approximately 4-6% for every 1% of entrained air)[13]. In all but lean mixes, this has been reported to result in a net reduction in compressive strength of as much as 15%[13]. Furthermore, air bubbles produced by air-entraining agents are reported to reduce the interfacial bond strength because the bubbles interrupt the intimate contact between phases at the interface and block the entry of paste into minute fissures. From the little data available, it would appear that the magnitude of this reduction depends upon the volume fraction of bubbles, the nature of the substrate and/or the type of stress applied to the interface. Wuerpel[14] reports a 10% reduction in shear-bond strength, to smooth steel reinforcing bars, between concrete containing an "optimum amount" of entrained air and equivalent control concrete containing no entrained air. At the other extreme, Matthys[15-17] reports that applied to brick and concrete block, non-air-entrained Portland cement-lime mortars exhibit significantly higher flexural bond-strengths (tested according to ASTM C-1072[18] and ASTM E-72[19]) than air-entrained masonry cement mortars; typically 70% to 80% higher, in the case of concrete block.

Unfortunately, there appears to have been a dearth of research on the effects of entrained air upon the bond to rock, possibly as a consequence of the difficulties in measuring the bond strength between mortar and rock. However, Roxburgh[3] has compared the compressive strengths of 500 mm cubic

Table 1: Properties of the river sand used for bleeding tests.

Properties		Grading analysis	
		Sieve size (µm)	Cumulative % Passing
Relative density	2,582		
Loose bulk density	1572 kg/m³	4750	100
Consolidated bulk density	1712 kg/m³	2360	99
Fineness modulus	2,06	1180	89
		600	62
		300	32
		150	13
		75	5,1

Table 2: Volumes of bleed-water decanted from test specimens at hourly intervals (mℓ). The percentage lime additions are relative to the weight of cement. The percentages in parenthesis indicate the proportional reduction in bleeding at that time.

Time (Minutes)	Control (no lime)	1% Lime	5% Lime	25% lime
60	178 (0%)	102 (43%)	62 (65%)	19 (89%)
120	132 (0%)	115 (30%)	87 (52%)	30 (84%)
180 (Vibrated)	180 (0%)	144 (26%)	108 (48%)	36 (83%)
Total Vol (mℓ)	490	361	257	85 (83%)

specimens manufactured with the same proprietary masonry cement (containing an air-entraining agent) as was specified for the Bakubung Dam by Shaw[12] with specimens containing straight-cement sand mortars. Although this masonry cement appeared to have been effective in arresting bleeding, there was no evidence of an improvement in compressive strength. This might possibly be due to the corresponding reduction in compressive strength of the mortar matrix and/or the reduction in interfacial bond as a consequence of the bubbles.
Thus, *it seems reasonable to speculate that, if bleeding could be* substantially reduced, without the formation of 'deleterious' air bubbles, properties of RMC could be improved to greater engineering advantage.

ROD RANKINE matriculated at St Stithian's in 1984 and completed his B.Sc. (Bldg,) at Wits in 1988. He *subsequently conducted an investigation into the use of* 3CR12 alloy as a new corrosion resisting reinforcing steel for Middelburg Steel and Alloys (now Columbus) for which he was awarded his M.Sc. (Eng). In 1990, he served in the SA Navy and in 1991 he worked for R.J. Southey on the Actinia oilrig during the final stages of the Mossgas project. From 1992 to 1998, he lectured on the academic staff at Wits where he became a senior lecturer and developed a research interest in formulating design and construction guidelines for labour-intensively built rubble masonry concrete structures. Recently, he joined the Cement and Concrete Institute's School of Concrete Technology.



ROBERT T MCCUTCHEON obtained his B.Sc. (Eng) in 1963 and a Graduate Diploma in Engineering in 1964 from the University of the Witwatersrand. Between 1965 and 1971 he worked as an engineer and chairman of the South African Voluntary Service (SAVS). During the SAVS work he developed an interest in community development in relation to housing and the provision of low-cost infrastructure. From 1975 to 1987 he worked in Iran, Botswana and East Africa. Since then he has been a professor of Civil Engineering at Wits. In 1994, he was appointed *Head of Department of Civil Engineering. He is author of* a Green Paper on Job Creation in Transport and Public Works Western Cape.



1.1 Other Possible Measures to Reduce Bleeding

The addition of condensed silica fume (CSF) would seem an obvious choice following reports[20] that it 'practically arrests all bleeding' on account of its extremely high specific surface area. However, CSF is a very expensive product and one that is difficult to handle because of its very low bulk density. Furthermore, CSF is difficult to disperse evenly throughout the mix with conventional free-fall mixers or manual-mixing techniques[21]. (Ideally, a forced shearing mixing action, such as is provided by the blades of a pan *mixer is required to disperse CSF*[21].)

The possibility of reducing bleeding by lowering the water content of the mix through the use of a water reducing admixture was considered and rejected. Although the addition of water reducing admixtures, such as calcium-lignosulphonates, would increase fluidity and therefore permit the water content to be reduced, they have been reported to increase bleeding[7,22], possibly because their lubricating effect facilitates greater settlement of the heavy particles and/or because they may retard the setting and therefore prolong the period of bleeding. Furthermore, the use of chemical admixtures is impractical in labour-intensive projects because of the risk of incorrect dosing.

Another possible material, with an extremely high specific surface area, which appears to have been overlooked in recent times, is lime (hydroxide of calcium and/or magnesium). Minute lime particles form naturally (without grinding), during the vigorous chemical reaction when oxides of calcium/magnesium are hydrated. These particles are flat and extremely thin. Their diameters are about 500 times smaller than grains of common cement and the specific surface area of lime is about the same as CSF (about 20 000 m²/kg).

2 EXPERIMENTAL

An extensive review of lime literature revealed no quantitative data as to the extent to which limes might be effective in reducing bleeding. Consequently, a simple bleeding test based on ASTM Standard Test Methods C 232-92[23] and C 940-89[24] was undertaken. River sand from a tributary to the Braamfontein Spruit in Johannesburg with properties presented *in Table 1 was mixed with a cement blend (CEM II B-V 32.5) in* a 5:1 volume batch mix. A water/cement ratio of 1,41 was used, which yielded a collapse slump. This mix was then divided into four, 15 kg batches. The first batch served as the control and calcitic hydrated lime (SABS 523[25] type A2) was added to the other three batches by weight of cement (1%, 5% and 25% respectively) and mixed thoroughly into the mortar. Immediately thereafter, each batch was placed in a 10ℓ polyethylene bucket with a lipped seal lid to prevent evaporation of moisture. At hourly intervals, bleed-water was decanted from the surface and measured. The volumes decanted at the end of the third hour were encouraged to the surface by equal amounts of vibration. The results are recorded in Table 2.

3 CONCLUSION

The reduction in bleeding afforded from an addition of as little as even 1% lime may be significant.

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INVESTIGATING THE USE OF LIME AS AN ADMIXTURE TO IMPROVE THE PERFORMANCE OF RUBBLE MASONRY CONCRETE MORTARS; WITH PARTICULAR REFERENCE TO BLEEDING

Part Two

Roderick G.D. Rankine

Synopsis

Bleeding and the entrapment of bleed-water beneath large horizontal rock surfaces have been identified as inherent problems in Rubble Masonry Concrete (RMC) construction, possibly precluding the attainment of full potential mechanical properties. While several measures to control bleeding of these mortars (by the addition of fine materials and/or the entrainment of air) have recently been implemented in practice, none would appear ideal. Hydrated lime has a large specific surface area and a longstanding history bearing testimony to its durability as a binder for RMC mortars. However, its propensity to arrest bleeding does not appear to have been quantified. In part one of this paper, this was investigated by means of a bleeding test which confirmed that small additions (of as little as 1% by mass of cement) of lime may significantly reduce bleeding. Part two of this paper explores other consequences of using lime as an admixture to rubble masonry concrete mortars. Added lime appeared to enhance short and long-term compressive strengths, probably by reacting with pozzolans, and it is argued that several other mortar properties may also be enhanced by the addition of a small quantity of lime.

4 CONSIDERATION OF OTHER CONSEQUENCES OF LIME ADDITION TO RIVER SAND CEMENT MORTARS

4.1 Workability

According to Walker and Gutschick[26], masonry limes consist of laminar tabular-shaped hydroxide particles, about 0.08 μm in diameter {that is about 1/500th of the size of cement particles (see Figure 3)}. These plate-like particles are hygroscopic and on wetting they disengage and are easily dispersed as colloids. Each particle becomes separated from its adjacent neighbour by a tenacious film of water that enables them to slide relative to one another in a manner analogous to graphite lubricant. It is this phenomenon which they claim affords lime-containing mortars their desirable rheological properties (a smooth, plastic, buttery and easy to spread consistency that hangs on the trowel and clings to vertical surfaces), properties highly sought after by masons and often referred to as 'fat' or 'butteryness'. On this premise, American sources boast about lime's ability to enhance the plastic flow of concretes as well as reduce the water:cement ratio. Lazell[27] even cites a case where a nominal

concrete mix of (1:3:5), containing crusher sand was to be spouted into a dam through long chutes declined 18° from a batching plant on a hillside. He claims that initially, the wet concrete would not flow down the chutes but dammed up and spilt over the sides, however, with the addition of 10% lime, the material flowed smoothly without segregating and yielded a stronger concrete. Unfortunately, he does not say whether this addition is by mass or by volume, nor does he indicate whether it was added as a dry powder or as a mature putty. The latter could have significance since the plasticity of putties made with non-autoclaved dry hydrated lime powder are reported to improve (indefinitely[28]) with maturation after mixture with water[28-30], evidently at least overnight[28,30].

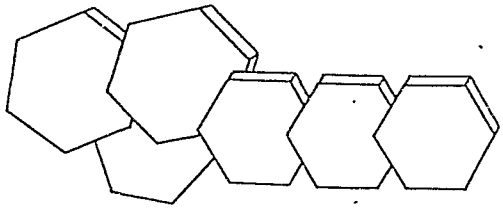

Fig 3 *Minute tabular magnesium hydroxide particles which become coated with a film of water and slide over one another in a manner analogous to graphite lubricant (Walker and Gutschick[26]).*

A review of literature related to workability shows the difficulty in defining this unquantifiable property. The best definition appears to be that given by ACI Committee 116 R-90[30]: "that property of freshly mixed concrete or mortar which determines the ease and homogeneity with which it can be mixed, placed, consolidated and finished". Perhaps the closest quantifiable relative to workability is consistence, which describes the mobility or ease of flow, as measured by the 'static' slump test or the more 'dynamic' flow test.

In an attempt to discover whether South African limes' have an ability to enhance the plastic flow of mortars, slump tests and flow tests according to SABS Method 862 Parts I and II[31] respectively, were conducted. Both hydrated calcitic lime (SABS 523 Type A2) and autoclaved dolomitic plastic lime (SABS 523 Type A2P) were added in incremental proportions to river sand-cement mortars with control slumps ranging from zero to collapse. In every case, the addition of lime appeared to stiffen the mix and reduce the slump. Furthermore, there was little detectable difference in slump between the Type A2 and Type A2P limes, even after allowing their pastes 24 hours of maturation (see Figure 4).

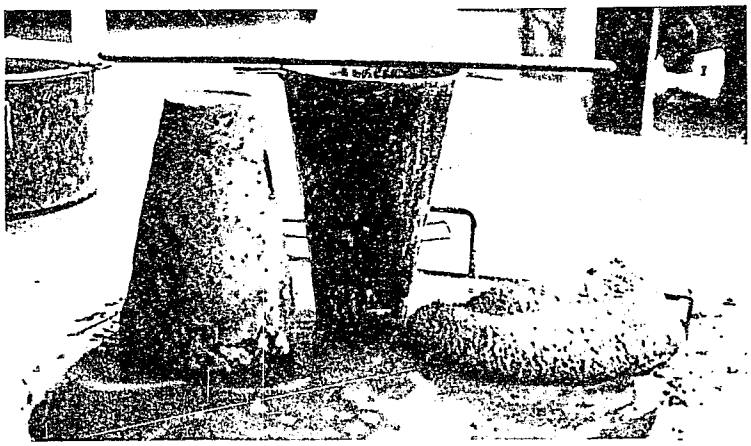

Fig 4 *The effect of dry hydrated lime on slump. The cement-mortar slump to the right of the picture contains no lime whereas that on the left contains an addition of 50% lime (compared to the mass of cement). The reduction in slump appears to be proportional to the amount of lime added, irrespective of the control slump or the type of lime.*

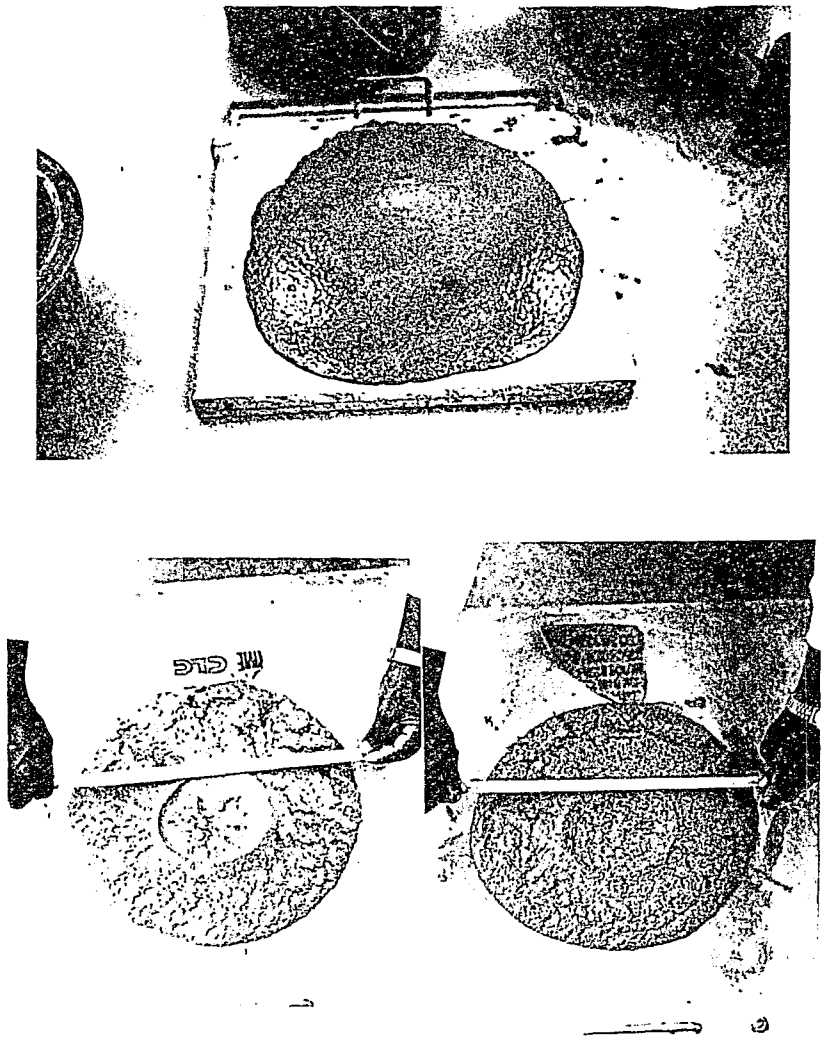

Fig 5 *The most extreme case showing the spread of mortars after completion of the flow test (SABS Method 862 Part II). All mortars had an identical content of cement (3 kg) and river sand content (18,84 kg) and a water:cement ratio of 1,5. The mortar in the top picture containing no lime spread to 660 mm. The mortar on the left containing Type A2 lime (3 kg) spread to 380 mm while the one on the right containing Type 2AP (3 kg) lime spread to 390 mm. Smaller proportions of lime also appeared to reduce the flow, although not as significantly.*

4.2 Hardening and Strength Gain

According to Sims[33], who was concerned about the aging of masonry dams, "*tests conducted on large samples of masonry* (from 1931-1985) *show that the strength achieved by mortar made of hydraulic lime and of cement* (presumably portland cement) *are not much different from each other, and not much less than cement concrete. The principal difference between these materials is the speed of setting. They reach a similar compre[illegible]e strength of about 30 MPa after about two years. The lime mortar sets considerably more slowly than cement, indeed the early specifications allow it to be used up to 24 hours after mixing. At 28 days, lime* (presumably hydraulic lime) *mortar usually had a strength of about 15 MPa.*"

Pure hydroxide-based mortars (ie. with zero portland cement content), stiffen as evaporation and suction by the masonry remove some of their free water. Compared with cementitious mortars, subsequent hardening occurs much more slowly and by different chemical processes. Calcium/magnesium hydroxides react with carbon dioxide to revert to the original stable carbonates found in nature. This re-carbonation, hereinafter referred to as carbonation, occurs from the surface moving inwards at a rate which depends upon the permeability and relative humidity of the mortar. At very low humidity, carbonation is precluded by the presence of water films through which CO_2 can diffuse and at very high humidity, CO_2 cannot easily penetrate the water filled pore spaces. A relative humidity of about 50% has been observed to coincide with the maximum rate of carbonation in portland cement concrete[34] and would probably also be optimal for lime mortars.

In addition to carbonation of lime, a pozzolanic reaction may also take place. The high pH environment of a lime mortar makes non-crystaline siliceous particles more soluble, allowing them to react and form a calcium silicate hydrate cementing material. Burnt clays in the form of pulverised roof tiles and pottery have been found in samples of Roman concrete and are likely to have contributed to its strength when pozzolanic sands were not available[35-37]. Vitruvius, Faventius and Palladius[37-38] all prescribed the addition of a third part of crushed earthenware to mortar when 'river sand' was used, which suggests that it was intended to act as a substitute for pozzolana when active, non-crystaline sand was not available.

So-called 'hydraulic limes' which could set under water, albeit slowly, were manufactured before the discovery of portland cement, by calcining specially selected argillaceous limestones (limestones streaked with bands of clay). These clays contain the essential ingredients; soluble silica, alumina and iron oxide, required to react with $Ca(OH)_2$ to form calcium silicate hydrates; a reaction independent of air. John Smeaton investigated many sources of limestone to produce an hydraulic mortar that might survive the onslaught of the sea for the foundations of the Edystone Lighthouse[39]. He concluded that, contrary to popular belief at that time, "*the most pure limestone was not the best for making mortar, especially for building in water, but that limestone intimately mixed with a proportion of clay, which is by burning converted into brick, is made to act more strongly as a cement.*" The Edystone Lighthouse and several hundred dams[33], many of which still survive today, bear testimony to the fact that hydraulic lime mortars do set and survive underwater. Rankine[40] (1866), was probably the first engineer to document the comparative strengths of various mortars with and without pozzolans. Although his mix proportions, testing details and procedures remain a

mystery, and should therefore not be compared with material strengths used today, it is nevertheless worth noting the significant long-term contribution to the compressive strength afforded to the mix by the addition of pozzolana.

Table 3 *Crushing strengths of mortars after 18 months reported by Rankine[40]. Sixteen years after these mortars were mixed, the common lime mortars were reported to gain a further strength increase (probably largely due to carbonation) of an eighth, and the hydraulic mortar an increase of a quarter. This would imply that, like carbonation, the pozzolanic reaction also continues for a long time.*

TYPE OF MORTAR	Strength (psi)	Strength (MPa)
Mortar of lime (presumably non-hydraulic) and river sand	440	3,0
Mortar of lime and pit sand (probably containing some clay)	800	5,5
Hydraulic mortar of lime and beaten tiles	930	6,6

Addis[41] has reasoned that because limes sold in South Africa are not hydraulic, they cannot be used to replace cement, but may be used in addition to cement. If we assume that the added lime's only contribution towards the strength of the mix is by carbonation, then this reasoning is sound. However, most fly ash cement blends often possess an abundance of pozzolana to react with the lime and result in long term strength development. The simple experiments, reported in Figures 6 and 7, confirm this assertion. The first experiment, reported in Figure 6, compared the effect of adding lime (equal to the weight of cement) to a 5:1 river sand:15% fly ash cement blend. The second experiment, reported in Figure 7, explored the strength developments of cubic mortar specimens containing equal volumes of fly ash and lime (no other binder) immersed in water, and kept dry.

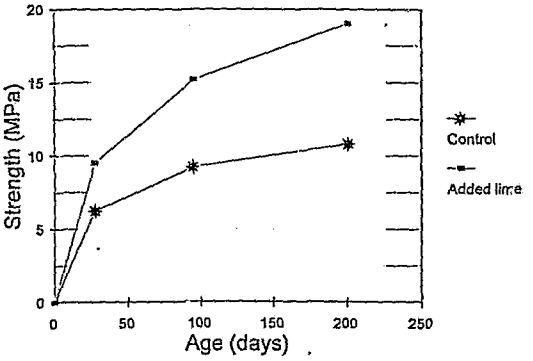

Fig 6 *Experiment to determine the effect of lime addition upon compressive strength of a 5:1 river sand:15% fly ash blend mortar. (1,17 water:cement ratio, 23 kg sand, 4,3 kg cement and 4,3 kg Type A2 lime)*

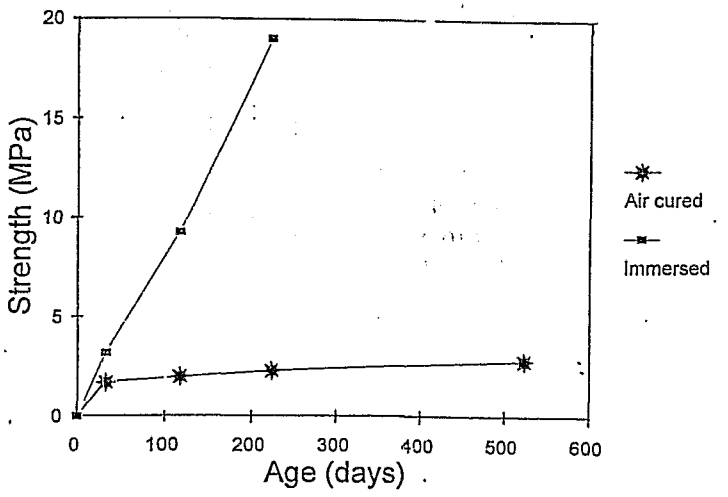

Fig 7 *The strength development of mortar bound with equal volumes of fly ash and lime. (Nominal volume mix 1:1:2 lime:fly ash: river sand)*

These tests are by no means exhaustive but they do confirm that the addition of lime to cement mortars does not appear to prejudice their compressive strength. The strength gain in the specimens containing lime, shown in Figure 6, is probably a result of a pozzolanic reaction between the lime and the fly ash and possibly also by the so-called 'fine filler effect' described elsewhere[42]. In contrast, where the same lime was used to replace cement (25% by mass of original cement) this was found to reduce the strength of the mortar by 45% at 28 days and 44% after 90 days. Further research is needed to establish whether there is an optimum ratio of fly ash to cement and lime to best utilise these ingredients.

4.3 Durability and Autogenous Healing

Before the advent of modern portland cement, in the first half of the nineteenth century, rubble masonry structures relied entirely upon lime and/or clay as the binder material for the mortar matrix. Hundreds of these ancient RMC structures have stood the test of time in an exemplary manner as a testament to the potential long-term durability of lime mortars. In fact, many are still in service today, having survived centuries and a few have lasted over 2000 years.

Studies[33,43-46] of aging and deterioration of ancient lime mortars seem to indicate a performance loss mainly as a consequence of an increase in porosity due to soft-water leaching and thermal changes such as freeze-thaw cycling.

Calcium hydroxide ($Ca(OH)_2$) is slightly soluble in water. The colder and softer the water, the greater its solubility. (0,08 g of $Ca(OH)_2$ will saturate 100 mℓ of water at 100 °C and at 0 °C this limit more than doubles to 0,17 g/100 mℓ.) Freshly placed mortar, containing calcitic lime, is very vulnerable to leaching of the $Ca(OH)_2$ by cold soft-water such as the snow-melt which drains in mountainous regions. If this water is flowing, as opposed to stagnant, it will be even more aggressive because it continually transports the dissolved $Ca(OH)_2$ away from the mortar and it never becomes saturated. By contrast, magnesium hydroxide {$Mg(OH)_2$ (brucite)} is far less soluble than calcium hydroxide. (0,0009 g of $Mg(OH)_2$ will saturate 100 mℓ of water at 0 °C.) Therefore, it may be prudent to specify autoclaved dolomitic limes for use in aggressive soft-water environments.

Moist or dissolved $Ca(OH)_2$ will react with CO_2, either in the air or dissolved in the water, to form calcium carbonate ($CaCO_3$) according to Equation 1.

$$Ca(OH)_2 + CO_2 = CaCO_3 + H_2O \qquad \text{Eqn 1}$$

During carbonation, of lime, the molecular mass and density increase as a molecule of CO_2 (molecular weight 44,01) bonds to each molecule of calcium hydroxide (molecular weight 74,07) to form calcium carbonate (molecular weight 100,07). Under the relatively low stresses experienced by hardening mortars, the calcium carbonate forms hex-rhombic crystals known as calcite (see Figure 8), which occupy about 4% more volume per molecule than the hexagonal calcium hydroxide crystals**. This slight volumetric increase is no doubt very effective in plugging relatively small pores and hairline cracks, thereby reducing the permeability of the masonry. Once carbonated, the exposed mortar surface becomes relatively insoluble in soft-water (0,0065 g of $CaCO_3$ will saturate 100 mℓ of water at 20 °C), thereby affording protection to its inner mass which may continue carbonating for hundreds of years. However, if the mortar is relatively porous or the cracks are too big, the process of dissolution will continue unabated as fresh water is constantly introduced and the solute is continually removed.

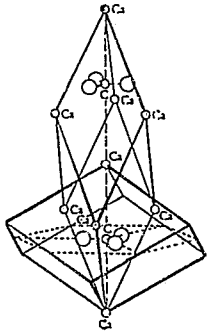

Fig 8 *Hex-rhombic structure of calcite.*

** The molecular volume increase, as $Ca(OH)_2$ is converted to $CaCO_3$, can be calculated by dividing each compound's molecular mass by its respective specific gravity. SG of $Ca(OH)_2$ = 2.078 and SG of $CaCO_3$ = 2.700. Similarly the molecular volume change, as $Mg(OH)_2$ is converted to $MgCO_3$, is at least 12%.

In the unlikely event of an excess of dissolved CO_2 in the water, over and above that consumed to cause carbonation, the process of decomposition may be substantially accelerated. This excess CO_2 (known as aggressive CO_2) is free to react with water to form carbonic acid (H_2CO_3), according to Equation 2, which further reacts with $CaCO_3$ to form calcium bicarbonate ($Ca(HCO_3)_2$) according to Equation 3.

$$CO_2 + H_2O = H_2CO_3 \qquad \text{Eqn 2}$$

$$H_2CO_3 + CaCO_3 = Ca(HCO_3)_2 \qquad \text{Eqn 3}$$

Unlike calcium carbonate, calcium bicarbonate is very soluble in water and is thus more readily removed.

Thus, it might be argued that the addition of a dolomitic lime to a cement mortar may be beneficial in combatting soft-water attack by virtue of its inherent insolubility and by rendering the mortar less permeable, thereby blocking access to the aggressive water.

4.4 Ductility and the ability to accommodate movement

Small movements induce higher stresses in more rigid, brittle assemblages than they do in assemblages which possess a greater degree of accommodation by their ductility and resilience. Numerous authors have commented on the ductility and resilience which appear to be afforded to mortars by the addition of lime. Alexander[47] is reported to have measured increased deflections at maximum load in simply supported brick masonry beams loaded vertically when a portion of OPC was replaced by lime. Boynton[28] cites the case of builders of high industrial masonry chimneys who desire a relatively weak but 'rich' mortar of 1:2:5 (cement;lime;sand) which is reported to be very flexible and resilient. Such chimneys, that are 115 m high, are known to sway as much as 600 mm in winds of 130 km/h. Straight-cement mortars are found to be too rigid and brittle for this purpose. However, despite these claims, the literature appears lacking in quantitative data on the elastic/plastic deformation of lime mortars or even the shape of their load-deflection curves. In an effort to explore these phenomena, a compressometer with LVDT (linear voltage displacement transducers shown in Figure 9) was used to measure the effect of adding lime to a 25% fly-ash extended cement mortar (with mix proportions presented in Figure 6 and tested after 90 days of immersion). The stress-strain curves derived from the plotted load-deflection curves of prisms (200 mm X 100 mm X 100 mm) are presented in Figure 10. The greater toughness measured in the specimen containing lime, as evidenced by the slight increase in area under its load-deflection curve shown in Figure 10, would appear to be a mere consequence of the additional strength gain.

Ductility is a property usually afforded to materials by metallic bonds or amorphous structures and very seldom by covalent and ionic bonds commonly associated with cements, concretes and rocks. Part of the answer to the mystery of the origins of lime mortar ductility, may lie in their prolonged green period before substantial strength gain occurs. By means of X-Ray diffraction analysis of hydrated pastes, Kassman *et al.*[48] have discovered that calcium hydroxide is more amorphous than crystaline. In this amorphous state, mortars are likely to be more tolerant of small movements. It is

therefore a fortunate coincidence that the rate of settlement and movement of most structures tends to be greatest during their construction and that this rate gradually diminishes with time. The slower stiffening and setting of mortars containing lime probably enables them to accommodate these movements at the most critical time.

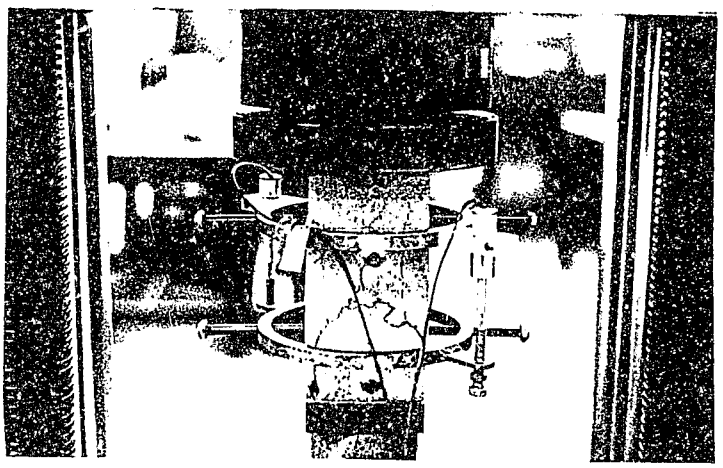

Fig 9 *Compressometer for plotting the load-deflection curve of mortar prisms.*

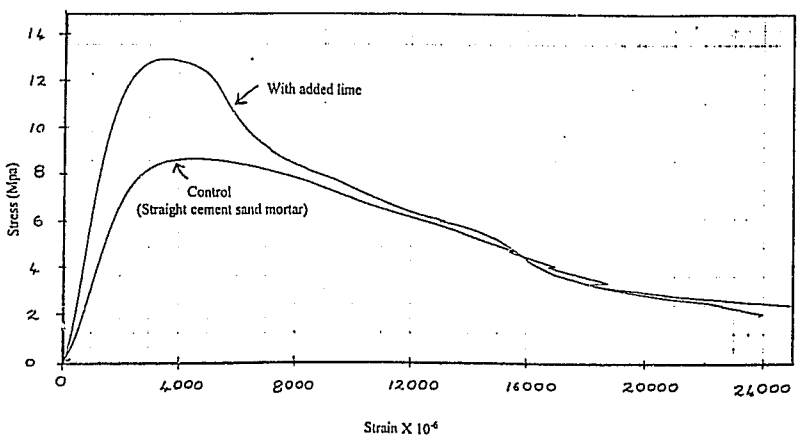

Fig 10 *Stress-strain curves derived from single load-deflection plots of mortar prisms with and without lime addition after 90 days immersion.*

4.5 Permeability and Efflorescence

Lime is probably the oldest waterproofing admixture for concrete. Many reservoirs and dams constructed in the USA during the first half of the twentieth century contained small additions of lime (1-15% in terms of weight of cement) to reduce permeability[27]. Advocates of this admixture claimed that it achieved its objective by acting as a filler material to fill voids, thereby densifying the concrete; rendering the paste less permeable as a result of its colloidal nature; improving the workability, thereby minimising segregation and honeycombing, and by permitting a lower water:cement ratio. It would appear that this reduction in permeability may not be realised where lime is substituted in place of portland cement. Perrie[19] has found that replacing a portion of OPC with an equivalent volume of lime (SABS 523 Type A2) did not result in a reduction in permeability.

Efflorescence is caused when water carrying $Ca(OH)_2$ in solution passes through a permeable portion of a RMC dam. On reaching the downstream face, the water evaporates depositing the $Ca(OH)_2$ which then carbonates to form unsightly calcite. Counter intuitively, despite efflorescence being caused by $Ca(OH)_2$, its addition to mortar for RMC dams may actually reduce efflorescence on the downstream face, if it is successful in reducing the permeability of the mortar, since this would reduce the volume of water flowing through the masonry and correspondingly the volume of transported solute.

4.6 Curing

The excellent water retention properties of lime may help to combat premature loss of moisture by masonry suction and bleeding. Furthermore, the reduction in permeability, as a consequence of adding lime (as opposed to replacing cement with lime), may reduce the rate of diffusion of gasses and water within the mortar, thereby combatting drying. Finally, the chemical process of carbonation, as shown in Equation 1, gives rise to one surplus molecule of water as each hydroxide molecule which is transformed into a carbonate molecule. This water must presumably benefit the hydration process of portland cements and pozzolans.

4.7 Economy and Practicality

Hydrated lime is probably the cheapest admixture to produce since it costs less to make than even cement (at least in terms of energy). However, it is increasingly difficult to purchase and where it is available, the price of 25 kg bags is beginning to include a scarcity premium. Nevertheless, 25 kg bags of lime can still be obtained at a significantly lower cost than 50 kg bags of cement blends (volumetrically equivalent to 25 kg bags of lime) from specialist lime distributors in Johannesburg. It is envisaged that, depending upon the specific properties of the sand available, additions of lime admixture, ranging from as little as 1% to as much as possibly 15%, might yield the desired effects. The practical problem will be to ensure that the unsophisticated workforce mixes the mortar in the correct proportions and that a system is implemented to preclude the possibility of 1 to 15% cement being added to a lime mortar. Ideally, it would be most convenient to purchase bags or silos of

specially blended and ready-to-use, masonry cement, containing the right proportions of cement, lime and fly ash, but without an air entraining agent.

4.8 Consequences of Adding Too Much Lime

An excess of lime results in a very sticky mortar with large air voids which become impossible to tamp out; in fact the very act of tamping seems to increase these voids. Such a mortar also develops an excessive drying shrinkage because lime putty shrinks even more than portland cement paste when it dries. Figure 11 shows a 100 mm test cube of lime putty (no aggregate whatsoever) which exhibited a linear shrinkage of 5% (that is a 15% volumetric shrinkage), in less than one month due to drying.

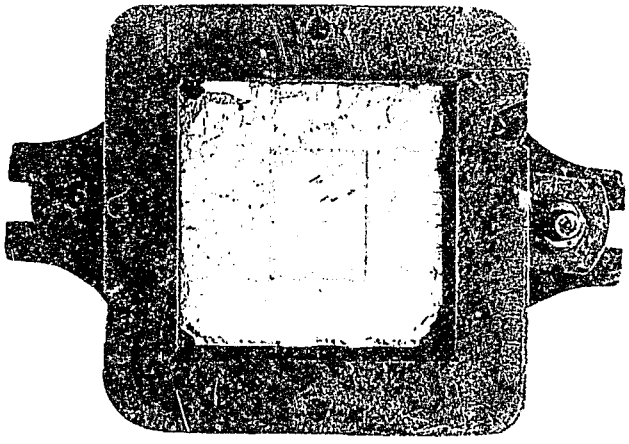

Fig 11 *Drying shrinkage of a 100 mm lime putty specimen in less than one month. The linear shrinkage is more than 5% and the volumetric shrinkage is more than 15%.*

5 CONCLUSION AND RECOMMENDATIONS

The addition of a small proportion of hydrated lime (even as little as 1% by mass of cement) to a fly-ash extended cement-river-sand RMC mortar has been shown to be extremely effective in reducing bleeding. Furthermore, the addition of lime does not appear to reduce the strength of mortar. Indeed, the results of this investigation would indicate that both the short and long-term compressive strengths of mortar cubes may be enhanced by the addition of lime, provided they are kept moist. Thus, it appears that the lime and fly-ash may engage in a long term pozzolanic reaction. Such a

reaction could logically be anticipated within the hearting mortar of thick RMC monolithic dam and arch bridge structures, which have a substantial cross-section, and are protected from drying by the outer masonry. In addition to these benefits, it has been argued that lime may also afford RMC mortars several other benefits including better water retention, resistance to early cracking, autogenous healing properties and reduced permeability. However, the addition of excessive quantities of lime may result in large entrapped air voids and high levels of dessication shrinkage. The exact type and quantity of lime admixture required for a RMC mortar will depend on the aggressiveness of the environment and upon the river-sand's grading and shape respectively. Structures which will be immersed in aggressive soft-water (particularly cold, flowing water) may benefit from the addition of less soluble autoclaved dolomitic or magnesian limes which contain magnesium hydroxide as opposed to only calcium hydroxide. River sands with a uniform particle size and/or sands which lack sufficient fines below the 300 µm particle size will require more lime than ideal sands. The exact amount of lime required to reduce bleeding to an acceptable amount can only be established by trial mixing. As a point of departure, 5% to 10% lime addition, by weight of cement, is proposed.

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APPENDIX 6

A MECHANICAL METHOD OF SENSITISING ELECTRONIC STRAIN MEASURING DEVICES IN RUBBLE MASONRY STRUCTURES

1st SOUTH AFRICAN CONFERENCE on APPLIED MECHANICS '96

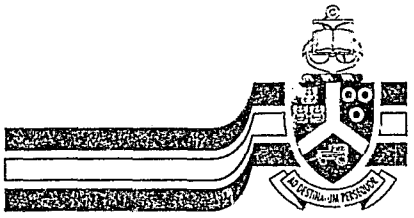
Incorporating:

4th National Symposium on CFD
14th Symposium on FEMSA
5th SA Aerospace Engineering Conference
Annual Seminar of the Strain Society of SA

JULY 1-5 1996, Eskom Midrand, Gauteng, South Africa

PROCEEDINGS OF 4 JULY 1996

Under the auspices of the South African Association of Theoretical and Applied Mechanics (SAAM)
Sponsored in part by the Foundation for Research Development and Eskom TRI.



University of Pretoria





Proceedings of the 1st South African Conference on Applied Mechanics (SACAM) '96, 1 - 5 July, 1996, Midrand, South Africa.

A MECHANICAL METHOD OF SENSITISING ELECTRONIC STRAIN MEASURING DEVICES IN RUBBLE MASONRY STRUCTURES

R.G.D. Rankine University of the Witwatersrand

ABSTRACT

The very high stiffness and low service stresses encountered in rubble masonry structures make live-load induced strains difficult to record with conventional electronic strain gauges. A simple mechanical strain amplification device is proposed to multiply tensile or compressive strains over a short length onto which electronic strain gauges are bonded. Calibration of the device, practical limitations governing the scope of amplification and some possible future research challenges are discussed.

BACKGROUND

The abundance of uneducated and unemployed people in Southern Africa has resulted in a recognition of the need to promote manual construction methods to enhance employment creation. The use of a building material (which is described more comprehensively elsewhere [1]), consisting of natural uncut stones placed manually in a matrix of cement mortar, has aroused great enthusiasm amongst those intent on increasing the employment content of their projects. Several dams and bridges [2-11] have been built using this technique, and it has proven to be a cost effective, socially acceptable and aesthetic alternative to more conventional structures. A lack of knowledge of the magnitude and nature of the strains experienced in these structures has been identified as an obstacle to their widespread adoption by at least one authority [11]. A theoretical and experimental

investigation at The University of the Witwatersrand [1,6] has been undertaken in an effort to develop more efficient structures of known reliability in future.

In a pilot study, strain measuring devices (electronic strain gauges attached to opposite faces of concrete prisms), were embedded into a bridge structure to measure live-load induced strains. However, the magnitude of the recorded strains was a fraction of what had been anticipated. Even electrical amplification of the signal to the threshold of possibility (without generating undue noise) proved unsuccessful in capturing this data.

Consequently, an improved mechanical strain multiplying embedment device was conceived, developed and refined. Strains as low as 10^{-9} can now be measured and four of these improved devices have been calibrated and installed in a bridge structure in Giyani in the Northern Province.

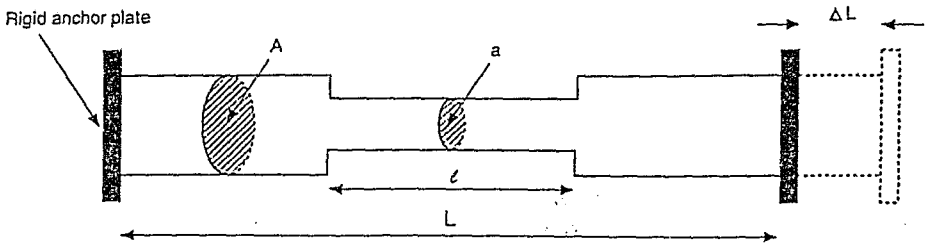
DESCRIPTION OF THE IMPROVED STRAIN CELL DEVICE

The device (shown diagrammatically in Fig 1) consists of a slender, thick walled hollow steel tube with a reduced cross sectional area over a fraction of its length, onto which, electronic strain gauges are bonded. Both ends of the tube are mechanically fastened to stiff steel plates to transfer any strain in the structure directly to the tube. The entire length of the tube is kept debonded from the surrounding mortar by means of a plastics conduit to prevent any transfer of load to the adjacent structure between its ends.

The tube section was deliberately chosen because of its optimal radius of gyration to resist buckling, thus enabling the device to measure compressive strains as well as tensile strains. The electronic strain gauges are bonded onto the surface of the necked region in a wheatstone bridge formation in such a way as to preclude the influences of temperature induced strains and bending stresses within the tube. Downward manipulation of the fraction of the cross sectional area within the necked region to that of the remainder of the tube results in a corresponding local increase in stress and hence strain within the neck. Less obviously, manipulation of the length of the reduced fraction (relative to the total length of the tube), also affects the local strain within the neck.

Table 1 quantifies the contribution of each governing ratio to the strain amplification factor, the derivation of which is presented below:-

NOTATION



ε_T = Avarage longitudinal strain between anchorages
ε_N = Longitudinal strain in the neck
ε_R = Longitudinal strain in the non-necked region
a = Cross sectional area of the necked region
A = Cross sectional area of non-necked region
L = Length between anchor plates
ℓ = Length of necked area
f = Amplification factor

$$\varepsilon_R = \varepsilon_N \times a/A \qquad \text{(EQN 1)}$$

$$\Delta L = \varepsilon_N \times \ell + \varepsilon_R (L - \ell) \qquad \text{(EQN 2)}$$

$$= \varepsilon_N \times \ell + \varepsilon_N \times a/A \ (L - \ell) \qquad \text{(Substituting EQN 1 into EQN 2)}$$

$$\therefore \ \varepsilon_N = \frac{\Delta L}{\ell + a/A \ (L - \ell)} \qquad \text{(EQN 3)}$$

$$\varepsilon_T = \frac{\Delta L}{L} \qquad \text{(EQN 4)}$$

$$f = \frac{\varepsilon_N}{\varepsilon_T} \qquad \text{(EQN 5)}$$

$$= \frac{\Delta L}{\ell + a/A \ (L - \ell)} \times \frac{L}{\Delta L} \qquad \text{(Substituting EQNs 3 \& 4 into EQN 5)}$$

$$f = \frac{L}{\ell + a/A \ (L - \ell)}$$

ℓ/L \ A/a	1	2	3	4	5	10	20	50	100
1	1	1	1	1	1	1	1	1	1
0.5	1	1.33	1.50	1.60	1.67	1.82	1.90	1.96	1.98
0.2	1	1.67	2.14	2.50	2.78	3.57	4.17	4.63	4.81
0.1	1	1.82	2.50	3.08	3.57	5.26	6.90	8.47	9.17
0.05	1	1.90	2.73	3.48	4.17	6.90	10.26	14.49	16.81
0.02	1	1.96	2.88	3.77	4.63	8.47	14.49	25.25	33.56
0.01	1	1.98	2.94	3.88	4.81	9.17	16.81	33.56	50.25

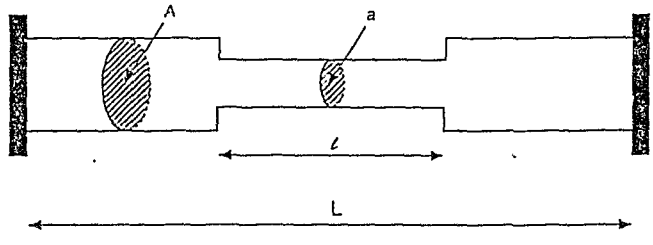


Table 1 Contribution of relative dimensions to the amplification factor.

It is interesting to note how the law of diminishing returns appears to limit the achievable amplification. There are very real practical limitations governing the length of the neck and its minimum thickness. The size of the four strain gauges themselves impose a lower limit on the area of the neck and the weight and strength of the tube material limit the minimum thickness to which the neck can be reduced. Placing the necked region close to one of the ends of the tube goes a long way towards reducing the bending stresses (within the neck) likely to be incurred in service.

CALIBRATION

Calibration is achieved by straining the entire device by a predetermined strain measured as the average displacement displayed in two horizontally opposed clock dial gauges (see Fig 2). The necessary force to achieve this is developed by rotation of an extremely fine pitched machine screw attached to a lever arm so as to increase its purchase. The strain cell is suspended in tension from a bicycle chain to preclude the interferences of any bending moments and torsion.

FUTURE RESEARCH CHALLENGES

1) Innovations to further enhance the mechanical strain amplification might arise through the combination of two materials with dichotomous elastic properties in the same strain cell. The material with low elastic modulus would be used in the necked region to promote strain and the stiffer material used throughout the remainder of the device to minimise strain. If aluminium (which has an elastic modulus of about 67 GPa) were combined with steel (which has an elastic modulus of about 200 GPa), the amplification factor could be increased a further threefold. The challenge to overcome is to develop a reliable mechanical connection between t two materials.

2) There is a great need to develop instrumentation for measuring small strains in structures such as dams which take vast periods of time to load and unload. Electronic strain gauge output tends to drift over long periods of time as a result of influences such as temperature and voltage fluctuations.

3) New improved strain cell amplifiers are currently being employed at The University of The Witwatersrand to determine simultaneously the elastic moduli (in compression) and poison's ratios of rubble masonry cubes containing various combinations of geological aggregates and mortars.

CONCLUSION

The extremely small strains encountered in rubble masonry structures are almost impossible to detect with conventional electronic strain gauges since incremental electronic gain amplification is rapidly accompanied by unacceptable noise generation. A simple integrated mechanical strain amplifier, which is shaped to amplify local strain only over the length onto which strain gauges are bonded, is proposed as a solution to this problem. Practical constraints severely govern the extent of achievable amplification of the prototype but it is proposed that further developments might arise through the combination of materials exhibiting dichotomous elastic moduli.

ACKNOWLEDGMENTS

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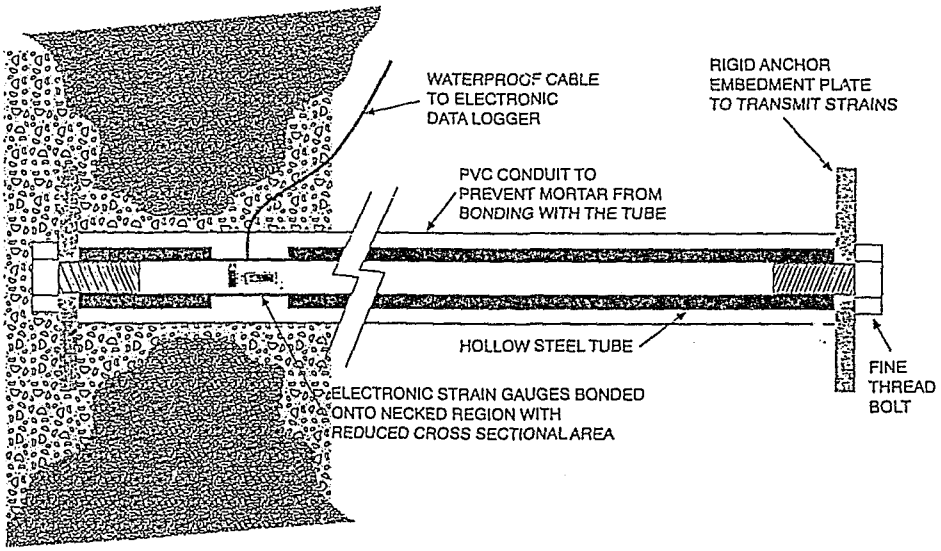

Fig 1 Diagrammatic representation of an integrated mechanical strain cell amplifier for recording small strains in rubble masonry structures.

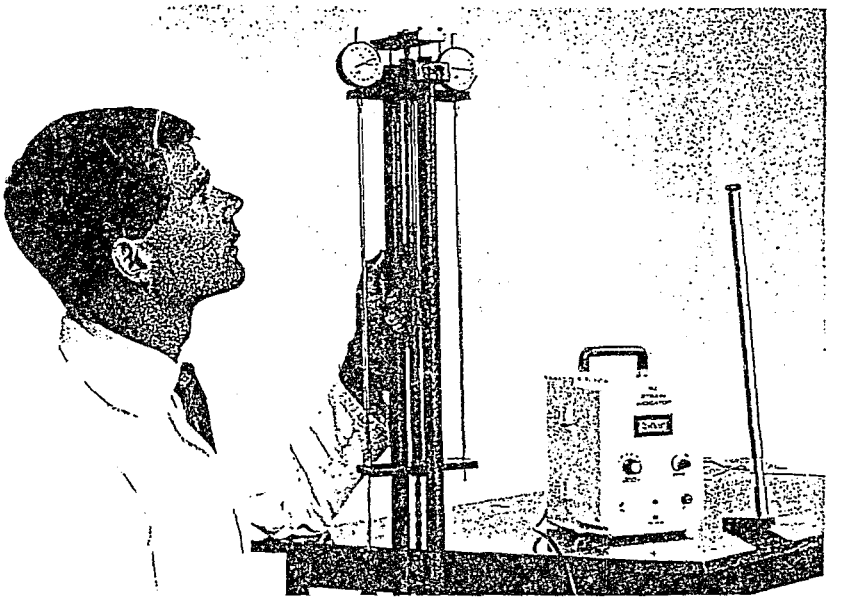

Fig 2 Apparatus for mechanical calibration of an integrated strain cell amplifier. A partially assembled strain cell can be seen on the extreme right of the photograph.

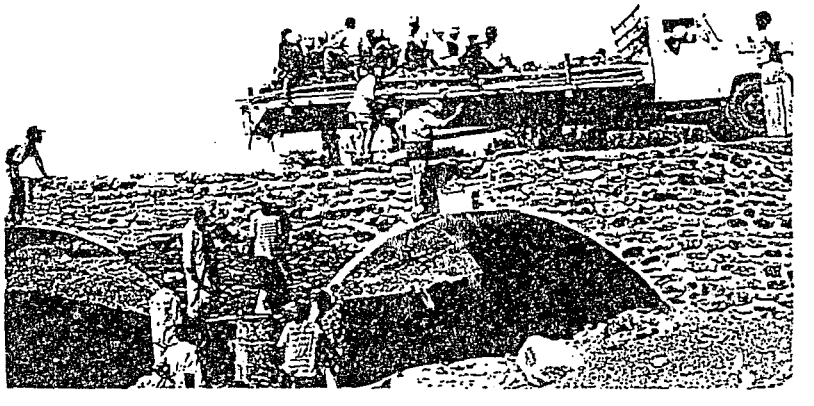

Fig 3 Measuring 100 kN live-load induced stresses in a bridge structure in the Bolobedu community near Tzaneen in the Northern Province.

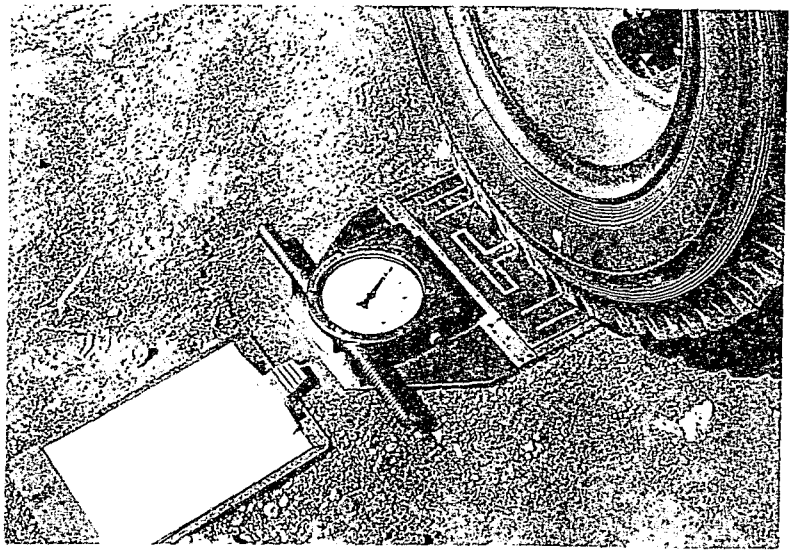

Fig 4 An expensive commercially available hydraulic vehicle weighing device.

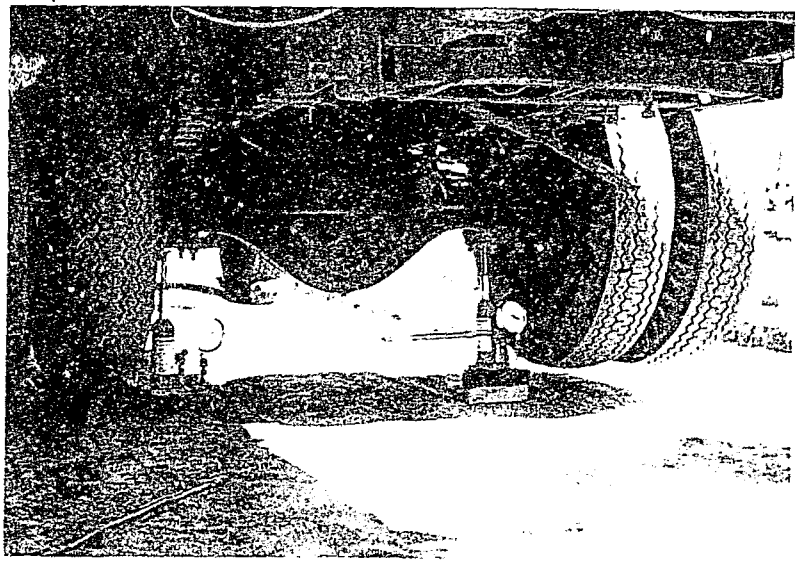

Fig 5 Custom-built, affordable axle weighing device made by attaching hydraulic presure gauges to read the oil pressure within 5 tonne bottle jacks. Because the jacking points are not directly beneath the wheels, the instruments must always be used in pairs directly under the axle to measure axle load. These instruments (together) cost less than 1/30th of the cost of the imported device shown in Figure 4.

DISCUSSION

E O'Brian (Experimental Stress Analysis Group. British Aerospace Airbus)

"I can't help but wonder why you didn't use a more appropriate technique of strain measurement such as Moiré interferometry or glass scales. I know of a case where the entrance to a railway tunnel in the UK was covered in patterned wallpaper and the strain in its surface measured by an observer from within a moving train. In your case, you need just have plastered one of the external elevational faces of a rubble masonry bridge with wallpaper and observed the interference."

R.G.D. Rankine

"I am no expert on the subject of strain measurement by interferometry but at an early stage, after some debate with colleagues, I dismissed this method for two reasons. Firstly, the rubble finish is far from smooth and plain; hardly an appropriate surface to receive 'wallpaper'. Secondly, I have my doubts about the continuity between the facade masonry and the hearting masonry since, according to the current South African practice, these two types of masonry are infrequently placed simultaneously and seldom tied with tie stones (see Fig A1). If you or anyone else has personal experience in overcoming these problems or if you know of some text which could provide me this practical guidance, I would be grateful and willing to try it. We did seriously consider the use of glass scales, some of which have adequate sensitivity, but unfortunately the cost of one scale together with its necessary accessories exceeds my entire research budget."



Fig A1 Facade masonry being laid against the hearting masonry after the structure became serviceable. Currently, in South Africa, the facade masonry is not placed at the same time as the hearting masonry and tie stones to bond the two together are seldom used.

APPENDIX 7

PROPOSED GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF RUBBLE MASONRY ARCH BRIDGES

VOLUME 40 NUMBER 3 THIRD QUARTER 1998

Journal

The South African Institution of Civil Engineering

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Die Suid-Afrikaanse Instituut van Siviele Ingenieurswese

R G D Rankine, M Gohnert and
R T McCutcheon

Proposed guidelines for the design and construction of rubble masonry concrete arch bridges

Synopsis

Practical design and construction considerations, other than detailed material specifications, are discussed with reference to precedent, experimental testing and theoretical modelling. The choice of site and positioning of the rubble masonry concrete (RMC) structure in relation to the flow of the river is critical. Ideally, these structures should be founded on, and anchored to, bedrock. Aperture size should be maximized and the structure should be designed to present the minimum obstruction to the flow of water and debris. Three methods of structural analysis are discussed, including a finite element model that is used to explore a typical arch structure's sensitivity to four different aperture shapes of equal area as well as a circular arch's response to submersion in water. A parabolic aperture was found to be optimal in limiting tensile stresses in the masonry and submersion was demonstrated to increase these tensile stresses by as much as 20 per cent. Cracks were modelled at points of maximum tension and shown to play a useful role in relieving tensile stress without compromising stability. An inference is drawn that these structures are sufficiently tolerant of the magnitude of geometrical errors expected from the use of low-level skills and limited supervision. Consideration of several failure mechanisms indicates that the RMC arch problem is primarily one of equilibrium rather than of material strength and that structural stability without reinforcing can be ensured.

Samevatting

Praktiese ontwerp- en konstruksieoorwegings, buiten gedetailleerde materiaalspesifikasies, word bespreek met verwysing na voorafgaande ekspevimentele toetsing en teoretiese modellering. Die keuse van terrein en posisionering van die messelwerkpuinbeton- (MPB) struktuur relatief tot die vloei van die rivier is krities. Die ideaal is dat hierdie strukture op rotsbodem gefundeer en geanker moet wees. Openingsgroottes moet 'n maksimum wees and die struktuur moet ontwerp word om die minimum steurnis op die vloei van water en rommel te hê. Drie metodes van strukturele ontleding word bespreek. Dit sluit in 'n eindige-elementmodel wat gebruik word om die sensitiwiteit van 'n tipiese boogstruktuur teenoor vier openingsvorme van gelyke oppervlak te bepaal, sowel as 'n sirkelvormige boog se reaksie op oorstroming in water. 'n Paraboliese opening blyk optimaal te wees in die beperking van trekspannings met tot soveel as 20 persent. Krake is gemodelleer by punte van maksimum trek en speel 'n nuttige rol in die verligting van trekspanning sonder om die stabiliteit in gevaar te stel. 'n Gevolgtrekking is gemaak dat hierdie strukture w draagsaam genoeg is om die orde van geometriese foute wat verwag kan word weens die gebruik van laervlak vaardighede en beperkte toesig te hanteer. Oorweging van verskeie swigmeganismes toon dat die MPB-boogprobleem primêr een van ewewig is eerder as materiaalsterkte en dat strukturele stabiliteit sonder wapening gewaarborg kan word.

SAICE Journal/SAISI-joernaal 1998, 40(3): 9-16

Introduction

Arch construction first appeared in the ancient civilization between the Tigris and Euphrates, some 2 000 years before the Roman conquest. The oldest stone arch bridge survives at Smyrna, Turkey, and was built in 900 BC (Peet, 1966). Most remaining masonry arch monuments show evidence of a serious effort to cut and shape the stones so that the joint planes were thin and perpendicular to the principal stresses (at least at the edges, if not the core), probably to reduce dependence upon the strength of the ancient lime mortars by relying on the bilateral constraint offered by flat stone faces perpendicular to the dilating mortar. However, there are exceptions, such as the stone bridge shown in Fig 1. Thus, with superior modern Portland cement mortars (as opposed to traditional lime mortars) and correct design, it seems reasonable to speculate that rubble masonry concrete* (RMC) bridges have the potential to serve mankind as well as their dependable forefathers.

The RMC arch bridge of recent times was pioneered by the Zimbabweans during the Rhodesian War of Independence (1973 to 1980). Their methods of design and construction evolved empirically and by serendipity, often at the expense of an entire structure being washed away by floods or being circumnavigated by an obstinate river. Their experience can therefore provide a significant reference to spare other developing countries from making the same costly mistakes. Unfortunately, their valuable

* Rubble masonry concrete is a composite building material comprising large irregular stones manually embedded in a mortar matrix for employment-intensive infrastructure. It requires little skill to assemble and has recently been successfully used to construct many significant dams and bridges in southern Africa.







Rod Rankine *completed his BSc (Bldg) at the University of the Witwatersrand in 1988. He subsequently carried out an investigation on a new type of corrosion-resisting reinforcing steel for Middelburg Steel and Alloys, for which he was awarded his MSc (Eng). In 1991 he worked for R J Southey on the Actinia oilrig during the Mossgas project. In 1992 he joined the academic staff at Wits where he developed an interest in technical aspects of labour-intensive construction, particularly appropriate design and construction guidelines for rubble masonry structures. In January 1998 he was appointed a lecturer at the School of Concrete Technology at the Cement and Concrete Institute.*

Prof Robert McCutcheon *obtained his BSc (Eng) in 1963 and a Graduate Diploma in Engineering in 1964 from the University of the Witwatersrand. Between 1965 and 1971 he worked as an engineer and chairman of the South African Voluntary Service (SAVS). During the SAVS work he developed an interest in community development in relation to housing and the provision of low-cost infrastructure. From 1975 to 1987 he worked in Iran, Botswana and East Africa. Since then he has been a Professor of Civil Engineering at Wits. In 1994 he was appointed head of the Civil Engineering Department. He is author of a Green Paper on Job Creation in Transport and Public Works (Western Cape).*

Dr M Gohnert *graduated with a BSc in civil engineering (1985) and Master of Engineering Management (1987) from Brigham Young University, Utah. He commenced employment with LTA Civil Designs and later Civemech, a subsidiary of Murray and Roberts. In 1990 he joined the Department of Civil Engineering at the University of the Witwatersrand. In 1995 he completed his PhD at the same university, where he is currently a senior lecturer.*

knowledge and experience has not been meticulously collated and disseminated. Consequently, many RMC bridge structures continue to be built in other parts of southern Africa without the benefit of this knowledge, and indeed without any specific guidelines or standard code of practice. Furthermore, a quagmire of confusion on the subject of arch analysis appears to torment many designers of RMC bridges. Indeed, Rankine et al (1995) report on a current practice of modelling RMC arch bridges as equivalent frames, necessitating the consumption of vast quantities of reinforcing steel in both horizontal directions to 'resist the bending moments generated between the points of support'.

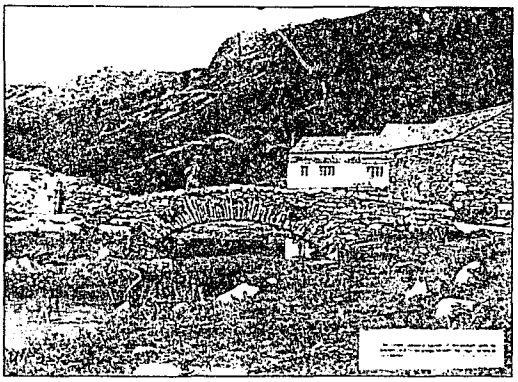

Fig 1: Ancient pack-horse stone bridge in Watendlath Cumberland, England

Choice of site location and foundations

The first stage of arch bridge design involves the selection of a suitable site. Upon this single decision rests the ultimate success of the entire project. If a good site is chosen, the bridge may be built in harmony with nature and be economic; if not, it is liable to be washed away by the first flood. Ideally, such a site should offer the following:

1. Bedrock beneath all arch abutments.
2. A straight section of the river or stream with well defined banks that will enable the bridge to cross perpendicular to its flow.
3. Moderately inclined banks that are neither too steep nor too shallow, so as to obviate the need for vulnerable and costly approach works.

Shelton (1985) has cited the undermining of foundations by river flow (which accelerates around obstructions) as being the most common single cause of low-level bridge failure in Zimbabwe. All too frequently RMC bridges are built as part of rural road upgrading projects on old drift sites, which were deliberately situated on wide sandy stretches of the river where the flow is shallow. These sites are often far above good founding material. In such cases, the construction of adequate foundations becomes very difficult and is likely to compromise the financial viability of most RMC bridge designs. Founding on unstable silty material is dangerous, owing to the tendency of riverbed material to become liquid, a phenomenon that has been reported to occur at depths as great as between 2 m (Mainwaring and Hasluck, 1985) and 10 m (Mainwaring, 1995) below riverbed. While some designers (Rankine et al, 1995) might argue that it is often not possible, or indeed economic, to found RMC bridges on bedrock, Shelton (1985) reminisces that never once was he unable to find a site that offered exposed rock right across the river as a foundation (Shelton, 1985).

Where a bridge is built as a skew crossing or on a curve of a river, the designer runs the risk of his structure being damaged or made completely redundant. The skew crossing tends to channel the stream towards one of its banks, particularly when it becomes blocked by debris during floods. The chance of that abutment being damaged is greatly increased if it is not founded on bedrock. Centrifugal force of a river at a bend tends to channel the flow towards the outside bank, particularly during floods, and the presence of the structure merely serves to accelerate this flow and enhance erosion. In such cases, rivers have been known to cut complete new channels around bridges, and remain permanently realigned, necessitating the construction of new structures (Mainwaring and Hasluck, 1985; Shelton, 1985).

In cases where river-bank gradients require significant modification, the viability of the structure is likely to be compromised. Earth fill approach embankments are a tempting shortcut, but are notorious for being washed away by floods (Mainwaring and Hasluck, 1985; Shelton, 1995). Any built-up approach produces appreciable obstruction to the river's flow, as it acts like a weir. Therefore, built-up approaches should be treated as a continuum of the bridge structure. They should be provided with apertures to reduce their obstruction and be as structurally competent as the bridge itself. Excavated approaches 'in cut' are preferable, but are liable to become choked with debris during floods, necessitating clearing afterwards.

Founding on rock

Where bedrock occurs at abutment sites, the economy of an arch bridge is likely to be excellent, since the problem of providing a suitable foundation to resist not only its weight but also its thrust has already been solved by nature. Where this rock is reasonably plane and perpendicular to the line of thrust, arches can be sprung directly from the rock surface. Where this is not the case, the casting of concrete bases to levels upon which formwork can be erected is recommended. In both cases anchorage via socketing and/or dowelling is recommended.

Founding on material other than rock

Where it is not possible to found on bedrock (owing to its non-existence at an accessible depth) and the prevention of liquefaction of the riverbed below the foundations can be ensured, there may be justification for the use of piles, bases or a reinforced concrete raft foundation. Consideration should be given to the maintenance of the horizontal reaction to arch thrust at abutments in case the structure, or indeed the riverbed, should move. Fig 2 proposes one practical way in which this may be achieved where a bridge is founded on bases. Where piled foundations are used, some raked piles inclined sub-parallel to the line of thrust may achieve the same result and in cases of bridges built on raft foundations, reinforcement of the slab may be used to tie both abutments together. In the absence of bedrock, resistance to transient loading may also be achieved by the use of raked piles and/or by increasing the width of the structure. Bases should be designed to minimize the danger of undermining by scouring during floods and also to provide the required bearing area, so as not to exceed the bearing strength of the river bed. To guard against the scouring action of the foundations, scour protection from one abutment to the other, terminating at cut-off walls on the leading and trailing edges, is required. The cut-off walls, particularly on the downstream side, must be built to a depth below which liquefaction of the riverbed material is deemed unlikely, and no less than about 2 m. In many rural areas, such as the Northern Province, gabions are deliberately not used as scour protection because the wire is frequently stolen. RMC, rip rap or grouted stone pitching has been preferred instead.

Founding partly on rock

Although bridges with alternate abutments founded on rock and other material respectively do exist, the practice is not recommended. A 3,7 m radius arch bridge, in the Sekhukhune area in the Southern District of the Northern Province, founded partly on rock and partly on clay, was reported to have been severely damaged during a flood in 1995 (Grobler et al, 1995). The abutment founded on clay is understood to have been undermined.

Forces to be resisted

Arch bridges and their foundations need to withstand transient water loads during floods, self-weight, traffic loads, internally generated arch thrust and possibly load redistributions caused by foundation distortions.

Transient loading

Bridges with a small aperture fraction produce an appreciable obstruction to a river's flow, frequently aggravated by driftwood blocking the flow through the apertures, effectively forming a weir. Overtopping must therefore be considered as an important loading condition, since it will cause significant lateral load and possibly overturning of the structure, as demonstrated by the collapse of an arch bridge after very heavy local rain reported by Konishi (1987). Flood loading is defined as a transient load and must be considered in combination with buoyed-up self-weight, but not with traffic loads. Data concerning actual values of flood-water pressures may be sought from other sources (Konishi, 1987) or from an assessment of water depth provided by the Department of Transport guidelines. The required moment, to restore stability against overturning, may be calculated by taking moments about the base of the downstream face,

assuming the buoyed-up weight of masonry to be no more than 1 400 kg/m³. Tall single-lane bridges are likely to require anchorage to resist this eventuality.

Traffic loading

An assessment of appropriate standards for rural roads (CSIR Report No 92/466/1) has considered the traffic that is likely to use the rural roads for which most RMC bridges are designed. The recommendation is made that NA and NB24 vehicle loads, as defined by TMH 7 (1981), should be used. The maximum vehicle loads that the bridges are required to withstand are thus 16 wheel loads of 60 kN each, consisting of four rows in two pairs. Each row of wheels is separated by a minimum of 2 m, and each row has four wheels spaced 1 m apart across the bridge. The wheel contact area may be defined by a square with a 240 mm side. If more ambitious structural designs are attempted, for example bridges to carry traffic on national roads, then NB36 loading would appear to be appropriate (where the maximum wheel load is 90 kN and the wheel contact area may be defined by a square with a 300 mm side). Where a structure is to be covered with fill and/or a sub-base, the compaction equipment used may subject the structure to a more severe point load, particularly if a steel roller acts directly on the rough rubble finish.

Methods of structural analysis

Although there appear to be no reported incidents of live-load-induced RMC bridge failures, their reliable analysis is a prerequisite for optimal design solutions for given locations and the construction of more ambitious structures in future. Numerous authorities including Tellett (1986), Hendry (1990), Page (1993) and de Bruin (1996) have reviewed a variety of methods of structural analysis of arch bridges, both simple and complex, with which arch bridge designers ought to be acquainted. Traditional design philosophy assumes that arch masonry possesses good compressive strength but zero tensile strength, and aims to prevent the formation of tension within the masonry by limiting the resultant line of thrust to lie within the middle third of the arch ring.

This is easily achieved in arches that carry either a pure uniformly distributed load or a pure point load. In the former case, for example, where the arch is required only to support its own weight, it would always meet these conditions, provided it approximated the shape of an inverted catenary. The inverted catenary arch perfectly traces its line of self-weight-induced thrust, just as a chain sags reciprocally in perfect tension. In the latter case, where the arch has only to support a point load, it would qualify for these conditions if it had a triangular shape whose apex coincided with the load. However, upon the application of point loads to the catenary arch, tensions quickly develop. Where the point load is static, such tensions can easily be alleviated by distorting the catenary arch shape towards that of a triangle. However, when the point load may be dynamic, as is always the problem in bridges, the solution is rather more complex since the arch ring must be capable of accommodating each unique shape of line of thrust as the load moves across its deck. This is achieved in practice by thickening the arch ring and/or by increasing the dead-weight of the structure itself and/or by tolerating a small amount of tension within the arch. It is generally assumed that the material is infinitely strong in compression and that, provided the reaction to thrust is maintained, a failure condition is reached when the line of thrust reaches the outer faces of the masonry at no fewer than four points, converting the structure into a kinematic mechanism by the formation of hinge points where the line of thrust alternately coincides with the intrados and extrados of the arch.

Three possible kinematic mechanisms are illustrated in Fig 3 for elementary point loads, from which it can be seen that hinge points are always formed on alternate sides of the arch. The diagrams show the lines of thrust originating from the point loads as straight lines. This is not only conservative, but unrealistic, since the line can be straight only if the structure itself is weightless. The true line of thrust is defined by the path followed by the resultant of all the forces acting on the arch across its span, including self-weight and external actions. Hence, the true lines of thrust will be bent favourably away from the intrados. However, the heavier the point load in relation to the weight of the structure, the straighter the line of thrust will become. Determination of the position of this true line of thrust may be obtained by calculation (Boothby, 1995) or by a graphical method (Curtin, 1982) of analysis. Fig 4 shows an advanced stage of collapse in two model arches with four and five hinges.

Calculation of the position of the line of thrust

The arch is divided into equal segments with point loads emanating

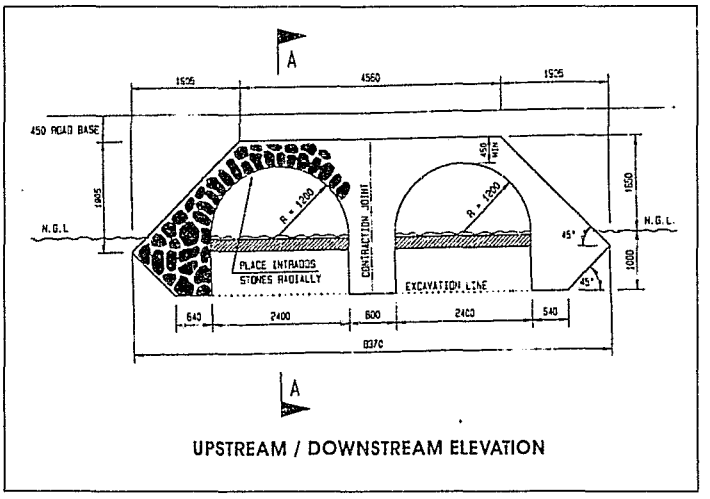

Fig 2: A proposed arch bridge foundation design for soils. The inclined foundation abutments are intended to maintain the reactions to horizontal thrust in the event of subsidence. Wing walls and scour protection have been omitted for clarity.

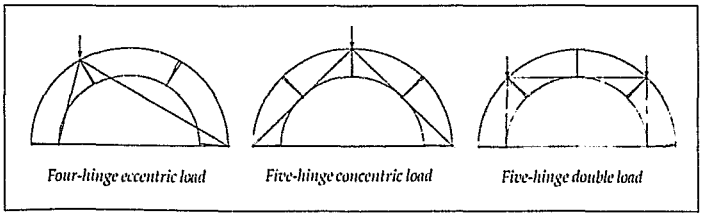

Fig 3: Possible arch ring kinematic hinge failure mechanisms that may occur provided reaction to thrust is maintained. The straight lines of thrust imply that the structures are weightless.

from their centres representing the self-weight of the structure, including the road and fill. Reactions are determined by taking moments about assumed hinges, and the position of the line of thrust relative to the intrados (measured vertically) is determined accordingly. A value that falls outside of the arch ring indicates an incorrect assumption of hinge position.

Graphical determination of the position of the line of thrust

Ideally, this method of analysis is best performed with a computer-aided drawing program to ensure graphical accuracy. The practice of performing the analysis provides the arch designer with an 'intuitive feel' with which to anticipate the consequences of future modifications to various arch design parameters, particularly regarding the contribution of self-weight in maintaining equilibrium. The worked example shown in Fig 5 considers a 90 kN wheel load at mid-span to be a point load shared equally between the two halves. Because of its symmetry, only one half is considered:

1. The arch is divided into an arbitrary number of equal segments (seven in this case). The weight of each segment (comprising the self-weight of the structure, including the road, fill and live-load) is represented as a point load emanating from the centre of each segment.

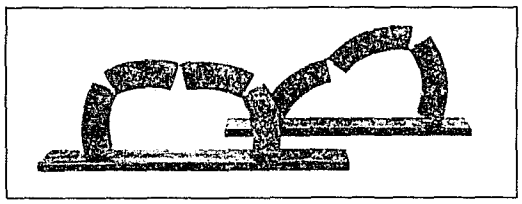

Fig 4: Model arches demonstrating advanced five-hinged concentric (left foreground) and four-hinged eccentric (right background) load collapse mechanisms

2. Horizontal thrust at the upper third of the crown is calculated by taking moments about one of the reactions (assumed at A in this case).

$H_c \times r = 50 \times 2{,}1 + 6 \times 1{,}8 + 7 \times 1{,}5 + 8 \times 1{,}2 + 12 \times 0{,}9 + 12 \times 0{,}6 + 11 \times 0{,}3$
$\therefore H_c \times 1{,}5 = 157{,}2$ kNm (substituting r)
$\therefore H_c = 104{,}8$ kN

3. The force diagram is plotted, using Bow's notation (on the left-hand side).
4. Starting horizontally at the crown reaction, the line of thrust is plotted on the arch profile parallel to the equilibrium vectors on the force diagram: 0-0 to the middle of the first segment, changing slope to 0-50 to the middle of the second segment and so on. The fact that the line of thrust plotted does not exactly intersect point A signifies that this assumed reaction point was not precise.
5. The process is repeated for different load conditions and/or arch shapes and is used to determine violation of design criteria such as the formation of kinematic mechanisms or the middle third rule.

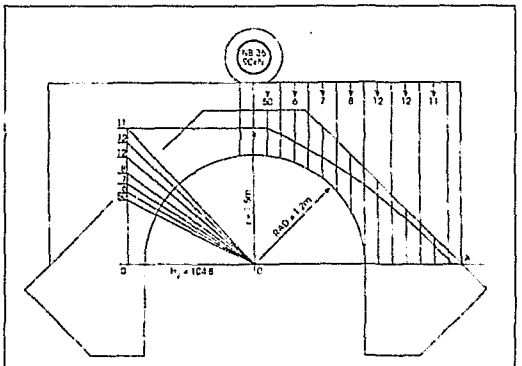

Fig 5: Graphical method of determination of the position of the arch's line of thrust. Each of the seven segments is 300 mm wide.

Finite element analysis and the evaluation of a simple model

Until recently, finite element computer models for the analysis of arch bridges were too sophisticated for most designers. They typically required very expensive computer hardware and software and the finite mesh had to be generated manually, a process that took much longer than graphical arch design methods. A newly released two-dimensional plane strain program, Prokon, was made available to this initiative for the analysis of RMC arches. Given a grid spacing within the limitations of its capacity, the model is able to quickly generate its own mesh and therefore easily accommodate design changes. Moreover, it is able to model structures comprising more than one type of material and can therefore accommodate additional overburden layers such as *fill, sub-grade and base-course and, in so* doing, distribute concentrated wheel loads over the masonry. However, additional overburden layers do not normally form part of an RMC bridge structure and there may be better ways of modelling the resistance of fill.

The model's limitations include a restriction to assumed linear homogeneous material response and its inability to model independent behaviour in compression and tension, thus precluding it automatically allowing cracks to develop. Furthermore, it makes the assumption that the foundations are infinitely stiff, an assumption perhaps realistic when founding upon rock, *but questionable in soils. Required input includes an esti*mation of material properties, namely density, stiffness and Poisson's ratio, as well as imposed point loads and UDLs. It assumes the materials to be homogeneous and allows the user to manipulate the mesh size to achieve an acceptable compromise between detail and processing time. The output data include quantified maximum compressive and tensile stresses, together with their co-ordinates and stress vectors, as well as deflections and their co-ordinates.

Geometrical considerations and failure mechanisms

Every effort should be made to minimize the bridge's obstruction to the flowing river, particularly when it is in flood. Large apertures are less likely to become blocked and require maintenance. The arch designer should therefore always aim to make apertures as big as possible. Shelton (1985) recommends a minimum span of at least 5 m over any river worthy of its name.

Zimbabweans (Mainwaring and Hasluck, 1985; Wooton and Stephens, 1987) have found that inclining the upstream elevation of the bridge, with *a wedge of additional masonry,* assists in lifting debris over the top of the structure and clear of its openings. Dos Santos (1993) proposes a catenary bridge deck (lowest at the centre of the river) to encourage overtopping of the structure at midstream to prevent scour from eroding the approaches and abutments. Kerbs and guide-blocks should be kept discontinuous to decrease obstruction to debris and flood water (dos Santos, 1993).

Finite element analysis applied to a typical structure and the exploration of aperture shapes

Table 1 shows some effects of four different aperture shapes, each with an equal cross-sectional area of 2,3 m². A two-dimensional plane strain

Table 1: Maximum masonry tensile and (compressive) stresses and deflections at mid-span (mm) in an RMC bridge with a constant aperture of 2,3 m² and NB36 wheel-loading at various distances (defined in terms of fractions of arch spans) from mid-span

Position of load	Circular 2.4 m span	Parabolic 2.7 m span $y = -0.692 N^2$	Gothic 2.6 m span	Triangular 3.0 m span
Load zero offset	0.267 MPa 30.5 µm	0.240 MPa 28.7 µm	0.340 MPa 30.6 µm	0.083 MPa 28.6 µm
Load 1/12 offset	0.221 MPa 29.0 µm			
Load 2/12 offset	0.201 MPa 26.1 µm	0.187 MPa 23.8 µm	0.199 MPa 25.0 µm	0.445 MPa
Load 3/12 offset	0.228 MPa 22.4 µm			0.379 MPa
Load 4/12 offset	0.275 MPa 18.6 µm	0.035 MPa 15.9 µm	0.260 MPa 16.5 µm	0.406 MPa
Load 5/12 offset	0.293 MPa 14.9 µm		0.270 MPa 13.0 µm	0.415 MPa
Load 6/12 offset	0.285 MPa 11.8 µm			
Max compressive stress	(0.573) MPa at 4/12 off	(0.803) MPa at 4/12 off	(0.547) MPa at 5/12 off	(1.070) MPa at 4/12 off

finite element model, Prokon, was used to compare [illegible] stresses and deflections at midspan. The structure was designed to carry a temporary deviation road as part of a national road rehabilitation programme in the Northern Province and was therefore designed to withstand NB36 loading. The assumed material properties, including a 150 mm sub-base immediately below the road level and a granular fill between the subbase and the road level, are presented in Table 2.

A sensitivity analysis revealed a 100 mm grid interval, for generating the finite element mesh, to yield slightly higher stresses than both larger and smaller spacings. As well as being conservative, this grid size was found to produce acceptable detail without excessively compromising the processing speed. A 1 m wide elevational section was assumed to carry the entire 300 mm wide NB36 wheel load. This assumes a load spread of only 350 mm, on either side of the wheel, over a depth of about 800 mm. A single axle load was found to represent the most critical load case in the short span structure under consideration. However, readers are cautioned that more complex axle combinations would, in all probability, represent critical load cases on more ambitious structures with greater spans.

Calibration and interpretation of the model's output

A glance at the maximum stresses confirms their very low order of magnitude. A comparison of these values with recorded live-load-induced stresses (measured with electronic resistance strain gauges) in a similar structure by Rankine et al (1995) confirms that the model is predicting stresses of a realistic order. Unfortunately, the measuring apparatus used was found to be insufficiently sensitive to record the minuscule service strains with much accuracy. Consequently, an improved mechanical strain amplifying device has been developed and refined to permit accurate calibrations (Rankine, 1996). However, it seems extremely unlikely that a structure (with the typical proportions of the one investigated) could fail as a result of masonry crushing. They are far more likely to fail in tension, in which case the triangular shape would fail first, as its load rolls one-sixth of its span off-centre, followed by the gothic arch with its load mid-span, followed by the circular arch with its load five-twelfths off-centre, followed by the parabola loaded mid-span. The parabolic shape, as adopted by the Zimbabweans for proprietary precast shells (Shelton, 1995; dos Santos, 1993) as permanent formwork, would appear to be optimal in limiting arch tension. However, the circular arch should probably be retained for ordinary construction, since parabolic formwork would be very difficult to construct with the humble resources available to most developing rural communities. An inference that may be deduced from the insensitivity of these structures to significant changes in aperture shape is their tolerance of considerable geometric inaccuracies, such as the exact arch shape and road level as well as formwork deformation. They would most certainly accommodate the magnitude of setting-out errors (of about 100 mm) that dos Santos (1993) anticipates from an unsophisticated workforce with limited supervision.

Finite element modelling of tension cracks and collapse mechanisms

Table 3 presents data generated by the model when triangular tension cracks are introduced by substituting triangular areas, at points of maximum tension, with a material of greatly reduced stiffness. In both cases the tensile stresses were inversely proportional to crack length, confirming that the cracks facilitate the relieving of tensile stresses. The corresponding increases in compressive stress remained lower than the maximums recorded in Table 1. Thus, even after load redistribution, consequent upon cracking, failure by masonry crushing remains extremely unlikely. Fig 6 contrasts the tensile stresses before and after cracking. The very small 'post-crack' tensile stress at the extrados to the right of the crown indicates the approximate origin of another tensile crack that needs to occur to create the fourth hinge prerequisite to the formation of a kinematic mechanism before the arch will collapse. Fig 7 shows this mechanism in action. However, this is extremely unlikely to occur, since this tensile stress is probably too low to cause a crack, and even if it succeeded the abutments and fill would be able to resist all but a cataclysmic kick of the hinge.

The coarseness of the aggregate interlock in RMC makes voussoir and/or haunch sliding failure mechanisms (Baker, 1987) extremely unlikely, even in severely cracked structures.

Buoyancy compensation in the estimation of self-weight

Self-weight plays a crucial role in the maintenance of arch equilibrium, since the heavier the arch structure itself, the greater its tolerance of point loads. As the arch becomes submerged, upon a rising water level, a proportion of its weight, equal to its displaced volume, will be relieved. Thus, counter-intuitive as it may appear, in supporting the structure and making it lighter, buoyancy also makes it weaker. The effect is significant considering that, upon immersion, the weight of RMC would be reduced by about 42 per cent from, say, 2 400 kg/m³ to 1 400 kg/m³. The phenomenon is even more pronounced where lower density base coarse layers are applied as fill above a structure. Table 4 presents the results of a finite element analysis that quantitatively explores the changes that occur as the same arch bridge is relieved of its displacement. The analysis was made simply by reducing the material 'density' of the immersed materials by 1 000 kg/m³.

The structure loaded mid-span experiences a 20 per cent increase in tensile stress as it becomes immersed. This is a result of the fabric being called upon to restore equilibrium upon the removal of dead weight. The reason this increase in tension does not appear to be mirrored by a compressive stress increase is because, unlike the tensile element, the compressive stresses are a composite of both the fabric and gravitational reactions. The fabric compression reaction opposes the tensile forces within the structure to maintain equilibrium. As the fabric reaction increases in proportion to load, as it must to maintain equilibrium, so the gravitational reaction diminishes at a faster rate. In this case, the rate of gravitational decrease appears to occur at about twice the rate of the fabric increase, reflecting a net decrease. The observed decrease in mid-span deflection (raising of the deck) supports this hypothesis.

Table 2: Material properties adopted for theoretical analysis

Material	*Elastic modulus (GPa)*	*Poisson's ratio*	*Density (kg/m³)*
RMC	28	0,2	2 400
Granular fill	0,12	0,35	1 800
Subbase	0,2	0,35	1 800

Table 3: Stress and deflection changes as a consequence of cracking at points of maximum tension under wheel load

	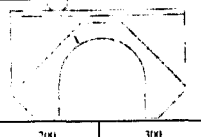		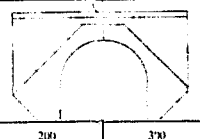	
Crack length (mm)	200	300	200	300
Max tensile stress (MPa)	0 182	0 150	0 221	0 075
Decrease in max tensile stress (%)	37%	48%	17%	72%
Max comp stress (MPa)	0 503	0 538	0 488	0 491
Mid-span intrados deflection (µm)	10 7	10 1	33 3	33 8
Increase in mid-span intrados deflection (%)	-28%	-32%	10%	12%

Table 4: The effect of buoyancy on an RMC arch structure

	 NB36 SINGLE WHEEL MIDSPAN		 NB36 SINGLE WHEEL ¼ SPAN OFF CENTRE	
	Dry	Buoyancy compensated	Dry	Buoyancy compensated
Tensile stress % Increase	0 261 MPa	0 312 MPa 20 %	0 281 MPa	0 316 MPa 12 %
Compressive Stress % Decrease	0 422 MPa	0 397 MPa 6%	0 548 MPa	0 485 MPa 11 %
Mid-span deflection % Decrease	30 1 µm	26 8 µm 11%	14 9 µm	11 5 µm 23 %

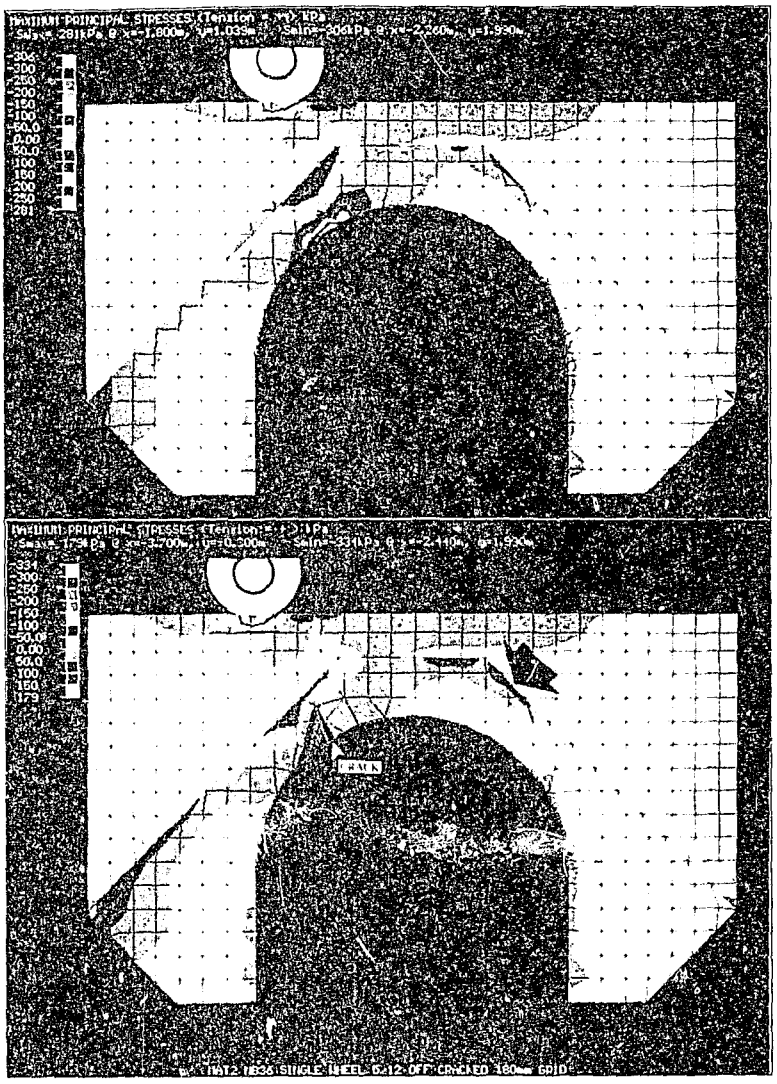

Fig 6: Tensile stresses in an arch loaded at five-twelfths of its span off-centre before (above) and after (below) cracking. The arrow pointing to the extrados to the right of the crown indicates a region of increasing tensile stress that needs to crack to form the fourth hinge, prerequisite to the formation of a collapse mechanism.

Anchorage and resistance to transient loading

Anchorage against sliding and overturning from transient flood-loading may best be achieved by socketing into bedrock and or grouting pre-cranked anchor bars into pre-drilled holes. Although standard jackhammer drill steels may be capable of forming a hole sufficiently large to accommodate 25 mm nominal diameter reinforcing bars, it may be preferable to use a lesser diameter bar to ensure a good grout surround. Bar spacers obviously help to achieve this cover. The practice of cleaning the hole with high pressure water and air and then filling the hole with liquid grout and placing the anchor into the grout enhances bond, cover and resistance to corrosion. If the grout settles, the hole may be topped up with more grout. The projecting bars should be sufficiently long and amply embedded into the structure to preclude masonry tensile failure immediately above the proximal end. Furthermore, the projecting bar should not be embedded directly in RMC; a cavity (of about 200 mm in diameter) should be left around the bar and subsequently filled with a cement-rich high slump concrete to increase the bar aggregate interlock, thereby affording greater pullout resistance. Anchorage may be further improved by linking the anchor bars with transverse reinforcing steel secured within the 'crooks' of their cranked radii.

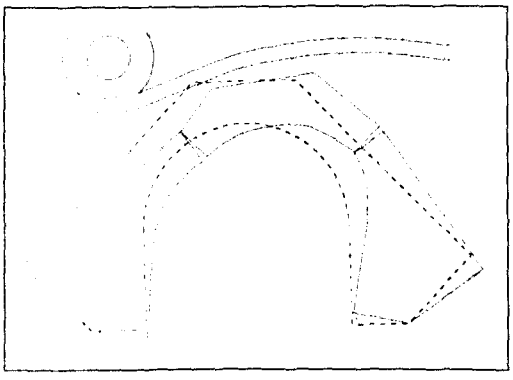

Fig 7: Unlikely four-hinged kinematic collapse mechanism

Centring or formwork

Traditionally, timber has been used as centring for arches for many hundreds of years. Recently, the high cost of timber has challenged builders to consider other alternatives. One method deployed in the Northern Province uses corrugated iron roof-sheeting pre-cranked to the intrados radius (Rankine et al, 1995; Grobler et al, 1995). In the case of large radii, these sheets are propped by gum-poles until the masonry becomes self-supporting (see Fig 8). A shortcoming of this method is that frequently, upon construction loading, the form distorts asymmetrically. To preclude this possibility, the use of stiff plywood profiles, onto which the ribs and sheets are attached, is proposed (see Fig 9). The adjustable jacks and the use of short sections, as opposed to one continuous form, are intended to reduce the difficulty of stripping and prevent damage during handling to facilitate maximum reuse.

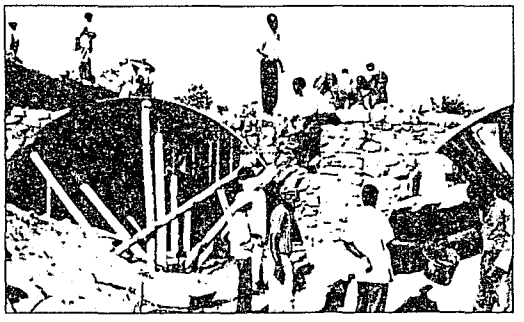

Fig 8: Cranked corrugated iron sheeting and gum poles used as centring in the Northern Province

Another method, claimed to cost only 25 per cent of the previous method, uses 50 mm saplings spanned between temporary masonry profiles over which old cement packets are draped (Rankine et al, 1995; Grobler et al, 1995). However, the sag between points of support and the much rougher finish increases the risk of blockage, reduces hydraulic area and may cause unacceptable turbulence in fast-flowing water.

A Zimbabwean company (Shelton, 1985; dos Santos, 1993) has patented a proprietary precast concrete permanent form shell named a 'shelvert',

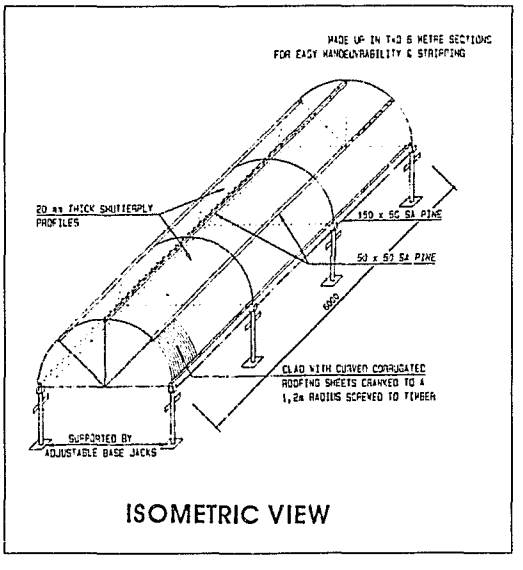

Fig 9: Proposed plywood stiffening to prevent distortion of the circular symmetry

which is delivered to site in two halves. Once joined at the crown the structure is stable and capable of supporting load without any reliance whatsoever upon the strength of the spandrel masonry (see Fig 10). One advantage of the system is that it is not limited to a circular intrados.

The small bridge (shown in Fig 11) at De Vasselot Tsitsikamma National Park became unserviceable as a result of corrosion of its steel liner after 25 to 30 years of service. The top of the liner remains extremely well protected, by zinc and bitumen; however, its bottom, which is subject to abrasion, has corroded away in the high chloride environment. It is proposed that the existing liner be reused as centring for an RMC structure without any additional support. Corrosion will eventually consume the centring, leaving a permanent and attractive RMC arch.

Placing of RMC

With the exception of the smallest streams, construction should be restricted to the dry season, when many southern African rivers cease to flow, to minimize the risk of flood damage whilst the new structure is most vulnerable. Nevertheless, to guard against the freak occurrence of an out-of-season flood, large apertures should be left in the formwork to allow water to pass under the structure. The stockpiling of river sand will also ensure that the contractor is able to continue working should the river start to flow.

The radial placing of stones, with respect to their longitudinal axes, around the apertures (as is shown in Fig 1) is recommended to derive maximum benefit from the bilateral constraint afforded by the flatter surfaces. The placing of stones in horizontal layers (with their longitudinal axes sub-parallel to the principal stress trajectories), as is presently the custom, has been shown (Rankine et al, 1995) to reduce the compressive strength, because the stiff inclusions tend to cleave the matrix apart rather than tie it together.

Arrangements for the curing of masonry, particularly in hot dry weather

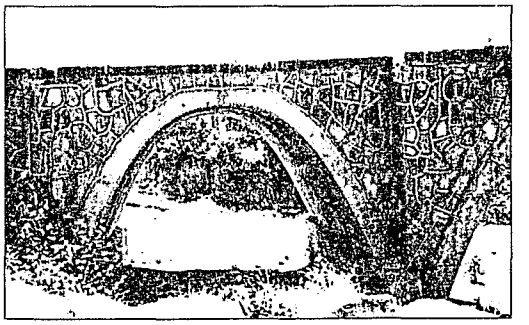

Fig 10: Proprietary precast concrete shells used as permanent formwork in Zimbabwe. The parabolic shape is highly efficient in minimizing tensile stresses in the structure. (Photograph courtesy of Fort Concrete Zimbabwe)

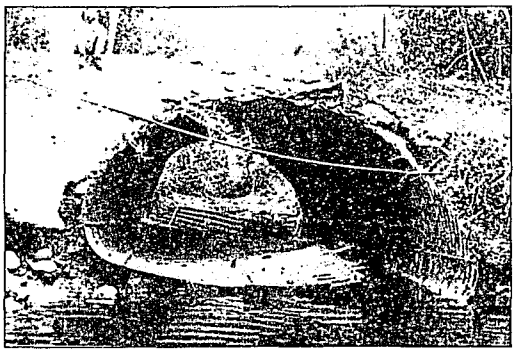

Fig 11: This corroded steel liner can be used as centring for a permanent RMC bridge replacement

and over weekends, are imperative, since interfacial bond, and thus tensile strength of the composite, is dependent upon cure.

Joints in multiple-span bridges

Although joint details in multiple-span RMC bridges have seldom been implemented in practice and no reported structural failures have been attributed to their omission, there is some argument for their insertion. Shelton (1995) cautions against the possibility of the formation of inclined tension cracks created as a consequence of foundation movement and/or temperature drop. The cracks would naturally tend to occur near the thinnest section at the crown and with continual thermal pumping may cause misalignment of the deck (see Fig 12). Varkevisser (1995) recalls an RMC bridge in Makumbura, Zimbabwe, that was cracked right through its crown, probably as a result of differential foundation settlement, yet it remained serviceable for many years.

The provision of vertical contraction joints from foundation to bridge deck between successive openings is an obvious solution to the problem. Such joints may be formed by building alternative sections of masonry and then painting the joint surfaces with lime-wash before building in the remaining intermediate masonry. Although discontinuity of moments and tension is desired at the joints, aggregate interlock should be fostered for the transfer of shear to preclude arch sections from sliding relative to their adjacent neighbours. Consequently, the practice of building these joints against smooth shutters or by the inclusion of planar bond breakers is not recommended.

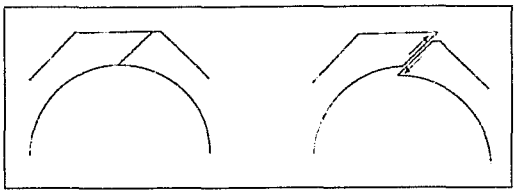

Fig 12: Possible inclined tension crack caused by contraction and/or foundation movement

Road surfaces

In the absence of layer-works, the RMC finish is typically too rough and uneven for use as a running surface for traffic. Screed toppings have been used (Rankine et al, 1995), but they tend to crumble and wear quickly (Addis, 1986). The Zimbabweans recommend a 30 MPa concrete slab about 150 mm thick (unreinforced) and 100 mm thick if mesh reinforced (dos Santos, 1993).

Maintenance

Properly designed and constructed structures are virtually maintenance-free. However, even the apertures of the best structures are prone to becoming fouled and need to be cleared of driftwood and other debris from time to time. Scour, once detected, must be attended to immediately, particularly when the structure is not founded on rock.

Conclusion

The competitive advantage of RMC arch bridges is very dependent on their situation. Ideally, a bedrock foundation and a straight section of stream with moderately inclined banks is required. The positioning and geometry of the structure must be designed to minimize obstruction to the flow of water and debris. A graphical method of determining the position of the line of thrust and a plane strain finite element model for structural analysis are proposed. The finite element model was used to explore the sensitivity of a structure to four different shape apertures of a constant area and the structural response of a circular arch to submersion. A parabolic aperture was found to be optimal in limiting tensile stresses, but is unlikely to succeed the circular arch for the latter's simplicity of construction. The effect of buoyancy was demonstrated to increase tensile stresses within the arch and thereby reduce its capacity to support point loads.

Failure by cracking in tension was considered a possibility and simulated theoretically, revealing that the cracks facilitated the alleviation of tensile stresses. Other modes of failure, including a hinged kinematic mechanism, were considered and shown to be extremely unlikely to occur in the structure investigated. Hence, it would appear that the critical case of the RMC arch is governed not by material strength, but by equilibrium, and that these structures may be designed to provide good service, without the provision of reinforcement.

This paper was submitted in December 1997.

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DISCUSSION ON / BESPREKING OOR

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Joernaal

Die Suid-Afrikaanse Instituut van Siviele Ingenieurswese

Proposed guidelines for the design and construction of rubble masonry concrete arch bridges

The original paper is by R G D Rankine, M Gohnert and R T McCutcheon

Practical design and construction considerations, other than detailed specifications, were discussed with reference to precedent, experimental testing and theoretical modelling. Consideration of several failure mechanisms indicated that the RMC arch problem is primarily one of equilibrium rather than of material strength and that structural stability without reinforcing can be ensured.

T G Kowalski

I am wholly in agreement with the authors of this timely and interesting paper regarding their guidelines and want to contribute certain aspects of an RMC multi-arch bridge designed in 1985 while with the KwaZulu Department of Works, which may be of interest to other practitioners.

My bridge, which I called 'The Ingwe Causeway', crossing the Sundays River in Northern Natal had particularly severe river flow conditions. To illustrate this, a low-level RC slab bridge previously completed a few kilometres further downstream had most of its spans not only damaged and some reinforcing exposed but jammed full with large boulders rolled down by a powerful current. With this in mind a robust design was adopted with all piers shaped to withstand the impact of the rolled smooth boulders, some estimated to be 80 kg to 100 kg each. To cater for these conditions, the massively sized piers had projecting half-round upstream breastworks and elongated fins at the back of each, which in turn were intended to smooth the highly turbulent flow and minimize downstream cavitation observed in several similar cases (Fig 1). The pier centres were at 3,9 m and arch spans 2,6 m, with a 1:4 rise to span ratio. The massive pier size and the proximity of rock permitted one-by-one span erection. The shuttering for the arches consisted of purpose-made segmental steel angle grids (Fig 2 overleaf) assembled in two easily transportable halves and propped with either adjustable props or gum-poles. The top segmental angle surfaces were decked with log halves which together resulted in a rigid and easily stripped shutter.

The bridge was designed for overtopping and was overtopped on several occasions without significant damage. Having retired from KwaZulu to start my small consultancy in 1994, I do not know its present state. I do know, however, that other similar spans have been successfully erected in KwaZulu using these 'standard' shutters.

The authors' paper is additionally useful in focusing on the medium of rubble masonry concrete. Socially it has a valuable but presently underused potential to provide rural employment. And using RMC, as indicated in my references, other types of structures can equally be built cost-effectively in areas of South Africa similar to KwaZulu-Natal.

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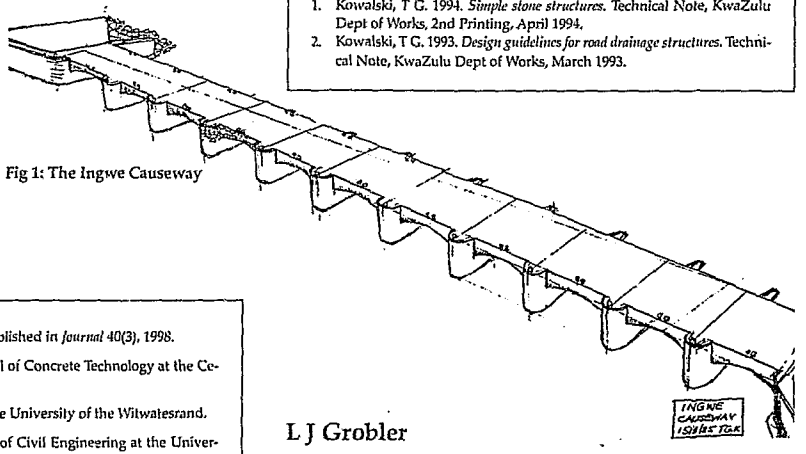

Fig 1: The Ingwe Causeway

This technical paper was originally published in *Journal* 40(3), 1998.

Rod Rankine is a lecturer at the School of Concrete Technology at the Cement and Concrete Institute.

Dr M Gohnert is a senior lecturer at the University of the Witwatersrand.

Prof Robert McCutcheon is Professor of Civil Engineering at the University of the Witwatersrand.

Dr T G Kowalski is with consulting engineers Kowalski and Associates in Camperdown.

L J Grobler is a SAICE member from Worcester.

L J Grobler

I agree that the choice of site is an important consideration in the design of rubble masonry concrete arch bridges. The authors propose that, 'ideally, such a site should offer ... bedrock beneath all arch abutments ...'. I would argue that the importance of the choice of site applies as much to many other bridge design types and should thus not be seen as a factor

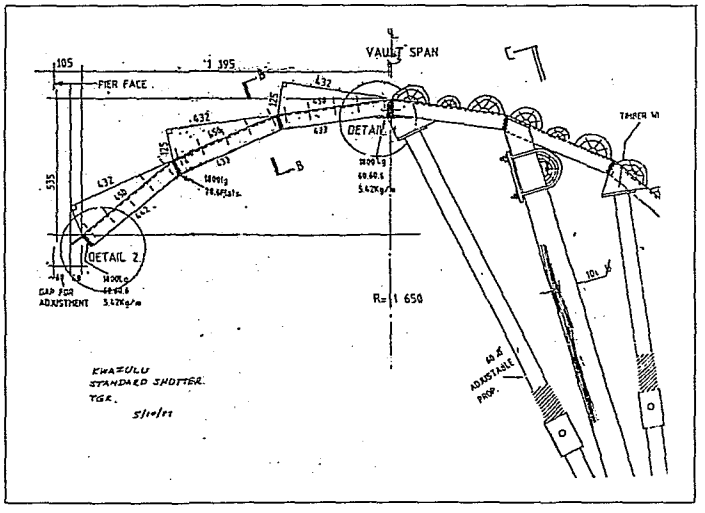

Fig 2: KwaZulu standard shutter

that limits the application potential of this empowerment-friendly design type if compared with other types.

In the absence of bedrock, the authors propose the construction of 'bases or reinforced concrete raft foundations, ... scour protection from one abutment to the other, ... and cut-off walls ...'. I was involved in the design, construction and monitoring of a number of multiple opening masonry arch causeways not founded on rock. Applying the remedies as proposed by the author rendered this design type technically feasible and economically viable compared with alternative designs.

Shelton reminisces about his experience in Zimbabwe that 'never once was he unable to find a site that offered exposed rock right across the river as foundation'. In the planning of more than 40 river crossings in the Northern Province I experienced almost the opposite. The difference in experience appears to stem from specific differences in geology and demography.

The authors consider transient loading as a factor that limits the application of masonry arch bridges because of the relatively small 'aperture fraction'. The paper deliberates on the mechanism of overturning and how to model it. It would, however, be most useful if the limits at which the masonry arch relative to alternative designs become unstable under flood conditions were determined. I argue that the larger weight of the masonry arch counteracts the effect of its larger obstruction. The net effect could well be that it is more stable under flood conditions. I observed several deck and rectangular cell structures being overturned, but have not observed the same in respect of a masonry arch.

The much less expensive arch formwork system of placing wood poles on masonry arch falsework walls is described. The point is made that its 'rougher finish increases the risk of blockage, reduces hydraulic area and may cause unacceptable turbulence'. The paper does not quantify the practical significance of these effects, which is doubtful.

The paper provides adequate proof that cracking of the arch does not affect the stability of the structure. It proposes, however, that the 'obvious solution (to manage cracks) is the provision of vertical contraction joints from foundation to bridge deck between successive openings'. Considering the failure mechanism confirmed by practical observations that cracks normally occur at the crown of the arch, and considering that with differential movement of the footing rotation of adjacent arches occurs in opposite directions around the pier support, to me the obvious solution is to provide contraction joints at the crown. This has the additional benefits that any differential movement of adjacent footings would not cause uncontrolled cracks in the crown, and the thickness of piers can be reduced.

It is clear from the paper that the stone masonry arch design is technically sound and practical to construct; however, the mentioned disadvantages and possible problems – which could deter some engineers from using it – are either not verified for their significance in practice, or apply also to other designs anyway.

Authors' reply

We gratefully acknowledge these contributions from two engineers who are experts on the subject, having had first-hand experience in building several notable structures with rubble masonry.

Mr Grobler's testimony to his success in founding economically viable structures on material other than rock seems encouraging. However, our hesitation in recommending this practice arises from Zimbabwean experience (reported in our paper) where structures, founded at depths in excess of 2 m below riverbed level, have been undermined and washed away. Many of the rubble masonry structures recently built in South Africa are founded on materials other than rock, less than 2 m below riverbed level. Fortunately, many of these structures [illegible]rse small tributary streams that are dry for most of the year and consequently the risk of them being damaged by flooding may be acceptably low. However, as far as we are aware, there is insufficient data available to quantify this risk with reliability. Unfortunately, our study was precluded from investigating this hydrological subject in depth. Research to address this question is urgently required. In the absence of such knowledge, it may therefore be prudent to assume that all flooding streams may render their beds incapable of supporting loads founded within a depth of 2 m.

Mr Grobler's point that the greater mass of an arch structure off-sets its tendency to be displaced by the stream flow is noted, but our paper does make reference to several masonry arch structures that were nevertheless destroyed by transient loading. Hence, we would encourage engineers to check the stability against overturning by taking moments about the downstream toe of the structure assuming total aperture blockage, thereby buoying up the structure and reducing its stabilizing mass.

That our paper fails to quantify the increase in drag coefficient due to the use of wood poles and cement packets as centring is true. The practical effect may be difficult to measure since the rougher finish is more likely to snare driftwood and flotsam Such centring would therefore be better suited to structures with large apertures.

We agree that the provision of joints at the crown of arches (three-pinned arches) would probably best facilitate differential movement of adjacent footings without causing uncontrolled cracks near the crown. The maintenance of shear transfer force across this joint is critical. Thus, it is essential that the joint be constructed with stones projecting to ensure aggregate interlock.

We are aware of Dr Kowalski's first reference and wish to include a sketch (Fig 3) extracted from it which proposes an innovative solution to the problem of maintaining the horizontal reaction to an arch's thrust, by means of a reinforced tie slab, in the absence of bedrock founding material.

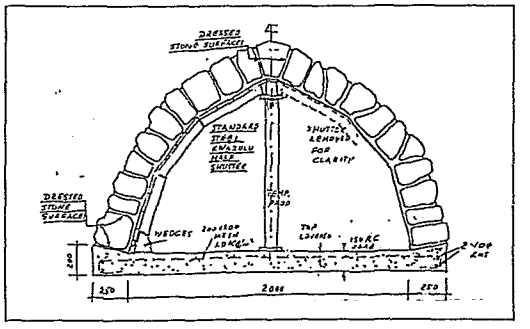

Fig 3: Rubble masonry arch aperture with integrated scour protection reinforced with steel to maintain the horizontal reaction to thrust generated by the arch

Author Rankine R G D
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